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Advances in Concrete Slab Technology

Proceedings of the International Conference on Concrete Slabs held at Dundee University 3-6 April 1979

Edited by
RAVINDRA K. DHIR
and
JOHN G. L. MUNDAY

Organized by
Civil Engineering Department, Dundee University

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Foreword

This book is the Proceedings of the International Conference on Concrete Slabs. The Conference was organised by the Civil Engineering Department, Dundee University, and was held at the University from 3 to 6 April 1979, a venue which turned out to be most appropriate for a conference in the field of concrete technology with the appointment of Professor Adam M. Neville as the Principal and Vice-Chancellor of the University.

The main purpose of the Conference was to provide an international forum for the exchange of information and ideas on existing practice and current research relating to slab technology, as well as to identify areas into which future research efforts should be directed. Since defects commonly occur in concrete slabs worldwide and failures or near-failures are not infrequent it was thought that the Conference could benefit the construction industry as a whole.

The Conference coverage was wide ranging. Professor Adam M. Neville, Principal and Vice-Chancellor of Dundee University, warmly welcomed the delegates to both the Conference and the University, and Mr William T.F. Austin, President of the Concrete Society provided a most stimulating and thought-provoking prologue to the Conference in his Opening Address. Fifty-five papers divided into six Technical Sessions were presented during the first three days of the Conference, each Session being devoted to a major subject area, with the scene being set for each Session by a distinguished keynote speaker. The international flavour of the delegate participation was well matched by that provided by the authors, altogether some twenty-four countries were represented across the world, from Finland to Nigeria and the United States of America to Singapore.

The excellent presentation of papers and the varied delegate interest produced searching discussion in each Session. During the fourth day of the Conference, which was set aside entirely for the Workshop Discussion Sessions, many of the arguments which were cut short by lack of time in the Technical Sessions were reopened and new ideas floated.

The success of the Conference, with an imaginative and ambitious exhibition providing a suitable background environment, was a confirmation of the need for such an event and also a tribute to the relentless efforts of the Organising Committee aided by several members of Dundee University, in particular of the

Civil Engineering Department, and the keen interest in the Conference shown by the authors, Keynote Speakers, Session Chairmen and delegates alike.

The Organising Committee is most grateful to all those who helped to make the Conference such a success, in particular Professor Anthony R. Cusens, former Head of Department of Civil Engineering, and Mr Walter McNicoll, Acting Head of Department of Civil Engineering at the time of the Conference, for their invaluable support. Additionally, the editors would like to express their gratitude to all those who assisted with the editing and final preparation of the Proceedings, especially: Mrs Judith M.E. Edmond, Miss Evelyn F. Clark, Mrs Sandra Nicoll, Mrs Maureen Golden, and Messrs Lean Teik Ong, W.F. Andrew Yap and Arun G. Apte. The editors also greatly appreciate the patience and understanding shown by their wives, Mrs Bharti Dhir and Mrs Grace Munday, both before and since the Conference.

The editing of this voluminous Proceedings has been most time consuming and although every care has been taken, the editors apologise for any errors or inaccuracies which may inadvertently have been overlooked.

Dundee
December, 1979

R.K. Dhir
J.G.L. Munday

Organising Committee

Ravindra K. Dhir (Conference Convener)
Dundee University

R. Malcolm W. Horner
Dundee University

Geoffrey C. Mays
Dundee University

John G.L. Munday
Dundee College of Technology

WELCOMING ADDRESS

by

Adam M. Neville

Principal and Vice-Chancellor, The University, Dundee

Mr. President, ladies and gentlemen. As you know my task here is to welcome you to the University of Dundee; I am very pleased that our University once again has organised, and is host to, an international conference. It is evident that the conference has a large support; I believe that there are about two hundred delegates representing twenty-four countries, which is a very considerable achievement. I personally also recognise the importance of the conference and that is why I am here: as our Chairman said, I had to alter some other engagements and I might just as well confess that last night I came back from ski-ing in Switzerland; this indeed is sacrifice in the cause of science. Well, I am here in the capacity of Principal of the University and therefore you could say a sort of professional welcomer of gatherings, conferences, meetings and so on. I find myself giving this type of address to medics, to historians and to others and it is very nice for me personally to meet engineers for once. I have not been out of engineering for so long that I have to ask you, "What do you do at your conference? Are you really going to talk for three days about concrete slabs?" More than once people have said to me, "You have written a book on concrete, a whole book on concrete?", so you know how I could feel.

Seriously, I am very pleased that our University is host to this conference and there is no doubt that we have a very active Department of Civil Engineering. It was headed by Professor Cusens until he moved to Leeds University, the present acting head is Mr. McNicoll and they and their colleagues are well known. I used to think, and I used to tell everybody, that Leeds was the best Department of Civil Engineering in the United Kingdom; I am just beginning to wonder whether one should not really revise this view. I hope you will all have a good look at Dundee University and tell me that I was wrong, that it is Dundee that takes pride of place.

Well it is not my task to give a long address since the President of the Concrete Society, Bill Austin, will officially open this conference. All I would like to say is, "Welcome to Dundee, welcome to the University of Dundee and I wish you a successful and pleasant time."

OPENING ADDRESS

by

William T. F. Austin

The President, The Concrete Society, U.K.

Mr. Chairman, Principal, ladies and gentlemen, I never know what I think until I hear what I say.

> I have had six friends throughout my life,
> they taught me all I knew,
> their names were 'how' and 'why' and 'when'
> and 'where and 'which' and 'who'.

If we know all the answers to those questions on any subject, we should be all right.

First of all 'who' and 'where'. You have all come from distant parts of the country, and indeed the world, to this conference and there are quite a number of you. That involves quite a lot of resources, indeed if you work it out it corresponds to about a fifth of the working lifetime of any one of us. International conferences are like that and it is up to us to ensure that as we travel home it has all been justified. I am confident that it will be. People only come to these conferences if they are keen and conscientious and have something to teach and learn, probably both. The more one knows about a subject the more one realises that one does not know it all. The self-styled "expert" is so often an example of "X" denoting a has-been and "spurt" being a drip under pressure. A good start has been made by the authors themselves who, together with the references contained in their papers, provide a very good guide as to the 'who' - whom one should contact and where one should look in search of further knowledge. Let us hope that there will be vigorous discussions to add to our stock of knowledge. A valuable part of conferences such as this consists of just meeting and talking to people. May you enjoy this and make some lifelong friends. Life is a poor thing without friends.

The 'how', 'which' and 'why' really form the subject of this conference and I am not going to attempt to compete with the mass of technical information placed before you in the papers presented to this conference. I leave that to the technical sessions; it is better to remain silent and be thought a fool than to open one's mouth and remove all doubt. On looking through the papers, however, it is clear that there is much valuable information upon how to design and construct concrete slabs, what materials to use and why. Occasionally there is justification for asking a further 'why' because the author has not given all the relevant facts, perhaps because he assumes that we know them. What appears obvious to us is not always obvious to someone who lives and works thousands of miles away, and they could be right. Scotland's national poet Burns once wrote:

> 'O wad some Power the giftie gie us
> to see oursels as ithers see us,
> it wad frae mony a blunder free us,
> an' foolish notion.'

For those who are more familiar with Scotch as a drink than Scots as a language, that means 'Oh would some power give us the gift to see ourselves as others see us, it would save us from many a mistake and wrong idea'.

The final question is 'when'. When we are designing something it pays us to reflect for a moment on what we are designing for in terms of time. Many of us have to be like the time lords and move constantly between past, present and future. When we are designing a bridge which we hope will be happy in, say, two hundred years time, it may pay us to consider what type of vehicles it will be called upon to carry in two hunderd years time. Two hundred years ago there were horse carriages, yet many bridges still function adequately. If, on the other hand, it is a power station that we are designing, then I would have thought that in twenty-five to thirty years it would have served its purpose because the machinery which it carries will be worn out. Then it will have to be demolished and although it is hateful to think of taking something down before you have even designed it, sometimes for such things as prestressed concrete one may have to think of that. Research workers tend to study the past, conduct experiments in the present and predict the unknown future. It could well be that somebody will be here from another country who has already experienced the problems which to us are the unknown future. If so, please let us know. To the practising engineer the answer to the question 'when' is so often 'yesterday, sooner if possible'. A plea, therefore, to developers of design methods, please make them as quick and as simple as the conditions of the problem allow. Some advocate the use of computers, some hate them, but whichever view you hold please remember to make a rough check, at least, of the results. If you are experienced you can probably tell at a glance whether the answer is reasonable; if not, you have to make a rough approximate calculation.

It was not for nothing that a computer manufacturer used to put on the console of his computer G.I.G.O. standing for Garbage In, Garbage Out. Please also remember that the introduction of yet another method of design will not be welcomed by busy designers unless it has definite advantages, so if you want to advocate a new method of design let us know what the advantages are.

The subject of this conference is basically how to make concrete slabs technically as good, sound and cheap as possible by using the proper materials, design method and construction techniques, and to indicate methods of repair and maintenance which are effective. However, all this is wasted unless the slabs are constructed with care by skilled craftsmen and form part of a structure properly suited to its purpose. Often this is purely utilitarian such as a road, runway or factory floor slab on the ground. All are subject to wear and the effects of climate and/or traffic. A long life is to be aimed at with the provision for repair and renovation, but on other occasions elegance and aesthetic considerations are involved. This applies particularly in housing, schools, colleges, public buildings and the like. Generally speaking we build things to make people happy, by producing a pleasant home in which to live, a convenient factory in which to work, a safe, comfortable road on which to drive or runway on which to take off or land. We like to think that all these good people want elegant structures. Unfortunately this is not so. Sometimes a client wants the cheapest concrete that he can get which will not actually come to pieces. This not only applies to clients, it sometimes applies to contractors, some of whom have cheated the good tenderers with whom they are competing by allowing in their price only for cheap and nasty materials and workmanship. They will, by many employers, be awarded a contract. If it is your misfortune to be engineer on such a contract there is no need to go out of your way to make this contractor happy; in fact make him unhappy by rejecting his bad work so that he will not trouble us again. While we often have to be content to make structures which are cheap and adequate we prefer to provide a better class of work which is ideally suited to its purpose, durable without maintenance and a joy to look upon. I can illustrate this quite clearly in the following Figures.

Figure 1 shows one way in which people treat slabs. Container park pavements are quite a problem in respect of wear, people will drop down the temporary props of these large lorries, you have all sorts of vehicles which handle the containers and indeed the containers themselves can be quite destructive to your beautiful efforts.

Figure 1 Heavy Loading on a Container Park Pavement from Lorries and Containers

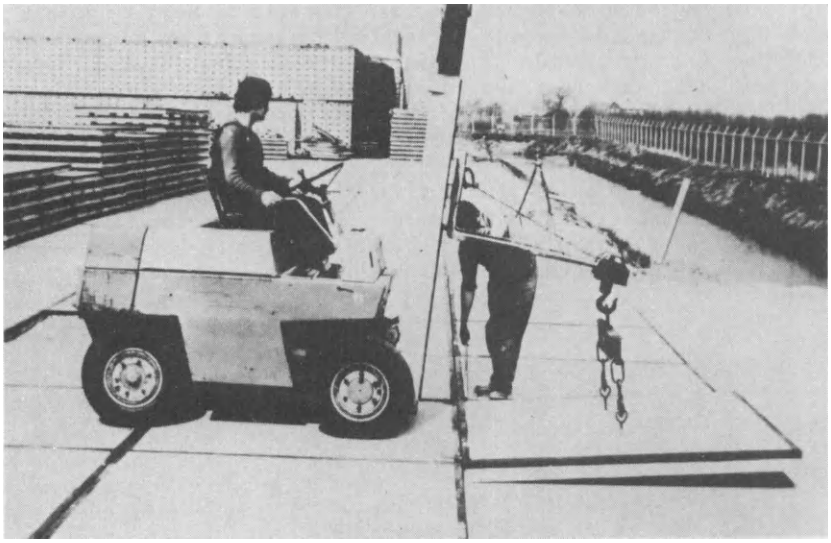

Figure 2 Precast Concrete Slabs in a Container Park

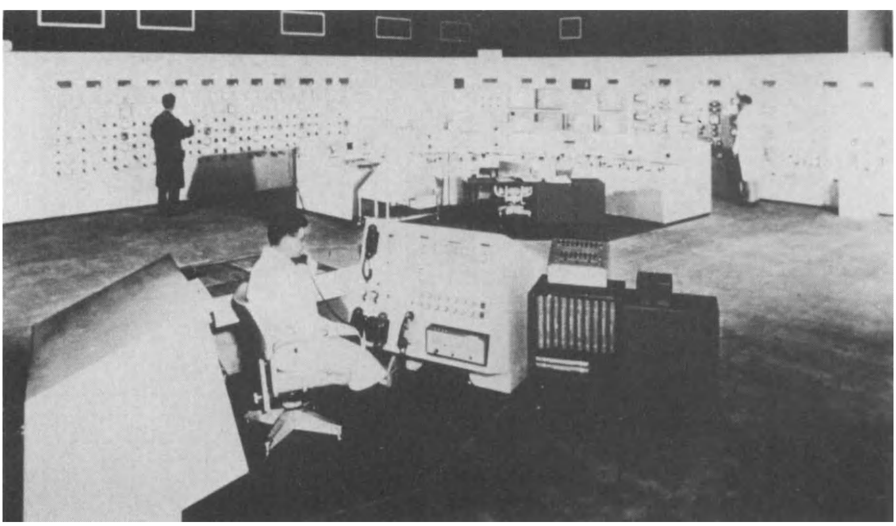

Figure 3 A Control Room in a Power Station

Figure 2 shows a system of precast slabs which are sometimes used in storage areas. They have their advantages and they have their problems, but in-situ concrete also has its problems as you might appreciate from Figure 3. In future I believe quite a lot of our industrial work will be more-or-less automatic and operated from a control room. Figure 3 actually shows a control room in a power station but I am sure that the same principles will apply, which will mean lots of ducts and cables and things. I do not know how you are going to put these in your slabs, but it certainly will give you a headache.

The assembly hall shown in Figure 4 I suspect was tidied up for the photograph - it came out of an architect's brochure. However, you can still see the sort of things that happen to factory floors once they are built.

Moving away from factories to residential buildings, Figure 5 shows a holiday village, one of ten, in the Lanquedoc in France. This one is called Le Grand Motte. These Pyramid blocks are built on what we call in our office the 'stackaplet' principle. For about two months in the year they are completely crowded by holiday makers from Paris and places like that and there is no doubt that one can have a very good holiday there.

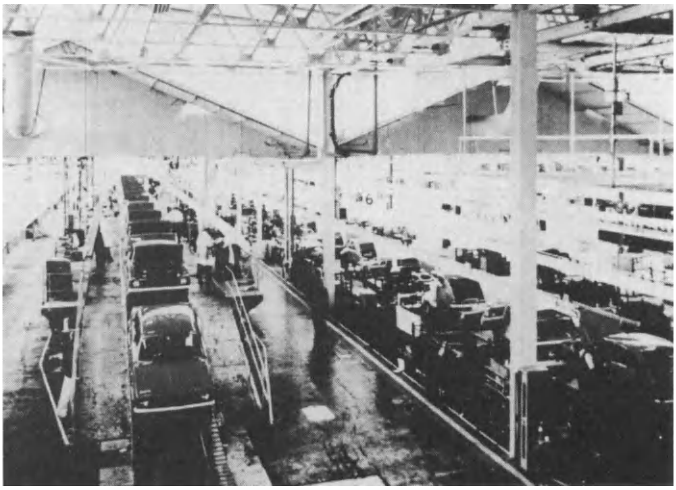

Figure 4 Motor Car Assembly Hall

Figure 5 La Grande Motte Pyramid Block

Figure 6 Paved Courtyard in Bonn

I would like to finish these few Figures with a view through the gates of the courtyard of a church in Bonn, see Figure 6, to remind us that we do not always have to use structural reinforced concrete slabs to get an attractive answer in concrete. Where they are necessary, however, I trust that you will be helped in designing and constructing them by the proceedings of this conference. I will leave you now to have coffee and to enjoy your conference.

Session 1

Concrete and its Constituent Materials

Chairman: R. Colin Deacon

Chief Advisory Engineer, North Region
Cement and Concrete Association, U.K.

Keynote Speaker: Cyril Hobbs

Managing Director
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CONCRETE AND ITS CONSTITUENT MATERIALS

Cyril Hobbs

Managing Director, John Laing Research and Development Ltd., U.K.

Had a conference like this been arranged 30 years ago, what would have been the main themes of papers on concrete as a material? Would they not have been on subjects such as mix design, effects of water-cement ratio, studies of workability, weightbatching, problems of segregation, methods of placing and compaction - all related to conventional concrete, that is, concrete made with natural aggregate, sand, cement and water?

It is interesting, in considering whether research is ever applied, to note that most of these matters have now been absorbed by the construction industry into its accepted technology, and very major advantages in the form of improved concrete qualities, reduced costs and increased confidence in concrete as a material have sprung from them. Today, engineers can base designs on concrete strengths which would have been imprudent 30 years ago. They can develop improved and more sophisticated design methods, as seen from some of the papers presented at this conference, confident that the economies derived from them will not be nullified by the shortcomings of the material used. They can argue about sophisticated Codes of Practice which, 30 years ago, would have been largely irrelevant.

There is still more research needed into conventional concrete, but the big gains of the past 30 years are not likely to be repeated and further improvements will be slow and marginal.

What about 15 years ago? By that time, the research emphasis had changed. Conventional concrete was recognised as having serious limitations in some areas, for example, it was heavy and a poor thermal insulator, so interest was shifting to new forms of concrete, such as aerated concrete or lightweight concrete made with artificial aggregate, to extend the scope for its use over a much wider range. Interest was also developing into the use which could be made of waste material, either as aggregates or, like pulverised fuel ash and ground blast furnace slag, to modify the properties of the cement paste. Concrete, in other words, was changing in the minds of research workers into something using a much wider range of constituent materials and having a greater range of potential properties than conventional concrete.

Some of this research too is now being applied and the use of lightweight aggregate and aerated concrete has grown substantially in the last decade, although they are still only a small proportion of the total consumption; full and unquestioned utilisation, every time they are advantageous, will probably take another decade, at least. It is disappointing, for example, to see how little reference is made in the Design Sessions of this conference to the use of lightweight aggregate concretes for slabs. Without doubt, however, lightweight concretes are beginning to be accepted and respectable, and it will not be very long before engineers are considering them quite naturally as amongst the range of options available for effective design.

This brings us to the present, and the themes of papers of the first Session of the conference. What do we see now? Fibre reinforced concrete, resin impregnated concretes, chemical workability aids, chemically changed cement to counteract shrinkage and sulphur-treated concretes. Add in other work, not presented at this conference, into concretes with plastic and other special aggregates, and we see another large and bewildering extension to the range of possible constituents of concrete, and a further widening of the range of properties of this once simple and familiar material.

Which of these new developments are going to be absorbed and used by the industry? Probably very few in their present form, and those few will, in any case, be absorbed into practice only slowly over the next 20 years or so. The technology of conventional concrete which showed us how to handle and control the putting together of a few simple ingredients, however, is rapidly flowering into a much more sophisticated technology, capable of dealing with confidence with a much wider and more heterogeneous range of potential ingredients and producing combinations of properties suitable for almost any application. From this new technology, therefore, will surely come some new combinations with sufficient advantages to make their use irresistible. Concrete of high tensile strength not requiring reinforcement? Concrete of exceptional durability, free of all kinds of cracking? Higher strengths with lower densities? All are possibilities, all as yet unrealised on any major scale, and all requiring years before they become recognised accepted practices.

What do we find in to-day's papers? Beckett makes a good case for steel fibre reinforced concrete and indicates a number of ways in which such reinforcement improves the performance of concrete, describing a number of field applications. However, there remain considerable difficulties in placing, and in cost terms he states that it will only be in the specialised areas where the material will have any large market. Even in these areas, economies in design are generally needed to offset the increased material cost.

The paper by Swamy and Ali is a further contribution from the authors into the understanding of the effect of fibre reinforcement on the properties of concrete. Indeed the literature in this field is growing apace and with it our ability to design such concretes with confidence. The authors support Beckett's technical claims, but do little to help on the economic front.

The paper by Sarid et al could justifiably be dealt with elsewhere in the conference. It is more a design study to achieve certain construction objectives than a study of ferrocement as a material. Nevertheless, ferrocement, as a sort of halfway house between conventional and fibre reinforced concrete, has shown itself to be a useful product for many purposes.

Swamy et al on lightweight aggregate supports the case for the structural use of lightweight aggregate concretes, but underlines also the problems of getting new materials like this fully accepted by designers. The authors of this paper, and Malhotra on superplasticisers indicate the greater readiness of engineers outside the U.K. to adopt new materials. One would like to feel that, in consequence, the U.K. had a lower level of subsequent failures, but there seems to be little evidence to support such a view.

Malhotra's paper covers a particular difficulty with superplasticised concrete and a means of handling it. Johansson and Petersons repeat and enlarge upon much work done elsewhere, but their paper has the serious shortcoming that the composition of the products studied is not given. The tradition of product secrecy in the concrete additives field is breaking down, but remains a major inhibitor to the growing use of these products. The authors also highlight some of the economic problems relating to these products. The final two papers - Liljestrom on shrinkage compensating cements, and Yuan and Chen on sulphur treated concrete - are appetitie whetters in fields where much more needs to be known before application on any scale can commence. In both cases, the practical difficulties of application to obtain satisfactory results are great.

So, in all cases, the papers promise good things, but indicate also major problems before these new techniques can be adopted. But wasn't it ever so, and shouldn't it, indeed, be so in a forward looking conference of this kind?

In spite of the recent spate of worries about failures caused by new technology, in spite of the growing concern about safety and indemnity, and in spite of restrictive regulations, the intensification of international competition is making more research a national imperative and the urgent application of that research a matter of national survival. So the rate of application of research will continue to accelerate, and some of these developments may be with us earlier than we would now dare to think.

What of the future? Forecasting is a dangerous art, and it would be foolish to try to guess the outcome of research into concrete in the next 30 or 40 years. But it might be useful to name a few objectives which most of us would probably agree were highly desirable, thus giving our researchers something to aim for. For example, whether we like it or not, we must admit that concrete is an ugly material, and we need ways of improving its colour, surface texture, and its general surface quality to get rid of the disparaging durability, especially with regard to its ability to mellow rather than to disfigure with age. For some purposes, and flooring slabs might be a good example, we need to have properties of toughness and flexibility, and we need its working qualities to be such that it is easily placed and can be brought to a high standard of finish without the need for screeding or other operations. We need it crack free and, if possible, movement free, so that the need to lay it in small bays with frequent joints can be obviated. Above all, we need it to have high tensile strength so that it does not require reinforcement.

THE DEVELOPMENT OF STEEL FIBRE REINFORCED CONCRETE WITH PARTICULAR REFERENCE TO ITS USE IN CONCRETE SLABS

R. E. Beckett

Project Manager - Wirand Concrete, National-Standard Company Limited, U.K.

ABSTRACT The background to the requirements for mix design, mixing, transporting, placing, finishing, technical development and field applications of steel fibre reinforced concrete, with particular reference to its use in concrete slabs, is discussed.

INTRODUCTION

The reinforcement of a brittle matrix by the incorporation of fibrous materials is a technique which has seen practical usage for many thousands of years. The reinforcement of mud with sticks probably represents the earliest known application of fibre reinforcement and, in more recent times, the use of natural fibres such as horsehair to improve the mechanical properties of gypsum plaster, or short asbestos fibres in cement paste, producing a wide range of thin sheet products, has been extended into the field of cement mortars and concretes.

The earliest published work appeared just before the turn of the century and this, like many that followed it up until the 1960's, did no more than illustrate that the mechanical properties could be improved by the random dispersion of discontinuous steel reinforcement. These pieces of steel were generally fairly substantial in cross section, such as nails. In the early part of the 1960's Romualdi of the Carnegie-Mellon Institute carried out a programme of work to relate the strength attainment in flexure to fibre spacing, using firstly continuous fine wire of 0.25 mm diameter at a known pitch and subsequently relating these results to results obtained with chopped steel wire fibres, generally 25 mm long, of the same diameter. This work was published and the patents were taken over by the Batelle Development Corporation, who then carried out a more exhaustive testing programme to examine all properties appropriate for the introduction of a new construction material. By 1969, this work had extended the patents and the market was developed by the sale of licence agreements, generally to steel wire manufacturers, and a network of such licensees now operate throughout the world.

MIX DESIGN FOR WIRAND CONCRETE AND MORTAR

When designing a conventional concrete mix, standard practice is to determine the water-cement ratio required for the concrete to attain the minimum specified strength, usually compressive strength, at a certain age, and to determine the aggregate-cement ratio to provide sufficient workability. With steel fibre reinforced mixes this method is not entirely satisfactory in that the addition of steel fibres will increase the compressive strength, by up to 30 to 40 per cent, and will also reduce the workability of the mix, depending on the rate of addition of the fibres. The aspect ratio (length to diameter) of the fibre wire will also affect the strength and workability of the mix, higher aspect ratio fibres generally giving higher strengths and greater reduction in workability for a given fibre content. It is, therefore, not practical to design steel fibre concretes based only on a water-cement ratio; it is also necessary to take into account the quantity and size of steel fibres to be added.

Although the incorporation of fibres increases the compressive strength of the concrete, its main purpose is to improve the flexural strength and impact resistance of the concrete, as well as to provide crack control. Therefore, because it is a universally accepted test, the flexural strength is normally the criterion on which the mix is designed. Based on the results of a large number of tests conducted over a long period of time, a method of mix design for steel fibre concretes and mortars has been developed. The data presented here is extracted from that method and provides an example of a typical solution.

Flexural Strength

The flexural strength of steel fibre concrete can be more than twice that of an unreinforced mix, and in the laboratory even greater increases have been obtained using conventional mixing and compacting techniques. Very high strengths have also been obtained using specialised production processes like hydraulic pressing.

The expected increase in flexural strength for a given rate of addition of fibres with varying ratios can be obtained from Figure 1. For example, if from a plain concrete of a flexural strength of 4 N/mm^2 a steel fibre mix with a flexural strength of 7 N/mm^2 is specified, the required fibre content will range from 1.5 per cent by volume for a fibre with an aspect ratio of 100, to 2.7 per cent by volume for a fibre with an aspect ratio of 60.

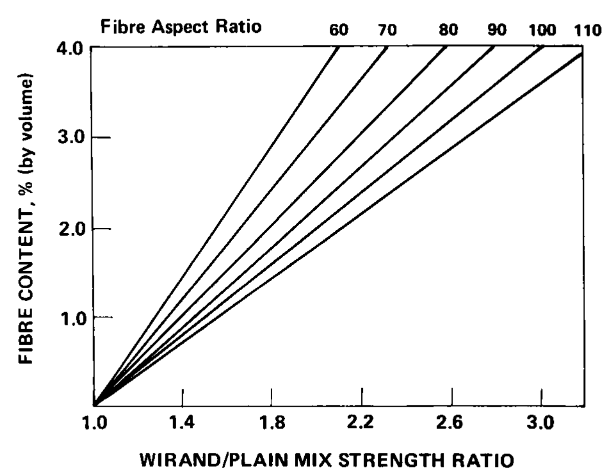

Figure 1 Average Increase in Flexural Strengths of Wirand Concretes and Mortars for varying Aspect Ratio Fibres

From the point of view of economy the high aspect ratio fibre should be chosen. However, the higher the aspect ratio of the fibre the greater the reduction in workability and the greater the difficulty in handling. In view of this, most applications now use fibres with aspect ratios of about 80. These fibres are still reasonably economical, particularly with the larger diameter fibres, and are very much easier to mix and place. Steel fibre concrete and mortar mixes are necessarily richer in cement than conventional concretes. This is because when designing a mix to reach a specified flexural strength, the closer the flexural strength of the plain mix to that required for the steel fibre mix, the more economical will be the resulting concrete.

Although flexural strength is used as the criterion for design, the addition of steel fibres to a mix imparts other desirable properties such as impact, shear and abrasion resistance. There may also be applications where the fibre is used purely for crack control, with no specified flexural strength. In such cases it has been found that a 2 to 3 per cent by mass (0.6 to 0.9 per cent by volume) addition of fibre wire is normally sufficient.

Workability

As stated earlier there is a loss of workability in a mix when steel fibres are added, the amount of loss depending on the rate of addition and aspect ratio of the fibres. As yet no satisfactory method of estimating the probable workability of the steel fibre mix has been developed. The test methods in regular use (i.e. slump, V-B, or compacting factor) are themselves not entirely satisfactory in that although some mixes may be quite workable using normal vibrating techniques, the test values obtained may indicate very low workabilities, particularly for mixes with fairly high rates of addition of fibres. The recommended procedure, therefore, is to add the fibre to an already fairly workable plain mix, i.e. one with a slump of between 100 and 150 mm. This will ensure the fibre reinforced mix retains a reasonable workability without the need to add extra water and, therefore, gives better control of quality.

Concrete or Mortar

The quantity of fibres required in a mix is one of the factors which determines whether a concrete or mortar is used. Theoretically, the maximum size of aggregate should not be greater than the spacing between individual fibres within the mix, given by the formula:

$$13.8d \sqrt{\frac{1}{p}}$$

where d is the fibre diameter and p is the percentage volume fraction of fibres.

However, recent work has indicated that this formula is possibly too restrictive in that a large number of applications have used aggregate sizes up to 16 mm, and in a few cases 32 mm, without any apparent loss of performance when compared with similar fibre mixes using smaller aggregates. Thus there may be some instances where a larger aggregate could be used but this should be confirmed by trial mixes.

Another factor which determines whether a mortar or concrete is used is the thickness of the section. It is generally recommended that a mortar should be used for member thicknesses less than 25 mm and concretes for thicker members.

Sand Content

In a steel fibre concrete mix the sand content is appreciably higher than in a conventional concrete. This is to provide adequate space between the larger aggregate pieces to avoid bunching of the fibres, to provide sufficient paste content to adequately coat the fibres for maximum bond, and to provide for the necessary mobility during placing. It has been found that the sand content will be between 40 and 60 per cent of total aggregate, depending on the grading of the sand.

Admixtures

Most proprietary admixtures that can be used with conventional concrete can also be used with steel fibre concrete, with the exception of calcium chloride which when used at correct dosages tends to induce surface corrosion of the fibres under certain environmental conditions. In a continuing long-term study, however, it does not appear to affect the structural performance of the concrete, neither does corrosion of internal fibres occur. The recommendation that calcium chloride should not be used is based solely on aesthetic grounds.

Physical Properties

Some indication of the physical properties which can be achieved with steel fibre reinforced concrete mixes, in relation to the corresponding plain concrete mixes, can be obtained from Table 1. Typical values for load-deformation relationships and modulus of rupture and impact strength are illustrated in Figures 2 (a) and (b) respectively.

Fibre Types and Grades

Basically there are two types of steel fibres: i) low carbon steel, round or duoform, of diameter 0.25 to 0.65 mm and length 20 to 60 mm and ii) stainless steel, round type 304 and 310, diameter 0.33 mm and length 25 mm. For all fibre types alternative grades of steel, diameters and lengths can be produced on request. Preferred fibres, round or duoform, include:

0.30 mm dia x 25 mm length
0.35 mm dia x 20 mm length
0.40 mm dia x 25 mm length
0.50 mm dia x 40 mm length
0.65 mm dia x 60 mm length

Recent developments have enabled the long mechanical process of wire drawing for the manufacture of steel fibres to be replaced by the low cost method of spinning fibres from a molten bath containing the appropriate grade of steel required, which when quenched produces a roughened surface fibre of approximately 'D' cross-section, of lengths 25 to 40 mm. A range of stainless steels are already produced, with alloy and carbon steels currently being developed.

MIXING, TRANSPORTING, PLACING AND FINISHING

Although steel fibre concrete has ingredients which are common to conventional concrete, the incorporation of fibres does have serious repercussions on the conventional concrete mixing plants. If the correct fibres have been chosen for the properties required of the concrete, but are added without due consideration

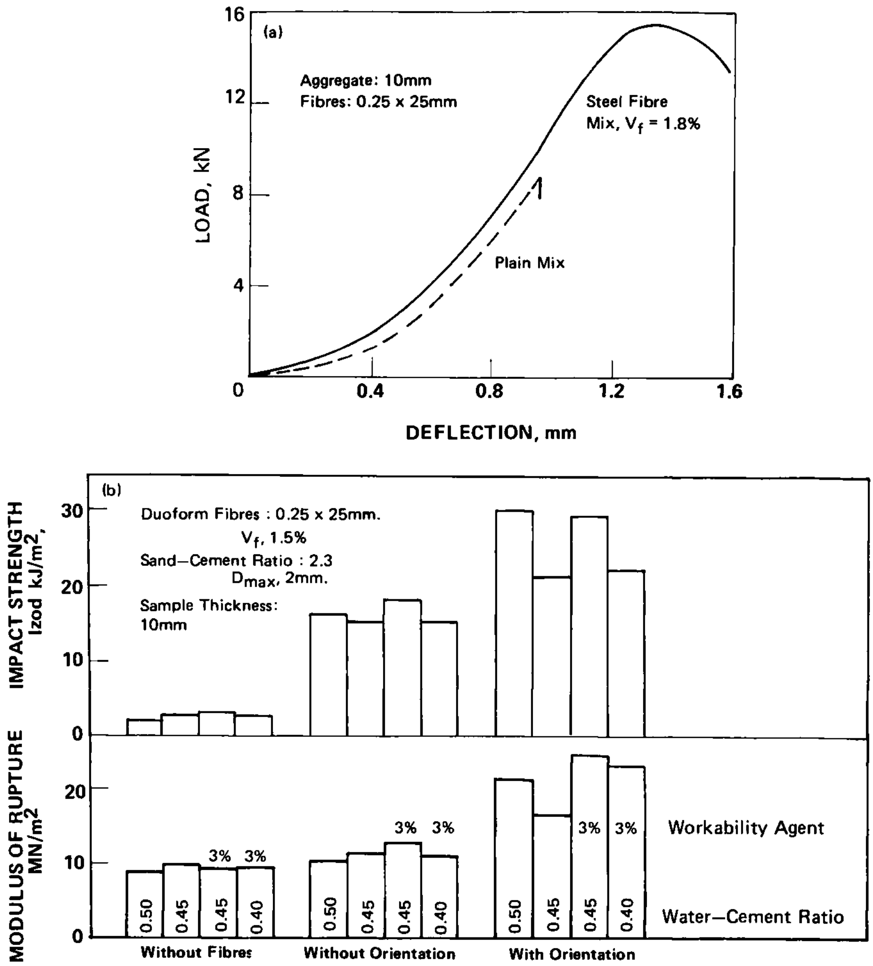

Figure 2 Typical Values showing the Effect of Steel Fibres in Concrete

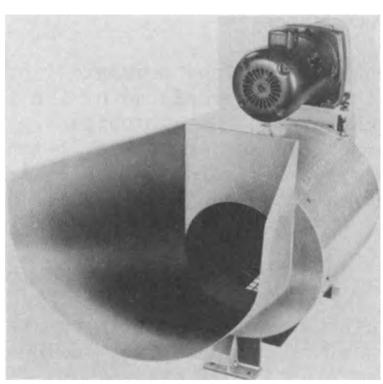

Figure 3 Mark III Fibre Dispenser

Table 1 Physical Characteristics of Steel Fibre Reinforced Concrete

PROPERTY	WIRAND CONCRETE	ADVANTAGE OF WIRAND CONCRETE OVER PLAIN CONCRETE
Flexural Strength, N/mm^2		
Proportional Limit	Up to 12	Can be more than 2 times higher
Ultimate	Up to 17.5	Can be more than 3 times higher
Compressive Strength, N/mm^2	Up to 90	Significant increase
Shear Strength, N/mm^2 (per Battelle)	Up to 5.1	Nearly 2 times higher
Modulus of Elasticity, kN/mm^2 (per B.R.E.)	28.4 to 40.9	-
Poisson's Ratio (per A.C.I. SP-44)	0.20 to 0.26	-
Impact Resistance, J	136	Nearly 3 times higher
Fatigue Endurance Limit Ratio	0.80 to 0.95	More than 70% higher
Abrasion Resistance Index (Sand Blast Test)	2	Twice as resistant
Spalling Resistance Index (Heat Test)	7	Several times greater
Freeze-Thaw Damage (Durability Index)	1.9	90% greater resistance
Thermal Expansion, $x10^{-6}/°C$	10.4 to 11.1	-

to the concrete mix, the widely reported balling-up effect will almost certainly result. To avoid this, steel fibre dispensers should be used. Several licensees have manufactured this equipment and proprietary machines have also been put on the market. One such machine is the Exactorate fibre feeder, produced by the National Concrete Machinery Company, Pennsylvania, U.S.A. The most versatile and easily adaptable to different types of concrete mixer, whether they be tilting drum, reversing drum, pan or readymix plant, is illustrated by the Mark III fibre dispenser, Figure 3, produced by the National-Standard Company Limited, which comprises a motor driven rotary mesh screen, into which the fibres are fed.

Some of the difficulties which have been experienced in the mixing of steel fibre concrete have been overcome, particularly in the field of sprayed concrete and while this is not to be discussed in detail it does illustrate that when the demand for large quantity production of fibre concrete is needed, and very often in difficult and restricted locations, the tenacity of the gunite machinery manufacturers has overcome the main problems associated with the handling and placing of the fibres. An example of this work is the BESAB nozzle feeder for the fibres, as illustrated in Figure 4. Transportation, placing and finishing of steel fibre concrete is carried out in precisely the same way as conventional concrete with the caution that it is not possible to pull the concrete along with a double vibrating beam tamp. It must be carefully spread to avoid a large build up in front of the leading beam.

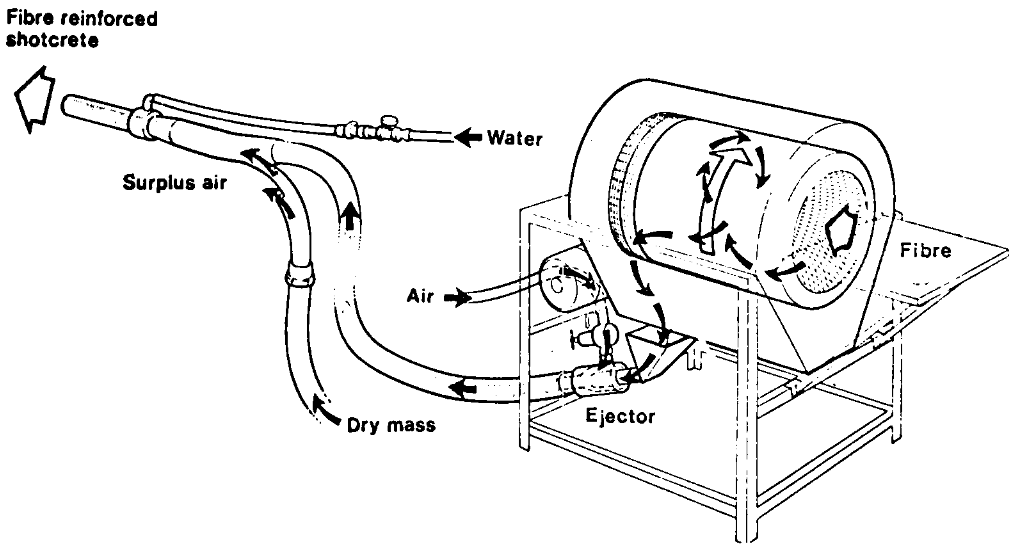

Figure 4 BESAB Nozzle Feeder for Fibres

TECHNICAL DEVELOPMENT

To enable a new material to be accepted by statutory authorities it has been necessary to conduct many field installations using fibre reinforced concrete. Some of these, by the very nature of the difficulties associated with the technical requirements of the slab and the mixing and placing of concrete which were experienced in the early 1970's, have produced results which are not necessarily indicative of the true potential of fibre reinforced concrete.

Two large projects have been undertaken in the U.S.A. to evaluate the behaviour and potential of steel fibre concrete. Firstly, in 1971, the U.S. Army Construction Engineering Research Laboratory tested the concrete pavements used for airfield construction, using both slab-on-grade and overlay pavements, subjected particularly to C5A aircraft traffic movement, weighing in excess of 330 tonnes and transmitting to a single wheel a load of 13 tonnes. Full instrumentation was carried out in the sub-base material and the slab subjected to a pattern loading which could be transmitted to a computer programme, thereby making possible direct comparison with other forms of construction previously tested. This work has been well documented, the conclusions reached being that the performance of the two feasibility test sections indicate that fibre reinforced concrete will function extremely well as a paving material. The in-service performance of the pavements verified the laboratory tests, confirming the application of this material to pavements. The field placements have shown that fibre reinforced concrete can be produced on a relatively large scale without undue difficulties, if proper mixing techniques are used. Some method of mechanically introducing the fibres has to be devised if a large scale placement is encountered. The trials indicated, however, that fibre reinforced concrete will provide an outstanding performance as a paving material, the major benefits probably being best realised by using the material for overlaying and strengthening existing pavements.

The second project was at Greene County, Iowa, in 1972, involving approximately four miles of overlay of fibre reinforced concrete of 50 mm and 75 mm thicknesses on a badly cracked rural highway. Two cement contents of 360 and 450 kg/m^3 with fibre loadings of 36, 60 and 96 kg/m^3 and a 10 mm maximum size aggregate comprising equal amounts of coarse and fines were employed. These overlays were either bonded, unbonded or partially bonded to the 30 year old badly cracked concrete highway. Some experimental work was carried out with chemically compressed cement, as well as with fly ash as a partial replacement of cement in a few sections. Water reducing agents were incorporated in the majority of cases. A comprehensive report of this work has been published by the Iowa Concrete Paving Association in June 1974. In summary they have indicated that the debonded, thicker overlay containing the higher concentration of longer length, larger diameter fibres, has performed more satisfactorily than any other combination.

FIELD INSTALLATIONS

The previously referred to trials have concentrated on airfield and highway installations, but similar trials have been carried out for factory and warehouse floors, bridge decks and overlays, and it is now possible to quote the results of a wide variety of commercial installations which have been carried out either as a total depth construction, or as an overlay.

Airfields

In airfield applications the work at Kennedy Airport, carried out on behalf of the New York/New Jersey Transport Authority, is probably the most comprehensive to date. The overlay was constructed during June 1976, and was an addition to steel fibre concrete used in May 1974. The aggregates and cement for a given size load were dry batched at the concrete batching plant and hauled to the site, using Smith-Mobile inclined axis transit mix trucks.

At the job site, two portable conveyors were set up to transport the steel fibres into the freshly mixed concrete. The small fibre shaker from the U.S. Steel Corporation was placed over the feed hopper of one conveyor. A steel rod grate (opening 75 mm x 100 mm) was placed in the feed hopper of the other conveyor. Both conveyors were petrol engine powered. The B.C.L. fibre feed chutes were used in all of the transit mix trucks for each load of steel fibre reinforced concrete produced. The use of the U.S. Steel shaker did not reduce the total mix time for a 10 m^3 load which ran at about 40 minutes for either feed line.

After the dry mix arrived at the site, the total amount of design mix water was discharged from the truck water tank into the revolving mixer drum. After a homogenous concrete mix was obtained, the transit truck was positioned under the discharge end of the conveyor and the steel fibres loaded into the freshly mixed concrete. During this operation, the drum on the truck-mixer turned at 14 to 15 rpm. As soon as all of the steel fibres were placed in the mix, the truck went to the construction site and discharged its load of steel fibre concrete.

The specified mix design for the steel fibre concrete produced for this project is given in Table 2. Every third load of steel fibre concrete was checked at the construction site for entrained air content, slump and density. For the first load the entrained air content was 4.5 per cent which was low, but the load was not rejected. The amount of air entraining agent was increased to 256 g/m^3 and this produced an entrained air content on average of 5.4 per cent. The average slump for all of the loads checked was 88 mm, the average density was 23,328 kg/m^3 and the average water-cement ratio was 0.405, which was less than the specified water content, but the field inspectors felt that the aggregates contained more moisture than that reported.

Table 2 Details of Mix Used

MATERIAL	CONTENT kg/m^3
Cement, Alpha Type II	443
Aggregate (SSD)	
Coarse, maximum size 10 mm	736
Fine	854
Water	203
Steel Fibre, 0.65 mm dia x 60 mm length	103
Sika Air Entraining Agent	0.220*
Pozzolith 100 Water Reducing/Set Retarder	1.105
Slump: 75 to 100 mm	

*Entrained Air, 6 ± 1%.

The overlay was a 30.5 m long by 38 m wide extension of the runway overlay constructed with steel fibre reinforced concrete in May 1974. The area for repair was stripped of the existing asphalt concrete overlay down to the original concrete pavement. The surface was cleaned and a few deteriorated areas repaired with a concrete grout mix. A double thickness of 0.15 mm polyethylene sheet was placed on the old concrete surface as a bond breaker. The unbonded overlay was formed in five lanes, 7.5 m wide, except for lane 5, which contained a small wing at the North West corner. Smooth faced (butt) sliding dowel construction joints were formed between lanes 1 and 2, 3 and 4, and 4 and 5. A sliding dowel key joint was formed between lanes 2 and 3. The North end of the installation was tied into the existing steel fibre concrete overlay with dowels only. The surface was textured with a hand broom finish. All lanes were cured under wet hessian for 7 days. The project was completed in five working days, plus the 7 day cure for the last lane placed, and the area was open to aircraft traffic 21 days after placement.

Factory/Warehouse Floors

This is probably best illustrated by a project undertaken in 1974 for a 13,000 m^2 warehouse floor. The total thickness of concrete was 150 mm comprising 100 mm of mesh reinforced conventional concrete, topped with some 50 mm of steel fibre reinforced concrete. The floor was finished by a double beam vibrator and then by power floating. The steel fibre mix was a 1:2:2 with 2 per cent by mass of 0.3 mm dia x 25 mm long fibres. Steel fibre reinforced concrete was chosen firstly because steel racking was to be fixed to the floor and it was specified that drill holes should not touch any mesh reinforcement, and secondly because an increased impact and abrasion resistance was required.

Rail Supports

Thinner concrete supports for rails in a British Rail tunnel project in Scotland have been achieved by the use of steel fibre reinforced concrete, which is particularly significant since headroom was restricted and it was necessary to eliminate the timber/concrete sleepers and ballast previously used. No published report is available of this work but it did involve ingenuity in getting ready-mixed concrete containing fibres transferred from the mixer truck to rail cars, which transported the concrete along the track to the position in the tunnel where the steel fibre concrete was to be placed.

In addition to the examples described above there have been many other field installations of steel fibre reinforced concrete, both as slabs-on-grade and as overlays.

CONSTRUCTION CONSIDERATIONS

A concrete slab on-grade is required to carry the traffic imposed on it and satisfactorily transfer that load through to the underlying ground. Traditionally, concrete slabs have been divided into a series of regular bays, whose dimensions are restricted by the amount of reinforcement which has been included either for structural strength improvement or anti-crack purposes. The use of steel fibre reinforced concrete can meet both these requirements.

Total depth construction of fibre reinforced concrete will exert crack control properties for the top surface as well as improve the spall and wear characteristics of the exposed concrete and, at the same time, increase the flexural strength throughout the slab depth. Since the material is reinforced in this way throughout its section, it has been shown that it is possible to reduce the overall depth of construction to approximately one-half of that which would have been used with conventional methods. Provided the ground has load carrying capacity sufficient to support the reduced spread of load caused by a reduction in slab thickness, it is not necessary to increase the hardcore layer under the concrete slab. However, if there were to be any doubt about the load carrying ability of the ground, the total depth of construction, i.e. concrete plus hardcore, could be maintained with the reduced thickness of concrete slab being compensated for by an increased thickness of hardcore.

The spacing of joints has been debated for some considerable time and experience has shown that it is possible to reduce the number of joints, particularly in the controlled environment of factory and warehouse flooring. However, a recommendation currently in force is that interpreting the fibre concentration to a weight of reinforcement per square metre, and using the recommendations for concrete ground floor slabs, as published by the Cement and Concrete Association, the joint spacing will very much follow the pattern of those used in conventional mesh reinforced slabs. This is particularly so now that the construction technique of a plain concrete lower layer, approximately two-thirds the depth of construction, with the top third as fibre reinforced concrete, replacing the anti-crack mesh, has become an economic consideration. It should also be noted, however, that the mixes used for fibre reinforced concrete, generally speaking, do contain more cement than is normally used in mesh reinforced slabs.

Load is transferred between adjacent bays of fibre reinforced concrete by the conventional dowel bar construction, as for normal mesh reinforced slabs when a monolithic topping is employed. In the total depth construction, where the slab is very much reduced in thickness, it is sometimes very difficult to incorporate these dowels and a joggle joint is probably the only way in which any load transfer can take place. The improved shear strength characteristics of the concrete will allow for this form of construction to be effective.

The widely publicised ability of steel fibre reinforced concrete to control cracking contains a misconception, as in the early stage of life of the slab the concrete in its plastic state cannot benefit from the incorporation of steel fibres and it must be emphasised that every care must be taken to avoid early drying shrinkage. Chemical surface treatment or the use of damp hessian have both proved to be quite satisfactory. Very thin slabs with high cement content mixes have in the past been prone to edge curling and in these instances it has been found necessary to reduce the bay sizes considerably.

DESIGN TECHNIQUES

A design project was undertaken by General Analytics Inc., in 1971 for the Battelle Development Corporation to investigate the utilisation of steel fibre reinforced concrete in bridge decks. The main points which emerged from this work are summarised below.

The use of steel fibre reinforced concrete in place of conventional reinforced concrete for the top 50 mm of a standard bridge deck slab provides a crack resistant surface, with the capacity of the deck slab remaining unchanged. In this application it is recommended that the top layer of conventional reinforcement be lowered an additional 25 mm, with the location 75 mm from the top surface.

If steel fibre concrete is used in the top 50 mm and the tensile strength attribute of steel fibre reinforced concrete is utilised, the top layer of conventional reinforcing bars can be eliminated. This would essentially reduce by 50 per cent the amount of conventional reinforcing bars used and simplify the placing of concrete.

The use of the two approaches discussed above does not alter the total thickness of the bridge deck slab from that which is normally used. Therefore, all existing standard details and drawings for other bridge components should not be affected and can be utilised and any previously designed structures not under contract could be easily changed to utilise the crack resistant qualities of steel fibre reinforced concrete.

On the other hand, if the tensile strength of steel fibre reinforced concrete is utilised to its allowable stress, bridge decks made entirely of steel fibre concrete, without any conventional reinforcement, are feasible for stringer spacings from 1.4 to 2.4 m, with from 38 to 50 mm reduction in the overall thickness of the deck slab. This would provide a somewhat lighter deck slab, while still providing a deck stiffness equal to or greater than that of the standard deck slab. Considerable work into the use of steel fibre reinforced concrete is in hand at many Universities and research centres around the world and the results of these studies are, or will be, available from the persons concerned.

ECONOMICS

A comparison of material and laying costs of conventional ground floor slabs with steel fibre reinforced concrete alternatives is given below.

Conventional Slabs

1. 150 mm thick with layer of B283 mesh.
2. 150 mm total thickness, 1 layer B283 mesh and 19 mm granolithic topping.
3. 150 mm thick with 2 layers B283 mesh.

Steel Fibre Alternatives

100 mm full depth with 3 per cent of steel fibres, by mass.
75 mm full depth with 3 per cent of steel fibres, by mass.
150 mm total depth with 1 layer B283 mesh, 50 mm topping containing 2 per cent steel fibres, by mass.

Material Costs Taken

Granolithic topping at 19 mm	£ 0.50/m^2
Concrete (conventional)	£17.50/m^3
Concrete (steel fibre)	£25.90/m^3
Mesh B283	£ 0.88/m^2 List

Labour Costs for Laying Concrete

150 mm slab, 1 x B283 and granolithic topped	£2.26/m^2
150 mm slab and 1 x B283	£1.33/m^2
150 mm slab and 1 x B283 and 3 coats Lithurin	£1.99/m^2

In the following examples, the maximum cost per tonne of steel fibre has been calculated for the total cost/m^2 of the alternatives to be equal to that for the conventional slab. Fibre inclusion at 40 kg/m^3 and 75 kg/m^3.

Alternative for 150 mm Slab + 1 x B283

	At 40 kg/m^3	At 75 kg/m^3
100 mm steel fibre slab	£466	£248
75 mm steel fibre slab	£837	£448

With 20 per cent Discount on Mesh Prices

100 mm steel fibre slab	£416	£222
75 mm steel fibre slab	£772	£413

Alternative for 150 mm Slab Granolithic Topped

100 mm steel fibre slab	£572	£306
75 mm steel fibre slab	£977	£525
100 mm slab + 50 mm steel fibre concrete	£245	-

With 20 per cent Discount on Mesh Price

100 mm steel fibre slab	£523	£279
75 mm steel fibre slab	£914	£490
100 mm slab + 50 mm steel fibre concrete	£245	-

Alternative for 150 mm + 2 x B283

100 mm steel fibre slab	£774	£412
100 mm slab + 50 mm steel fibre concrete	£567	-

With 20 per cent Discount on Mesh Price

100 mm steel fibre slab	£675	£359
100 mm slab + 50 mm steel fibre concrete	£485	-

The above costs are based on using 0.5 mm dia. fibres which have an ex-works price of £570/tonne. If one uses a fibre inclusion of 75 kg/m^3 then it can be seen that steel fibre reinforced concrete is not competitive on a direct cost basis.

Averaging All the Above Costs and Comparing to Current Price

Steel fibre reinforced concrete at 40 kg/m^3 versus mesh at list: Current price is just about competitive.

Steel fibre reinforced concrete at 40 kg/m^3 versus mesh at list less 20 per cent: Reduction of 9 per cent necessary.

Steel fibre reinforced concrete at 75 kg/m^3 versus mesh at list: Reduction of 35 per cent necessary.

Steel fibre reinforced concrete at 75 kg/m^3 versus mesh at list less 20 per cent: Reduction of 41 per cent necessary.

The costs of steel fibre reinforced concretes have been calculated without regard to their performance in service. Where a hard, wear resistant surface is needed, particularly in industrial locations such as steelworks, very expensive alternatives to steel fibre concrete have in the past been used with moderate success. Comparative studies by engineers have proved conclusively that although the initial cost might seem much higher, the very much better long term performance compared to the alternatives has resulted in a cost-performance position which shows that steel fibre reinforced concrete is more economic.

It must, therefore, be concluded that for the present, with current steel wire prices, which continue to rise, although very often less significantly than does the price of mesh reinforcement, it will only be in specialised areas that the steel fibres will have any large market. However, the introduction of the low cost production technique of melt extraction will improve the economics of fibre reinforced concrete construction to be more compatible with the alternative of mesh reinforcement.

CURRENT PROJECTS AND USES

Application areas for steel fibre concrete are given below.

Tunnel linings and rock stabilisation. Speed of application through guniting process. In one tunnelling application increases of the order of six times were reported when guniting replaced traditional timber lagging and hard pack techniques. Thinner steel fibre sections following more closely the rock profile reduce material waste. Predictable, non-catastrophic mode of failure offering important safety advantages. Improved strata control through early sealing of exposed rock. Competitive costs demonstrated in numerous applications.

Concrete pipes. Improvement in manufacturing process. Less surface crazing - fewer in-plant breaks. Greater strength makes possible the use of thinner lighter sections. Easier handling on site through lighter weight.

Roads/airfield runways. Thinner overlays can be quickly laid by slipforming or paving train techniques, this results in less disturbance to existing services and adjacent areas. Cost competitive with bituminous products in areas where curing period can be tolerated.

Precast products. Savings of the order of 50 per cent were recorded for a product manufactured in steel fibre concrete as an alternative to cast iron. Thinner sections mean easier handling through reduction in weight. Mechanised production through hydraulic pressing techniques. Improvements in serviceability through physical properties.

Flooring. The reduced slab thickness possible with total depth construction offers material savings. Monolithic wearing surface becomes more durable, with greater impact and abrasion resistance because of the use of smaller aggregate size, higher cement content and the steel fibres. Steel fibre reinforced concrete slabs can be laid in larger bays thus reducing costs for laying and jointing materials. Reinforcement extends to the top edges of the slab, this controls surface spalling which results in less cracking and breaking up at the edges of joints. Steeper temperature gradient reduces effect of temperature change. Need for granolithic surfacing often eliminated.

Pipe coating. Greater flexural strength and impact resistance gives increased performance characteristics. Mechanised manufacturing process offers overall economy in comparison with traditional reinforcement techniques.

Refractory concretes. Extended high temperature service life through crack control and spall resistance. Increases of the order of 3 times are typical but in one application furnace doors, which regularly failed in 5 to 6 weeks, have been replaced with steel fibre concrete doors which are still in use after several years. Reduction in maintenance and replacement costs and more predictable life particularly under thermal shock conditions.

The work to date has been encouraging, not least as a consequence of the need in many parts of the world to find an alternative fibre to asbestos, as used in asbestos cement products. The work here is generally for thin sheet applications, and the effective use in magnetic orientation to improve the characteristic strengths of steel fibre reinforced mortar mixes has been very effectively demonstrated by the work carried out at the Innovations Institute in Stockholm.

ACKNOWLEDGEMENT The information received from various sources around the world is greatly appreciated.

INFLUENCE OF STEEL FIBRE-REINFORCEMENT ON THE SHEAR STRENGTH OF SLAB-COLUMN CONNECTIONS

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ABSTRACT Tests are reported on the influence of fibre reinforcement on the punching shear strength of reinforced concrete slab-column connections. It is shown that fibres not only delay and control the development of tension cracks but that they also substantially reduce all the deformations at all stages of loading. The service load that can be sustained for a given serviceability criterion is thereby increased by 25 to 50 per cent. Both the first crack load and the ultimate punching shear strength are enhanced, and catastrophic failures are changed into gradual failures. The fibres enhance post-yield ductility by about 200 per cent and the energy absorption characteristic by about 400 per cent.

INTRODUCTION

Punching shear failures in reinforced concrete flat slabs and flat plates are usually sudden and catastrophic in nature. Such failures are undesirable since they do not allow an overall yield line mechanism to develop before punching. In design, the problem of punching shear failures in slab-column connections is taken care of by thickening the slab, increasing the flexural reinforcement near the column or by providing suitable shear reinforcement. In this investigation, the punching shear strength of slab-column connections is improved by the addition of short, discrete steel fibres, uniformly dispersed and randomly oriented, to the concrete.

It is well established that the presence of fibres in concrete increases its tensile strength, ductility and crack control characteristics (1). The most unique property of the fibre reinforcement is its capacity to reduce the stress concentrations at the tips of flexural as well as shear cracks and thereby to slow down the propagation of such cracks. Fibre reinforcement can therefore be successfully used to increase the first crack strength and improve the serviceability characteristics of structural members in flexure (2). This means, therefore, that fibre reinforcement should also be able to effectively resist and control the varying direction and magnitude of principal tensile stresses causing shear failure in various concrete elements. Tests on rectangular, T and I beams show that fibre reinforcement can effectively act as shear reinforcement and increase their shear resistance (3,4,5,6). Studies on the mechanism of shear transfer show that fibre reinforcement can increase substantially the contributions to shear resistance from the compression zone, dowel action and aggregate interlocking (7,8).

Another potential use for fibre concrete as shear reinforcement is in the punching shear zone in concrete slabs where conventional shear reinforcement is difficult, and often impractical, to place. Tests on slab-column connections show that fibre reinforcement increases the shear resistance, ductility and energy absorption properties of the structural member (9,10,11). However, there is no systematic test data to show the influence of fibre reinforcement on the shear strength of slab-column connections. The tests reported in this paper form part of a major study into this area.

EXPERIMENTAL PROGRAMME

Size of Test Slab

The prototype selected for this study was a flat-plate structure with equal column spacing at 4.0m centres in both directions, see Figure 1. The slab is supported on 150mm x 150mm columns, and is adequately overdesigned for shear failure to occur.

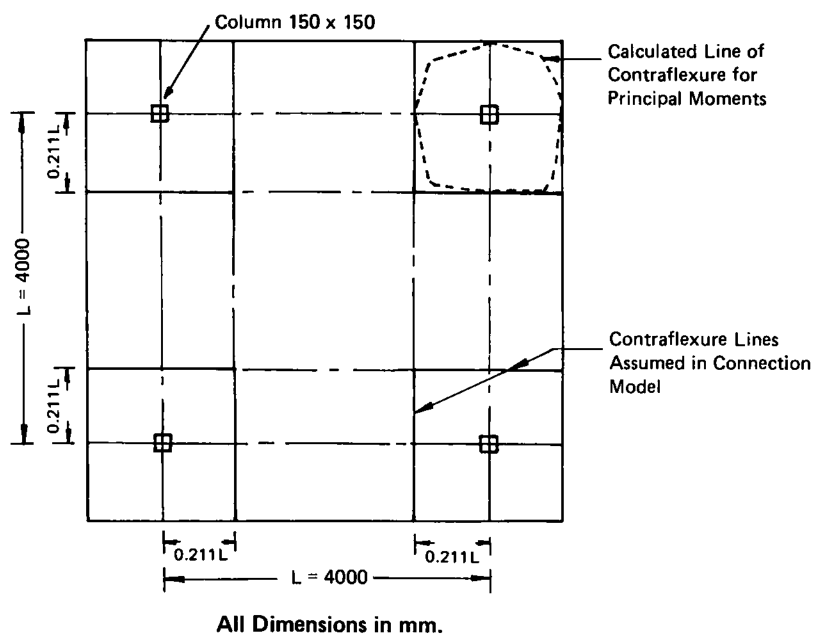


Figure 1 Slab System Showing Location of Contraflexure Lines

The connection specimens tested in this investigation are one-to-one full-scale models of the prototype connection and adjacent slab areas in order to avoid any possible size effects. This also avoids the possibility of distortion of the shear and flexural strengths from the higher than normal ratio of tensile to compressive strength of micro-concrete used for small scale models.

Reinforcement and Concrete Mix Details

All the test specimens were overdesigned in relation to the slab flexural strength and had the same percentage of steel reinforcement, see Figure 2. The steel fibres were crimped, 0.5mm diameter x 50mm long, and used in volume fractions varying from 0 per cent in slab S1 to 1.2 per cent in slab S4.

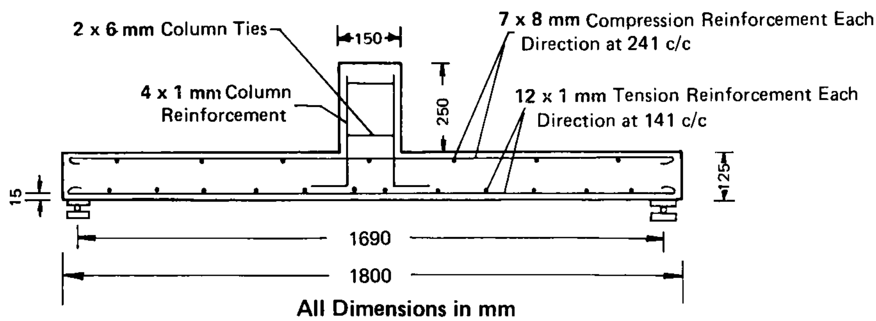


Figure 2 Steel Reinforcement for Slabs S1 to S4

The concrete used in the slabs was a fly ash concrete with mix proportions of 0.7:0.3:1.8:2.2 (cement:fly ash:sand:gravel) with a water to (cement + fly ash) ratio of 0.47, all by mass. A plasticizing admixture was used at the rate of 2.29kg per m^3 of concrete. Ordinary Portland cement was used with washed river sand and 10mm maximum size aggregate. The concrete mix was designed to produce a 28-day compressive strength of 45 N/mm^2 for 150mm cubes air dried in the laboratory. All batches of concrete had a measured slump of 98 to 107mm for plain concrete and 84 to 42mm for concrete with 0.6 per cent and 1.2 per cent steel fibre volume respectively.

Testing Arrangements

The slabs were simply supported on all four sides and loaded centrally through the stub columns, Figure 2. Rubber packing pieces were provided immediately under the slab surfaces to ensure uniform contact along the supports. Extensive measurements of steel and concrete strains, deflection, and rotation of the slabs were recorded at several sections throughout the range of loading. In addition, the crack patterns and failure modes of the slabs were recorded carefully.

TEST RESULTS AND DISCUSSION

From the extensive data obtained from the several series of tests carried out in this programme, only essential data relevant to the subject of the paper are discussed here.

Crack Patterns

Both research and practical experience show that crack widths in concrete could be reduced considerably by reducing the crack spacing. In fibre concrete elements, the fibres can be expected to act in a similar manner and restrain crack widening. In the plain concrete slab S1, the first visible crack occurred in the tension face under the reinforcement in both directions and around the column stub at about 18 per cent of the maximum load. The presence of fibres delayed the formation of the first crack to about 22 per cent of the maximum load in slab S3 with 0.9 per cent fibre volume and 19 per cent of the maximum load in slab S4 with 1.2 per cent fibre volume. In the plain concrete slab the tension cracks began to develop in the diagonal direction towards the slab corners at about 30 per cent of the maximum load, and the lateral cracks (perpendicular to the sides of the slab) appeared at about 55 per cent of the maximum load. These cracks widened with further increase in load. In the fibre concrete slab connections, the diagonal cracks and the lateral cracks appeared at about 45 per cent and 60 per cent of the maximum load respectively.

The crack patterns were generally observed to be about the same for the fibre concrete connections as for the plain concrete slab connection, except that in the former, the cracks were much finer and more in number than in the plain concrete slab connection S1. In the fibre concrete slabs, and in particular in slabs S3 (0.9 per cent fibre volume) and S4 (1.2 per cent fibre volume), some of the diagonal and lateral cracks tended to be partially discontinuous, whereas in the plain concrete slab connection all the cracks were completely continuous, Figure 3(a) and 3(c).

Failure Modes

All the slabs failed in punching shear, Figure 3. None of the slabs had any cracks on the compression face except for the punching lines directly in the vicinity of the column faces.

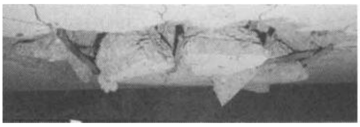
In the plain concrete slab connection, the punching failure was complete and sudden. In the fibre concrete slab connections, the punching shear failure was gradual, and the punching perimeter was bigger. The process of punching was incomplete in slab S4, Figure 3. In these slab connections, the slabs cracked in an almost circular perimeter whereas in the plain concrete connections, the perimeter tended to be more square in plan, Figure 3.

Deformation Characteristics Under Load

Deflection Behaviour

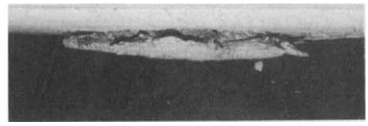
Figure 4 shows typical load-centre deflection curves of the slab connections without and with fibre reinforcement. The first crack load shown in the Figure refers to the visually observed load, while the service load (118.1 kN) and the ultimate design load (137.5 kN) are those based on CP110 (12). The load-deflection curves show that the presence of fibre reinforcement causes substantial reductions in the centre deflections. At the first crack, service and ultimate design loads, the reduction in deflection varied from 5 to 33 per cent, 20 to 38 per cent and 21 to 36 per cent respectively as the fibre volume content increased from 0.6 per cent (slab S2) to 1.2 per cent (slab S4).

The increase in the service loads of the fibre concrete slab connections, taking the deflection at the service load of the plain concrete connection S1 as the criterion of serviceability, is given in Table 1. Considering that 137.5 kN is the ultimate design load of slab connection S1 based on CP110, both slabs S3 and S4 will have service loads higher than the ultimate design load of the plain concrete slab connection. The practical advantages of the presence of fibre reinforcement are thus obvious.



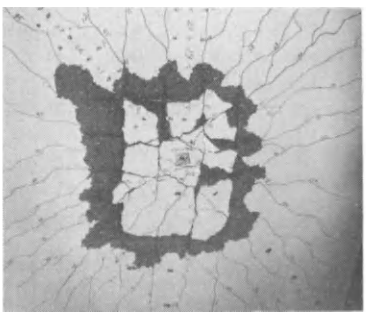
Slab S1, V_f = 0.0%

(a) Sudden Punching Failure



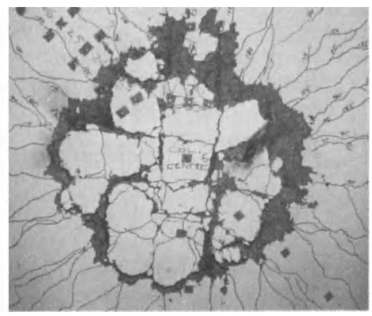
Slab S2, V_f = 0.6%

(b) Gradual Punching Failure



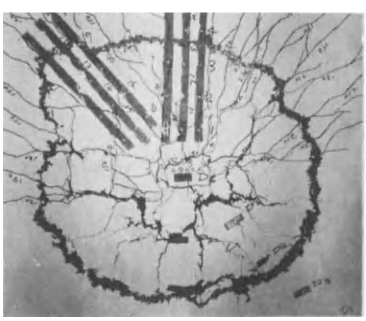
Slab S1, V_f = 0.0%

(c) Complete Sudden Punching



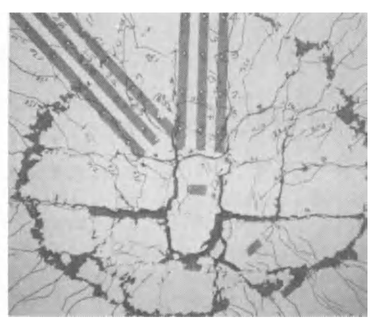
Slab S2, V_f = 0.6%

(d) Gradual Complete Punching Failure



Slab S3, V_f = 0.9%

(e) Gradual Complete Punching Failure



Slab S4, V_f = 1.2%

(f) Gradual Incomplete Punching Failure

Figure 3 Failure Patterns of Flat Slabs with and without Fibre Reinforcement

Table 1 Service Load Based on Deflection Criterion

SLAB NUMBER	STEEL FIBRE VOLUME %	CUBE* STRENGTH N/mm^2	FIRST CRACK LOAD kN	DEFLECTION mm	SERVICE LOAD kN	INCREASE IN SERVICE LOAD %
S1	0	50.7	35.0	6.72	118.1**	0
S2	0.6	48.7	43.1	6.72	113.4	13.0
S3	0.9	47.2	56.9	6.72	147.8	25.1
S4	1.2	46.1	53.1	6.72	159.2	34.8

*Cube strength at 28 days. **Slab S1 service load 118.1 kN as in CP110.

Steel and Concrete Strains

Figures 5 and 6 show the tension and compression steel strains measured near the centre of the slabs and Figure 7, the concrete compressive strain at the top of the slabs. The steel and concrete strains were all very small in the elastic stage of loading until the first crack appeared, then they increased steadily and then rapidly, as yielding approached. Yielding of the tension reinforcement near the centre occurred for all the slabs at about three-quarters of the corresponding maximum load.

The steel strains in the compression reinforcement were initially compressive and gradually changed into tension with further increase in load. Similar phenomena in the change of concrete strains at the compression face have been observed in reinforced concrete beams failing in shear, and the load at which the strain changes from compression to tension is indicative of the load at which diagonal tension cracking occurs within the slab. The formation of the diagonal tension crack thus causes a redistribution of stresses and a change in the curvature at the top of the slabs. None of the compression reinforcement, however, reached their yield strains, see Figure 6.

In the fibre concrete slabs the concrete strains at failure exceeded 3500×10^{-6} whereas in the plain concrete slab connection, the maximum strain remained below this value. Immediately after failure, the concrete strains suddenly decreased due to slab rebound and load release; however, the load drop in slabs with fibres was less than that in the plain concrete slab connection.

Comparison of Deformations and Role of Fibre Reinforcement

The primary role of the fibre reinforcement is to act as a crack arresting mechanism and as a consequence its presence reduces all deformations. The maximum measured values of tensile steel strain, concrete compressive strain and the rotation for all the slabs at first crack load, at service load (118.1 kN) and at maximum load are given in Table 2. It is clear that fibre reinforcement substantially reduces all deformations, particularly at service loads. The fibre reinforcement also delays the formation of the diagonal crack, as shown in Figure 6, and although all the slabs failed in punching shear, the confining action of the fibres in the compression zone enabled the concrete to reach higher strains than occurred in the corresponding plain concrete slab connections, see Table 2.

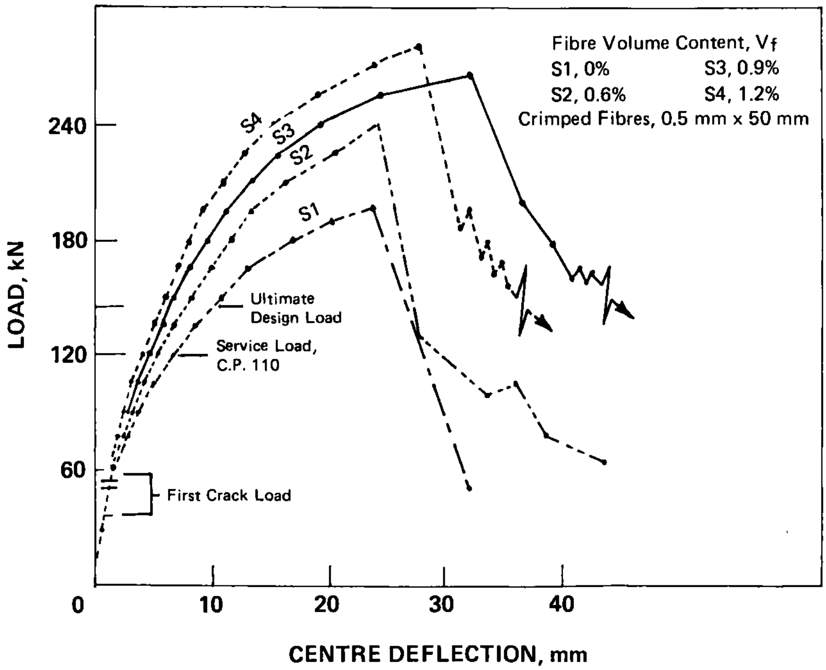


Figure 4 Load-Deflection Curves at Mid-Span

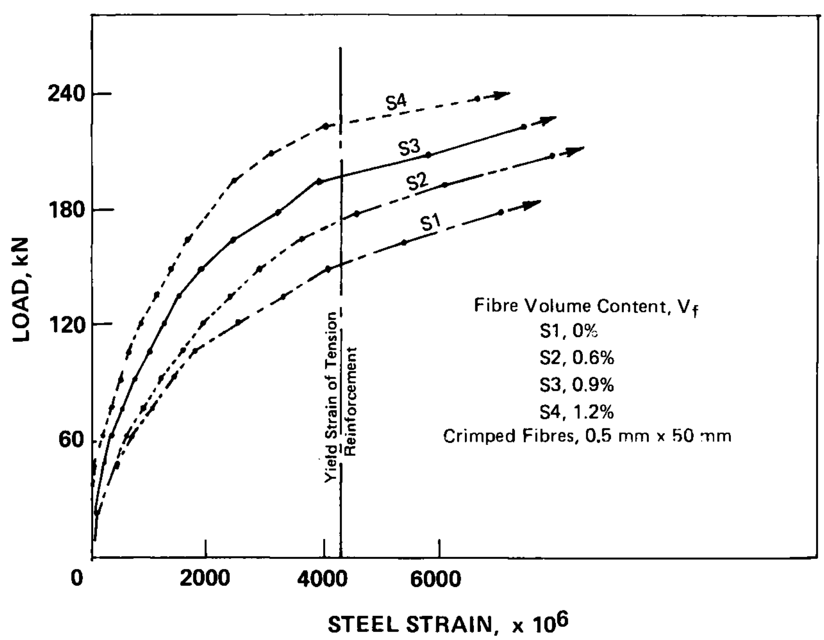


Figure 5 Load-Tension Steel Strain Near Mid-Span

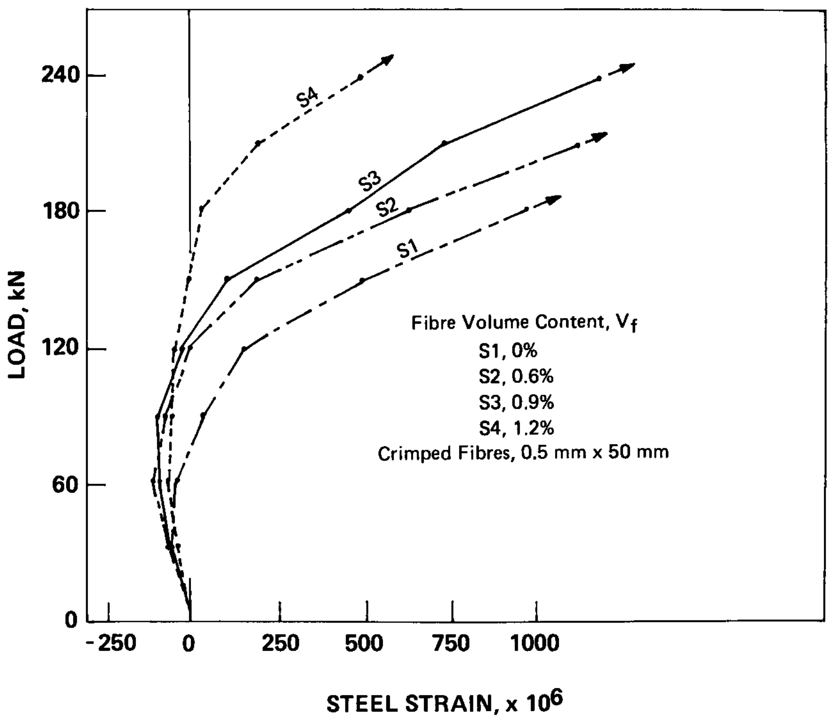


Figure 6 Load-Compression Steel Strain at Mid-Span

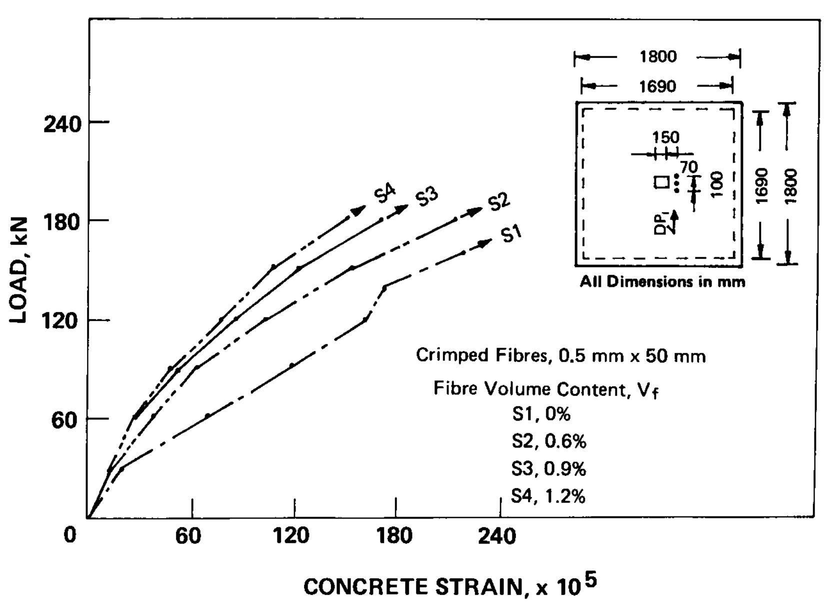


Figure 7 Load-Compression Concrete Strain Curves at DP_1

Table 2 Deformation Characteristics of Test Slabs

SLAB NUMBER	PUNCHING LOAD kN	DEFORMATIONS Maximum Tensile Steel Strain x 10^6	Maximum Concrete Strain x 10^6	Maximum Rotation radians x 10^3
(a) At First Crack				
S1	197.7	426	24.3	0.95
S2	243.6	470	22.7	1.07
S3	262.9	364	25.0	1.41
S4	281.0	290	33.5	1.45
(b) At Service Load				
S1	197.7	2369	159.3	9.30
S2	243.6	1877	98.8	7.75
S3	262.9	1265	82.4	5.38
S4	281.0	847	79.8	5.38
*(c) At Maximum Load**				
S1	197.7	13000	291.0	41.2
S2	243.6	9890	341.0	29.7
S3	262.9	10980	367.0	24.7
S4	281.0	10490	368.0	27.9

*Readings at 95 to 100 per cent of the maximum load.

Table 3 Service Loads Based on Deformation Criteria

SLAB NUMBER	STRAIN, x 10^6 or RADIANS, x 10^3	SERVICE LOAD kN	INCREASE IN SERVICE LOAD %
(a) Steel Strain			
S1	2369	118.1*	0
S2	2369	132.5	12.2
S3	2369	161.7	36.9
S4	2369	195.7	65.7
(b) Concrete Strain			
S1	159.3	118.1*	0
S2	159.3	153.7	30.1
S3	159.3	173.8	47.2
S4	159.3	182.2	55.1
(c) Rotation			
S1	9.3	118.1*	0
S2	9.3	132.9	15.1
S3	9.3	157.1	33.0
S4	9.3	161.9	37.1

*Service load 118.1 kN according to CP110.

The service loads that can be imposed on the fibre concrete slab connections taking the deformations at service load (118.1 kN) of the plain concrete slab connection as the criteria of serviceability are given in Table 3. It is clear that the presence of fibres can, for a given serviceability criterion, substantially increase the service load that a slab-column connection can carry. Considering Tables 1 and 3, the service load for a fibre concrete slab connection, with about 1 per cent fibre volume can be increased by 25 to 50 per cent beyond that of a plain concrete slab connection depending upon the type of serviceability criterion.

Post-yield Ductility

These tests confirm that the presence of fibre reinforcement considerably increases the post-yield ductility of a structural member, and hence its energy absorption characteristics. The results of the tests reported in this paper are summarized in Table 4 which shows (a) the deflection of the slabs at 25 per cent of the maximum load (after reaching the maximum load), (b) the ultimate ductility of the slabs as measured by the ratio of the centre deflection at 25 per cent of the maximum load during failure to the deflection at first crack (Figure 8) and (c) the energy absorption capacity as determined by the area under the load-deflection curve up to 25 per cent of the maximum load. These results show the unique advantage of the fibre reinforcement in improving the failure behaviour of structural members.

Table 4 Post-yield Ductility and Energy Absorption Characteristics

SLAB NUMBER	PUNCHING LOAD kN	DEFLECTION, mm		DUCTILITY Δ_2/Δ_1	INCREASE IN DUCTILITY %	ENERGY ABSORPTION CAPACITY kNm	INCREASE IN ENERGY ABSORPTION CAPACITY %
		At First Crack (Δ_1)	At 25% Maximum Load (Δ_2)				
S1	197.7	0.98	32.32	32.98	0	4.10	0
S2	243.6	1.17	62.88	53.74	63	11.00	168
S3	262.9	1.45	90.32	62.29	89	17.09	317
S4	281.0	1.17	82.97	70.91	115	16.83	311

Strength Characteristics and Tensile Membrane Action

The presence of fibre reinforcement increases not only the first crack load but also the ultimate punching shear loads of the slabs, as shown in Tables 1 and 2. The sudden drop in resistance and the loss of continuity associated with punching shear failure permitted the slabs to rebound almost towards their original position. Some slab resistance survived after the punching failure which was supported by the tensile membrane action of the flexural reinforcing bars. Further loading caused the reinforcement to be displaced and moved from their original embedded position.

Figure 9 shows a comparison of the loads carried by the slabs during loading and after failure. The tensile membrane action of the plain concrete slab connection corresponded to 24.8 per cent of the maximum load, which is in agreement with

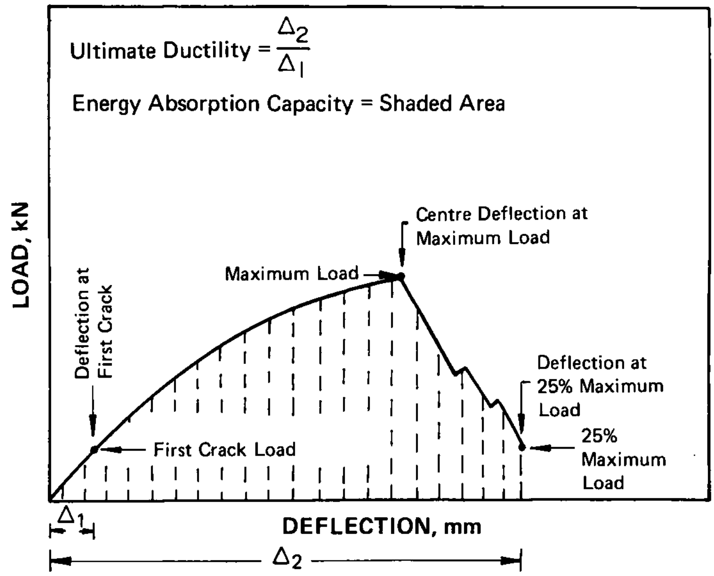


Figure 8 Determining Ductility and Energy Absorption Capacity

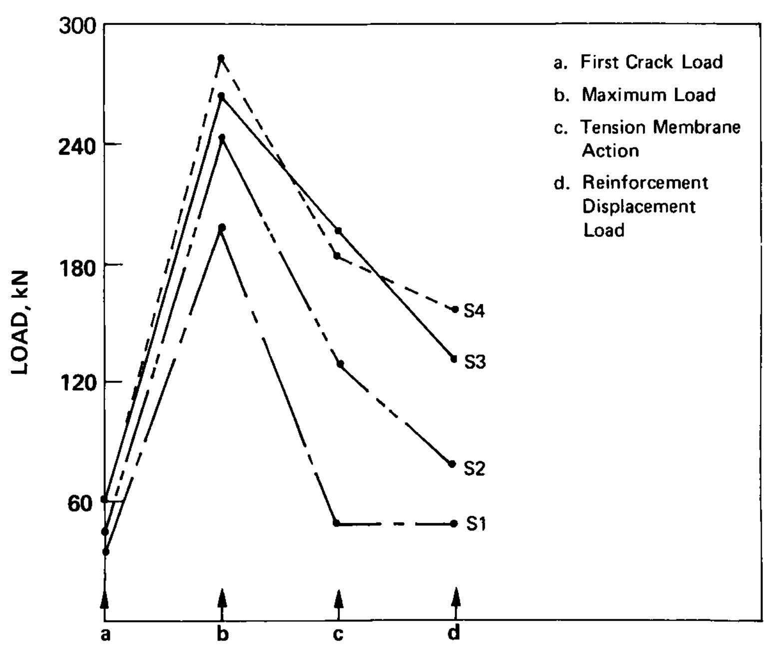


Figure 9 Test Results Comparison

data reported by other investigators. The reinforcement displacement also started at the same load. In the fibre concrete slabs both the tensile membrane action and the load at reinforcement displacement were superior to those of slab S1.

CONCLUSIONS

The presence of fibre reinforcement delays the formation of first crack, and controls the development of tensile cracks in the diagonal and lateral directions. The cracks were much finer and more in number compared to the plain slab connection.

The fibres produced a gradual punching failure, and at a fibre volume of about 1 per cent, the punching failure was not complete. The fibres transformed a square failure plane in the plain concrete slab into a circular failure plane in the fibre concrete slab connections.

The fibre reinforcement substantially reduced all the deformations of the plain concrete slab connection at all stages of loading. For a given serviceability criterion, the presence of fibres increased the service load of the corresponding plain concrete slab by 25 to 50 per cent.

At about 1 per cent fibre volume, the post-yield ductility of the slabs was increased by about 100 per cent, and the energy absorption characteristic by about 300 per cent. The confining action of the fibres in the compression zone enabled concrete strains higher than 3500×10^{-6} to be sustained at failure by the fibre concrete slab connections.

The presence of fibres increased not only the ultimate punching shear load of the corresponding plain concrete slab connection, but also the tensile membrane action and the load at which the reinforcement was displaced from their original position. At 1 per cent fibre volume, these increases were about 40, 70 and 50 per cent respectively.

The results show that 1 per cent is just about the optimum fibre volume content for structural members.

ACKNOWLEDGEMENTS The data presented here form part of a much wider study on the use of fibre reinforcement in structural members. The authors acknowledge gratefully the financial support given to the second author by the Ministry of Culture and Art, Baghdad, Iraq, and the support of crimped fibres by GKN Rolled and Bright Steel Ltd., U.K.

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RIBBED SLABS MADE OF FERROCEMENT

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ABSTRACT The reported study deals with a new type of slab for semi-prefabricated construction, indicating the main advantages of this type of slab in comparison to well known techniques of building with skins. The slab is made of a thin (15mm) flat layer of ferrocement stiffened by a set of ribs, and a series of experiments indicating the potential application of the proposed element as a building component are described.

INTRODUCTION

The aim of this study was to develop a low cost slab which is easy to handle and erect. To achieve this goal an element with minimum weight during handling and erection is required. At the final stage of the completed building the elements need not necessarily be light because of a number of factors, of which at least two, the requirement for a monolithic structure and the requirement for non-structural properties such as acoustic and thermal insulation, cannot be avoided.

With this view in mind a large number of construction methods have been developed in recent years. The commom features of these methods are that i) the skin requires additional temporary supports during erection, Figure 1(a), ii) the final product is simply a solid concrete slab, Figure 1(b), and iii) the thickness of the precast part is relatively large in comparison to the overall depth, Figure 1(b).

PROPOSAL FOR A NEW SLAB

These properties which are a feature of skin type construction prevent the realization of the great advantage of using the skin as a layer in a sandwich type element where it provides the structural properties while an appropriate fill or additional layer provides the required non-structural properties. To overcome this difficulty the use of a ferrocement ribbed skin is proposed, see Figure 2. The advantages of this approach are numerous. The weight of the prefabricated part of the slab is reduced considerably, eliminating the need for temporary supports, while the upper slab and cast in-situ joints create a monolithic structure. Due to the greater ductility of ferrocement, breakage of the units during handling is reduced. Non-structural properties can be provided in the already built structure by filling the space between the skin and the cast in-situ concrete layer. This space may

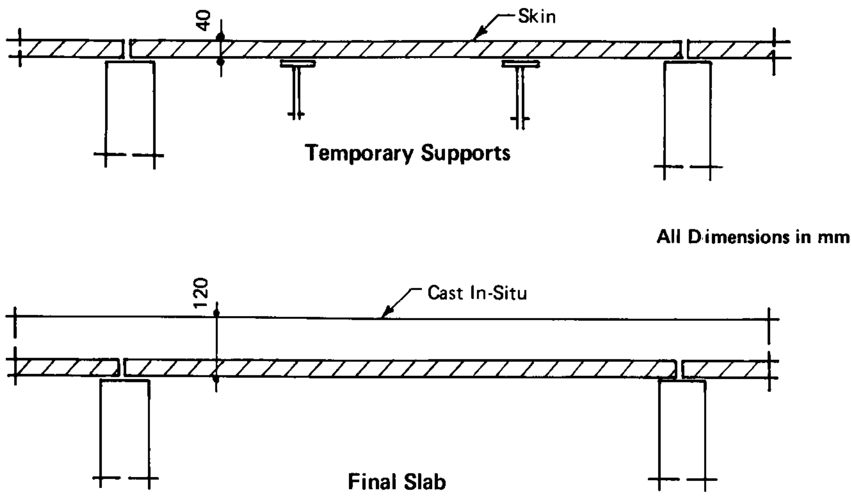


Figure 1 Skinned Slabs

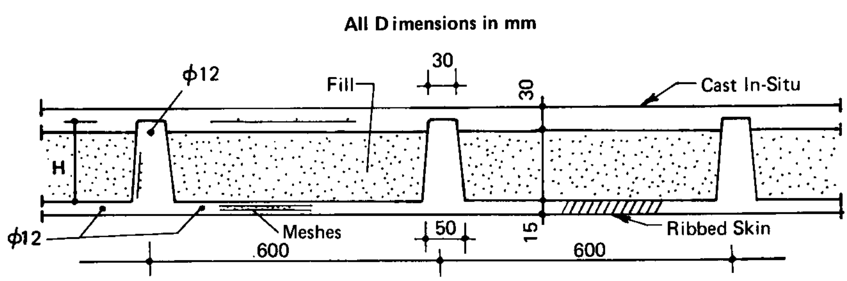


Figure 2 Proposal for a Ribbed Ferrocement Slab

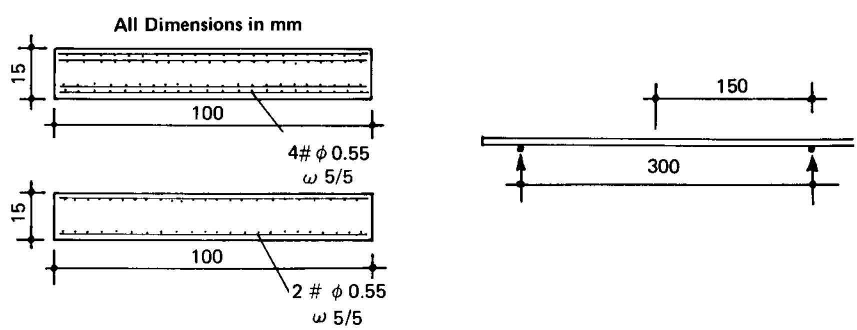


Figure 3 Ferrocement Plates

also be used for various service installations and openings may be cut in the thin ferrocement layer at any stage either during construction or for later repairs. The skin may be used also as a wall panel, and thus a higher degree of standardization can be achieved, and the lower face of the slab is smooth and ready to paint without additional plastering.

PRELIMINARY RESEARCH

As a first stage in the development of the proposed slab the following series of experiments was carried out.

1. Flexural tests on 400mm x 100mm x 15mm ferrocement plates, Figure 3, plates being reinforced by four layers of mesh or two layers of mesh, made of 0.55mm diameter wires at 5mm centres. The compressive strength of the mortar used was 50 N/mm^2 at 28 days.
2. Bending tests on a series of ribbed slabs, the details of which are given in Figure 4. The slabs were cast in two continuous stages using steel formwork, the ribs being cast first and then the mesh placed and the flat 15mm thick layer cast. The compressive strength of mortar used was, as for flexural tests, 50 N/mm^2 at 28 days.

TEST RESULTS

The results of the tests on thin plates are presented in Figure 5, which shows moment deflection curves, and in Figure 6, which shows the crack patterns developed, and are summarized in Table 1. The results of the tests on ribbed slabs are shown in Figures 7, 8 and 9, which show moment-deflection curves, moment-elongation curves at the external face and the distribution of tensile stress respectively. During the tests it was observed that the average distance between cracks was 35mm, see Figure 10, and the maximum crack width before failure 0.2mm. The crack width at 70 per cent of the ultimate moment did not exceed 0.1mm. The ratio of cracking moment to ultimate moment was about 0.50 which is high in comparison to ordinary reinforced concrete.

Table 1 Test Results for Thin Plates

PROPERTY	FOUR MESH PLATE	TWO MESH PLATE
ULTIMATE MOMENT, Nmm/mm	490	356
DEFLECTION-SPAN RATIO At Failure	1/28	1/75
At 2/3 of Ultimate Load	1/320	1/500
AVERAGE DISTANCE BETWEEN CRACKS AT FAILURE, mm	8	10
CRACK WIDTH AT FAILURE, mm	0.2	0.2
CRACKING MOMENT, Nmm/mm	300	300

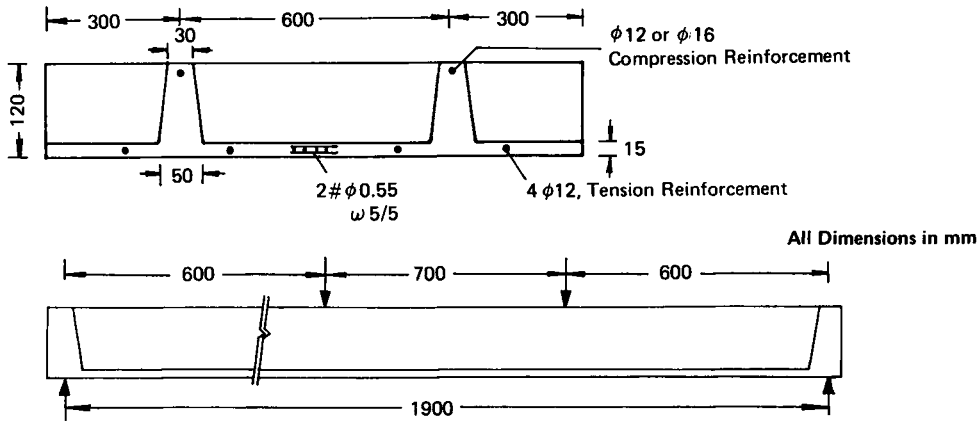


Figure 4 Ribbed Slabs for Experimental Study

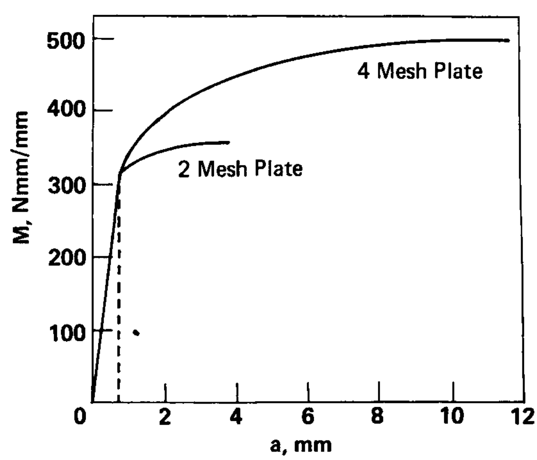


Figure 5 Ferrocement Plates - Moment Deflection Curves

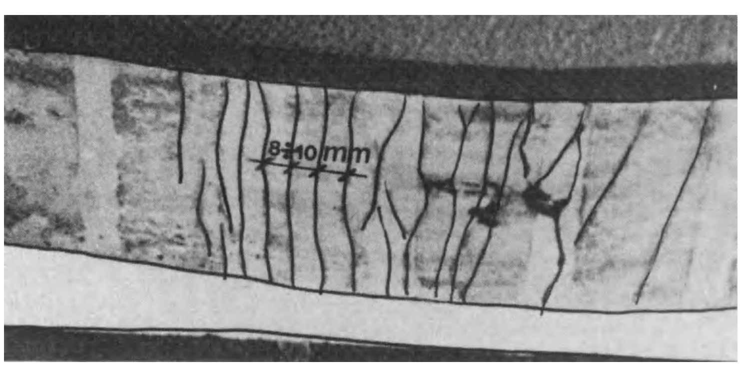


Figure 6 Ferrocement Plates - Crack Pattern

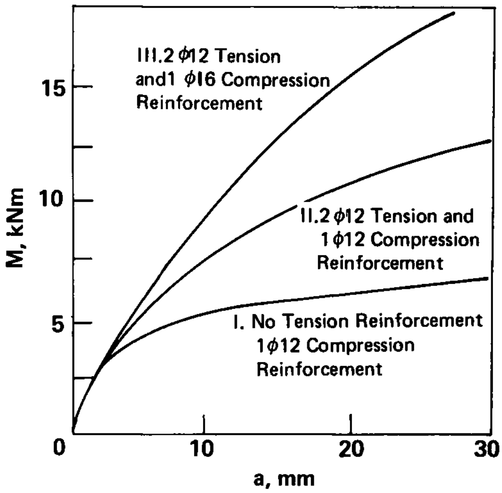


Figure 7 Ribbed Slabs
Moment Deflection Curves

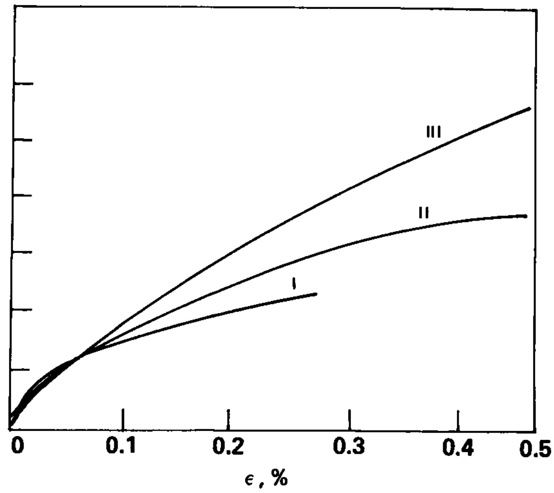


Figure 8 Ribbed Slabs
Moment Elongation Curves

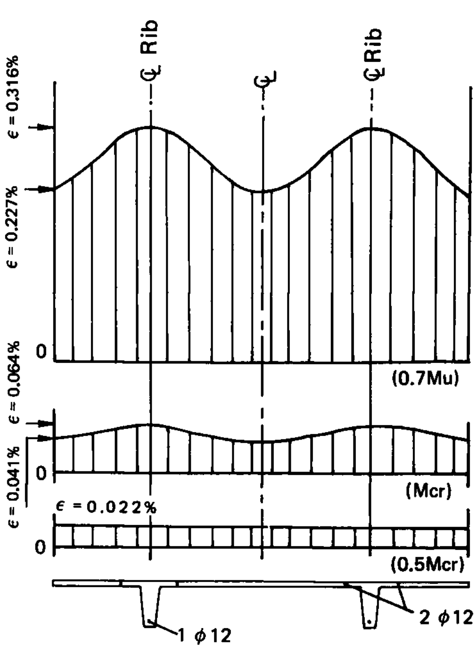


Figure 9 Ribbed Slabs
Tensile Stress Distribution

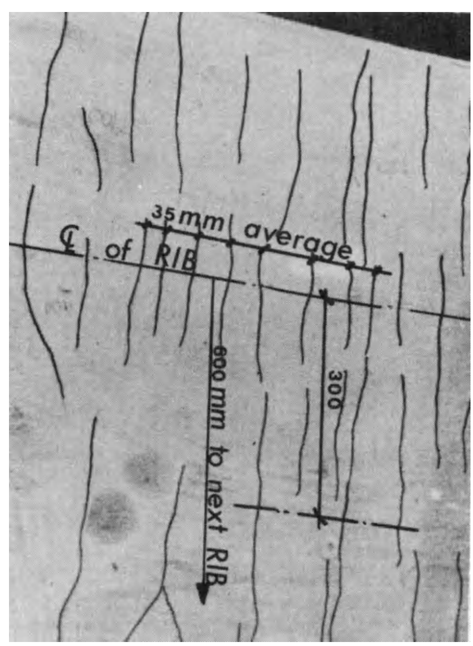
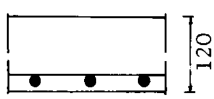
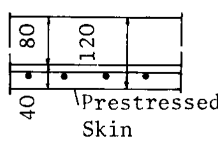
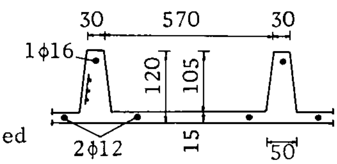


Figure 10 Ribbed Slabs
Crack Pattern

ECONOMIC ASSESSMENT

To assess the economic advantages of the proposed slab a cost comparison, based on material quantities, was made between a solid, prestressed skin and ferrocement ribbed slab. The three slabs were designed for a 3.6m span simply supported and carrying a live load of 0.15 tons/m^2. The results given in Table 2 show the comparison referred to a base unit taken as the price of concrete of grade 20 and of mild steel reinforcement with a yield point of 240 N/mm^2. Price ratios for different grades of concrete and types of reinforcement were taken as concrete grade 20; grade 40: grade 50= 1:1.15:1.20 and mild steel: meshes: prestressing wires= 1:4:4.

Table 2 Comparison of Three Types of Slabs

	SOLID, CAST IN-SITU	SKINNED	RIBBED FERROCEMENT
SECTION dimensions mm			
GRADE OF CONCRETE	20	20, Cast in-situ 40, skin	50
REINFORCEMENT	M12-200B (main) M8-250B (shrinkage)	157mm^2/m wires M8-250B (shrinkage)	2 Layers Mesh 2M12(B) 1M16(T)
EQUIVALENT GRADE 20 CONCRETE, m^3/m^2	0.12	0.12	0.026
EQUIVALENT MILD STEEL, kg/m^2	6.1	7.9	12
COST* (Unit)	465 (1.00)	510** (1.10)	362 (0.78)

*Unit price for steel taken as 4 per cent of unit price for grade 20 concrete.
**Price should be increased due to need for temporary supports.

DISCUSSION AND CONCLUSIONS

Comparison between the behaviour of the two types of ferrocement plates shows that the main advantage of the four mesh plate is due to its much larger inelastic range. In ordinary slab structures this is not an essential property and therefore the less expensive two mesh plate may be employed. The small number of mesh layers (low specific area of steel) will result in a different crack pattern than in ordinary ferrocement but again, as long as the cracks are not wider than some specified width necessary to prevent excessive deflections and corrosion, they can be tolerated. The reduced ultimate strength of the two mesh plate can be compensated for by shortening the space between the ribs or by transferring the loads directly to the ribs. The behaviour of the model ribbed slabs indicated that the required stiffness and ultimate strength can be obtained without much difficulty by varying the appropriate total depth of the section. Since the depth is controlled by the height of the ribs only, economical sections are easy to obtain.

USE OF LIGHTWEIGHT AGGREGATE CONCRETE FOR STRUCTURAL APPLICATIONS

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ABSTRACT Mix design, strength and deformation test results of lightweight aggregate concrete suitable for structural applications in reinforced and prestressed concrete are presented. It is shown that a wide range of concretes of high strengths and high early strengths can be obtained by suitable mix designs. All the mixes have excellent workability for use with fibre reinforcement. Tests with a high early strength cement show that strengths of 10 to 25 N/mm^2 can be obtained at 12 hr, while one day and 28 day strengths varied between 25 to 45 N/mm^2 and 50 to 70 N/mm^2 respectively. Fly ash can successfully replace part of the cement and sand contents to enhance durability under marine environment.

INTRODUCTION

Although lightweight aggregate concrete has been used extensively all over the world for structural applications in both reinforced and prestressed concrete construction, its use in the United Kingdom has been relatively restricted to prestige buildings or parts of a prestige structure. There is still considerable feeling amongst engineers, designers and architects that lightweight concrete is best suited as an insulation material or for structures of lesser importance. The potentialities of the material do not seem to be properly appreciated in an engineering sense.

There is considerable evidence to show that structural lightweight concrete is a technically sound material with adequate structural properties (1). The real need is for the designer and the builder to be aware of these special properties of the material and to take them into consideration in design and construction. Many of the supposedly unfavourable properties of the material can then be put to advantage in practice. The slightly higher rise in temperature due to heat of hydration with lightweight aggregates can, for example, be used to advantage in winter concreting and to produce better frost resistance in the early life of the concrete; it can also be used for early strength development.

The lower elastic modulus, on the other hand, helps to reduce both residual stresses and to make lightweight concrete less liable to cracking when normal volume changes due to shrinkage and thermal movement are restrained. Structural lightweight concrete should therefore be able to shrink more than normal weight concrete without cracking because of this reduced modulus and because of its relatively higher tensile strength, although concretes with very high shrinkage may still crack when used in slabs, for example. The self-curing effects and the phenomenon of delayed shrinkage have similar effects on plastic shrinkage and strength development, and also account for the lower creep often observed with structural lightweight concrete (2). This explains why shrinkage cracks so common with normal weight concrete, especially in flat slabs, are so often absent with lightweight concrete construction.

One significant factor not readily known to the designer is the superior aggregate-matrix bond observed with lightweight concretes, and its influence on the strength and stiffness characteristics of structural members. On the other hand, the better insulating properties of lightweight concrete could also lead to steeper temperature gradients as well as steeper moisture gradients where one of the surfaces is exposed and covered by black roofing or ashphalt; the resulting stresses will be reduced by the lower elastic modulus but their possible effects could cause structural distress and should be considered at the design stage.

There is considerable evidence to show that lightweight concrete can offer an economic alternative to other forms of construction (3, 4). Economic assessment should, however, be made on material and construction costs together with structural performance. Reasons for economy differ from one situation to another, and there are no clear cut limits beyond which lightweight concrete can be considered to be advantageous. To judge the economic advantages of the material, a project should be considered in its entirety rather than on individual components. The use of a lightweight concrete deck in the San Francisco-Oakland Bay Bridge, for example, was reported to have resulted in a saving of 5 per cent of the total cost ($3 million).

There is also considerable evidence to show that aggregate absorption capacity does not determine the permeability of a concrete. Because of the high porosity of the aggregates, lightweight concretes have a substantially larger pore volume than comparable normal concrete. Depending on the nature of the pores i.e. whether they are closed in themselves or possess a closed external shell, the water absorption of lightweight concrete may vary from being slightly more than to about twice as much as that of comparable dense concrete. The impermeability and durability characteristics of lightweight concrete can thus be very high (5, 6).

Experience with concrete ships and maritime structures shows that structural lightweight concrete can give excellent performance in service, particularly in relation to durability, and resistance to abrasion and fatigue. One of the early ships, Selma, 6340 tons deadweight and 132 m long had only 10 mm cover to the reinforcement which showed excellent condition in the expanded shale lightweight concrete after some forty years; another ship Peralta was still afloat after some fifty years!

All these tend to show that structural lightweight concrete can provide an alternative construction material both in terms of economic cost and engineering performance. At the University of Sheffield extensive study has been carried out over many years on the engineering properties and structural behaviour of reinforced and prestressed structural members made with lightweight aggregate concrete. In this paper, the authors present some pertinent data on the properties and behaviour of structural lightweight concretes with particular reference to their use in elements such as floor and roof slabs.

TEST RESULTS AND DISCUSSION

Strength Characteristics

Aggregate Particle Strength

The two notable characteristics of all lightweight aggregates are their porosity and particle strength. Due to their cellular structure, the bulk density and specific gravity of the aggregates are related to their moisture condition. At present there is no reliable test to relate the aggregate strength to that of concrete made with it. The results of the compaction-crushing type of test are greatly influenced by the gradation of the aggregate under test and the depth of compaction at which the crushing strength is defined. Tests by the authors and other investigators show that there is no correlation between aggregate strength obtained from such tests and the compressive strength of concrete (1, 7, 8, 9). The compressive crushing test, proposed by Hummel (10) which includes both particle shape and surface texture could form a useful criterion of particle strength, but further research is needed to confirm this.

Lightweight Concrete Mix Design

Most British lightweight aggregates have strengths varying from about 5 to 30 per cent of that of dense aggregates (1, 11) and the aggregate porosity may vary from about 25 to 75 per cent of the aggregate volume. Although for the lower quality aggregates there is a ceiling limit to the strength of the concrete that can be obtained from those aggregates, for the better quality aggregates, in spite of their low particle strength and high proportion of voids, the mechanical strength of the aggregates is not the sole guide to the strength of the concrete that could be obtained from the aggregates. One reason for this is the more uniform stress distribution occurring within the concrete system under external loads. Further, some aggregates such as expanded shales, clays and slates, and sintered pulverized fuel ash aggregates show 50 mm compaction strength well above the compressive strength of the paste matrix (12, 13). With expanded clay (Aglite) aggregates (particle strength ≈ 10 per cent of that of dense aggregates), strengths of 70 N/mm^2 have been obtained at 28 days.

Conventional mix design methods are not readily applicable to lightweight concrete mixes. However, mix design charts have been developed based on numerous tests (1) and some of the results based on these are shown in Tables 1 to 6. In these tests, three types of cement have been used: ordinary Portland cement, ultrafine Portland cement (specific surface 750 to 800 m^2/kg) and a trial high early strength (HES) cement (specific surface 450 m^2/kg). The major difference between the ultrafine cement and the HES cement is that while the former accelerates the hydration process by way of particle fineness, the latter does this through its chemical composition.

Mixes for Prestressed Concrete

Tables 1 to 3 give the details of mixes of concrete made with ordinary Portland cement which are suitable for prestressing. Table 1 gives the mix proportions and compressive strength at various ages for all-Lytag (sintered pulverized fuel ash aggregate) concrete mixes (i.e. Lytag coarse + Lytag fines) for cement contents of 450 to 600 kg/m^3. Table 2 gives similar details for Lytag-sand concrete mixes with cement contents of 450 and 500 kg/m^3. Table 3 gives details of the 450 kg/m^3 Lytag-sand concrete mix with steel fibres. The mix given in Table 3 has been used for an extensive study on prestressed concrete members. The minimum strength of

35 N/mm^2 at transfer (5 days) was easily achieved, and the 28 day strength varied between 55 and 65 N/mm^2. The fibre concrete mixes showed substantial increases in flexural strength with a distinct first cracking load.

Table 1 Lytag Concrete Mixes

MIX CONTENT, kg/m^3				COMPRESSIVE STRENGTH, N/mm^2			
Cement	Lytag fine	Lytag 12mm	Effective Water	1 Day	3 Days	7 Days	28 Days
450	398	608	300	11.6	32.0	44.2	56.8
500	359	608	300	12.2	36.0	50.4	61.2
550	320	608	300	13.8	36.3	51.0	78.5
600	281	608	300	16.3	38.2	54.3	83.7

Table 2 Lytag-Natural Sand Concrete Mixes

MIX CONTENT, kg/m^3				STRENGTH, N/mm^2			
Cement	Sand Zone 3	Lytag 12mm	Effective Water	3 Days	5 Days	7 Days	28 Days
500*	462	696	188.3	36.0^a	44.8	50.4	77.8
450**	516	696	184.0	32.0^a	39.3	44.2	63.2
450	516	696	184.0	3.9^b	3.6	3.3	3.5
450	516	696	184.0	3.0^c	3.1	3.2	3.6

*75mm slump. **140mm slump.
a, compressive strength; b, modulus of rupture; c, tensile splitting strength.

Table 3 Lytag-Natural Sand-Steel Fibre Concrete Mixes*

MIX	COMPRESSIVE STRENGTH, N/mm^2		DENSITY, kg/m^3		MODULUS OF RUPTURE AT 28 DAYS	
	5 Days	28 Days	5 Days	28 Days	First Crack	Failure
Normal Concrete						
A	39.6	54.4	1968	1868	-	4.25
B	42.6	59.8	1897	1868	-	2.85
Fibre Concrete						
A	45.5	55.7	1988	1928	5.63	7.44
B	50.1	66.0	2006	1952	4.20	5.71

*mix quantities (kg/m^3): cement, 445.5; sand, 511; Lytag (12mm) 689; fibre, 78, water 173.25.
Mix A: crimped fibre, 0.50mm dia x 50mm, 1% volume.
Mix B: Japanese fibre, 0.339mm dia x 25mm, 1% volume.

Lytag-Fly Ash Mixes

Pozzolanic additions to cement are known to give concretes having better long term durability in sea water, and Table 4 gives details of mixes used for a study in reinforced concrete members with a required strength of 45 N/mm^2 at 28 days. Mix C, with cement (OPC) and PFA contents of 287 and 123 kg/m^3 respectively, gave adequate flow and strength characteristics without and with fibre additions. In these mixes a plasticizing agent was added to reduce the water content and friction between fibres and aggregates. Mix C1 gave compressive and flexural strengths of 7.4 N/mm^2 and 1.08 N/mm^2 respectively at one day and 37.0 N/mm^2 and 2.90 N/mm^2 at 7 days. In another study involving Lytag (coarse and fine)-fly ash mixes with the same design strength of 45 N/mm^2 at 28 days, one and three day compressive strengths averaged 9.2 N/mm^2 and 25.7 N/mm^2 respectively. At six months, the strength was 55 N/mm^2.

Table 4 Lytag-Natural Sand-Fly Ash Concrete Mixes

MIX*	EFFECTIVE W/C RATIO	SLUMP mm	COMPRESSIVE STRENGTH, N/mm^2				DENSITY kg/m^3
			3 Days	7 Days	14 Days	28 Days	
A	0.35	30	29.0	41.6	49.0	52.6	1875
B	0.37	90	26.2	37.6	44.5	48.6	1865
C	0.40	160	24.2	37.0	43.2	45.3	1853
D	0.45	230	18.3	27.1	37.0	39.7	1846
E	0.50	collapse	13.7	23.9	28.7	33.0	1837

*Mix contents (kg/m^3): cement, 287; Fly Ash, 123; Sand, 560; Lytag, 696.

MIX	ADMIXTURE** cc/kg	FIBRE VOLUME %	SLUMP mm	28 DAY STRENGTH, N/mm^2			DENSITY kg/m^3
				Compressive	Flexural	Splitting	
C1	2.5	0	160	45.3	3.24	3.02	1853
C2	4.0	0.5	90	42.5	6.04	4.06	1892
C3	5.5	1.0	40	47.8	6.80	4.86	1920

**per kg of (cement + fly ash)

High Early Strength Concrete

In many practical situations, and in precasting, high early strength is an important consideration for rapid construction and economic use of prestressing beds (14, 15). In the tests reported here a high early strength (HES) cement was used. The HES cement is chemically a Portland cement and complies with BS12, although the sulphuric anhydride (SO_3) content is nearly on the maximum limit. This cement has a fineness (450m^2/kg) slightly higher than that of ordinary and rapid hardening cements; its setting time is normal, and in fact longer than many rapid hardening cements.

Tables 5 and 6 give the strength development characteristics obtained in two separate studies. With the mix details shown in these tables, the one day strength varied between 25 and 45 N/mm^2 while the 28 day strength ranged from 50 to 70 N/mm^2. At 12 hours, these mixes developed strengths of 10 to 25 N/mm^2. In general, the strengths at 12 hr, 1 day and 3 days ranged respectively from 20 to 30, 50 to 60 and 70 to 80 per cent of their corresponding 28 day strengths.

Table 5 High Early Strength Concrete

MIX	AGE AT TEST, DAYS			
	1/2	1	3	28
(a) Compressive Strength, N/mm^2				
A*	15.1	25.7	34.0	48.0
A1**	9.9	30.0	43.5	60.0
B*	13.1	38.6	49.3	65.1
B1**	14.3	39.0	52.2	66.2
(b) Flexural Strength, N/mm^2				
A	2.0	2.0	3.4	4.0
A1	2.4	2.2	3.8	4.3
B	3.1	3.0	4.4	4.8
B1	2.5	2.9	4.2	4.5
(c) Elastic Modulus, kN/mm^2				
A	-	14.3	15.9	17.9
A1	-	16.4	18.4	18.8
B	-	17.7	20.1	20.3
B1	-	17.5	19.4	21.2

*wet cured for 3 days, then dried in internal environment
**continuous drying in internal environment
Mix Contents (kg/m^3):
Mix A: cement, 370; sand, 590; Lytag, 696; water, 180; with slump, 180 + mm.
Mix B: cement, 520; sand, 440; Lytag, 696; water, 190; with slump, 110mm.

Table 6 Strength Results with HES Cement

SPECIMEN	CEMENT TYPE	MIX PROPORTIONS (C:S:A:W)*	COMPRESSIVE STRENGTH, N/mm^2 Age At Test, days						
			1/2	1	3	7	28	50	90
(a) Lightweight Concrete									
S1	HES	1:1.48:2.13:0.58	10	24	42	49	53	-	55
S2	OPC	1:1.59:2.02:0.58	-	8	22	32	41	-	48
S3	OPC	1:0.57:1.27:0.37	-	17	37	50	59	-	65
S4	HES	1:0.58:1.23:0.37	26	44	53	60	66	-	68
(b) Gravel Concrete									
N1	OPC	1:1.13:2.78:0.58	-	12	31	38	50	53	-
N2	OPC	1:0.83:1.77:0.37	-	24	47	57	67	71	-

*Cement:Sand:Aggregate:Water

The same high rate of development of flexural strength and elastic modulus was obtained from 12 hr onwards. Flexural strengths of 3.5 to 4.5 N/mm^2 were obtained at 24 hr while 80 to 90 and 90 to 100 per cent of the 28 day elastic modulus were obtained at one and 3 days respectively.

In Table 6, the strength development of mixes with HES cement is compared with corresponding mixes with OPC using lightweight and gravel aggregates. With lightweight aggregates, HES cement mixes give higher strengths than the corresponding OPC mixes at all ages from 12 hr to 90 days. Comparing OPC mixes of lightweight and gravel aggregates with the same water-cement ratios, lightweight concrete consistently gives lower strength, the reduction on average amounting to about 30 per cent at 1 day and reducing to about 15 per cent at 28 days. The aggregate particle strength is thus a contributing factor to concrete strength (9), although these results do indicate that with good quality aggregates, lightweight aggregates can produce concretes of high strength similar to those with normal weight aggregates.

Comparing lightweight aggregate concrete with HES cement and gravel aggregate concrete with OPC (both of identical water-cement ratios and similar mix proportions), the HES cement concrete shows higher strength development up to 3 days; at high water-cement ratios, the lightweight aggregate concrete with HES cement gives consistently higher strengths compared to gravel concrete with OPC. At low water-cement ratios, both concretes produce strengths of the same order at 28 days.

The curing regime appears to have a significant effect on lightweight aggregate concretes with low HES cement content. The wet curing has a beneficial effect up to about 24 hr, but thereafter dry curing is more beneficial, and at 28 days concrete continuously dry cured gave about 25 per cent higher strength than concrete wet cured initially for 3 days. This has some practical significance in that wet curing is often impracticable at site, and dry curing is thus beneficial to lightweight aggregate concretes.

Drying has a greater significant effect on flexural strength of lightweight aggregate concretes than that on normal aggregate concretes (15). Wet or dry curing has no great effect on the early strength development up to about 7 days; however, continued drying reduces flexural strength due to moisture gradients, but the original strength is restored with further drying at ages of 3 to 6 months. This again has some practical significance in that flexural cracks appearing in partially prestressed members at early ages are likely to close with further ageing of the member.

Table 7 Shrinkage Behaviour of HES Cement Concrete

SPECIMEN	CEMENT TYPE	WATER-CEMENT RATIO	SHRINKAGE, x 10^6 Laboratory	SHRINKAGE, x 10^6 Outside
(a) Lightweight Aggregate Concretes, shrinkage measured at 100 days				
S1	HES	0.58	585	500
S2	OPC	0.58	500	400
S3	OPC	0.37	446	374
S4	HES	0.37	480	425
(b) Gravel Aggregate Concretes, shrinkage measured at 50 days				
N1	OPC	0.58	335	260
N2	OPC	0.37	273	215

Shrinkage Behaviour

Table 7 gives some data on the shrinkage of lightweight and gravel aggregate concretes of similar composition. Comparing lightweight aggregate concretes with HES cement and OPC, concretes with HES cement give about 15 per cent more shrinkage than concrete with OPC. Comparing lightweight and gravel aggregates with OPC, the latter show about 30 per cent less shrinkage compared to the former. These results are, however, short-term and should be related to long-term values (14).

CONCLUSIONS

This paper shows the great potential of lightweight aggregate concrete for a wide range of structural applications in reinforced and prestressed concrete. The mixes, the data of which are presented here, have been used for short and long-term structural testing. Limitations of space preclude any reference to structural behaviour of these concretes and these will be reported elsewhere.

It is shown that with a cement content of 450 kg/m^3, strengths of 40 to 50 N/mm^2 could be obtained at 5 days and strengths of 55 to 65 N/mm^2 at 28 days with and without steel fibre inclusions. Fly ash replacement of cement can be successfully carried out with lightweight aggregates, and such mixes can be designed for any strength range. For the mixes used in this study, strengths of 35 to 55 N/mm^2 were obtained at 28 days. All these mixes had a high degree of workability.

Concretes with the new HES cement not only give excellent workability but a high rate of strength development. Compressive strengths of 10 to 25 N/mm^2 and 25 to 45 N/mm^2 were obtained at 12 hr and 1 day respectively. Similar increases were also obtained with flexural strength and elastic modulus. The shrinkage of lightweight aggregate concrete with HES cement was only about 15 per cent higher than that of similar concrete with OPC.

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EFFECT OF REPEATED DOSAGES OF SUPER-PLASTICIZERS ON WORKABILITY, STRENGTH AND DURABILITY OF CONCRETE

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ABSTRACT This paper gives results of a laboratory investigation to determine the effect of repeated dosages of superplasticizers on workability, strength and durability of concrete. The test results indicate that large increases in slumps of superplasticized concretes can be maintained for several hours by the addition of a second dosage. Apart from one instance, the addition of a third dosage was found to be undesirable. The repeated additions of sulphonated melamine and naphthalene based superplasticizers caused substantial loss in entrained air content of the concrete, while an opposite trend was observed for concrete incorporating the lignosulphonate based superplasticizer.

INTRODUCTION

Superplasticizers, which have been introduced in North America in recent years, are slowly finding acceptance in the concrete industry. A number of research establishments both in Canada and the U.S.A. are performing laboratory studies in the use of these admixtures and are attempting to delineate their uses and limitations (1-5). One problem with the use of these admixtures is that superplasticized concrete tends to lose slump rather rapidly and in actual field concreting operations, where placing of concrete is often delayed due to a variety of reasons, superplasticized concretes would lose their advantage and would have to be re-tempered. There are only limited data available on the effect of re-tempering on the properties of fresh and hardened concrete. Walz and Bonzal (6) advise against the use of re-tempering with superplasticizers but offer little rationale. This investigation reports the results of a laboratory study performed at CANMET to obtain information on the effect of repeated dosages of each of a number of superplasticizers on the properties of high strength concrete.

CONCRETE MIXES

A total of four concrete mixes each of 0.062 m^3 were made in the CANMET laboratory in 1977 using a counter current mixer. The materials, mix proportions and the procedure for the incorporation of the superplasticizers in the fresh concrete are described below.

Materials

Normal Portland cement CSA Type 10 (ASTM Type 1), crushed limestone of 19 mm maximum size as coarse aggregate and a local sand as fine aggregate were used. To keep the grading uniform for each mix, the sand was separated into different size fractions and then re-combined to a specified grading. The air-entraining admixture employed with all the mixes was of a sulphonated hydrocarbon type. The three types of superplasticizers used were as follows:

Sulphonated naphthalene formaldehyde condensates. Superplasticizers A and C fall in this category. Superplasticizer A is of U.S. origin and it is usually available as a soluble powder or as a 34 per cent aqueous solution with a density of 1200 kg/m^3 and is of dark brown colour. It has a negligible chloride content. Superplasticizer C is of Japanese origin and is usually available as a 42 per cent aqueous solution with a density of 1200 kg/m^3 and is also of dark brown colour with negligible chloride content.

Sulphonated melamine formaldehyde condensates. Superplasticizer B belongs to this category. It is of German origin and is usually available as a 20 per cent aqueous solution with a density of 1,000 kg/m^3. It is limpid (clear) to slightly turbid (milky) in appearance, with chloride content of 0.005 per cent.

Modified lignosulphonates. Superplasticizer D falls in this category. This is of French origin but is now being manufactured in Montreal. It is usually available as a 20 per cent aqueous solution, with a density of 1100 kg/m^3. It is light brown in appearance and contains no chlorides.

Most of the above superplasticizers are mainly made from organic sulphonates of the type RSO_3 where R is a complex organic group, frequently of high molecular weight.

Mix Proportions

The graded coarse and fine aggregates were batched (by mass) under dry room conditions. The coarse aggregate was then immersed in water for 24 hr, the excess water was decanted and the water retained by the aggregate was determined by mass difference. A predetermined amount of water was added to the fine aggregate, which was then allowed to stand for 24 hours.

A standard mix with a water-cement ratio of 0.42, an aggregate-cement ratio of 4.77 and a cement content of 379 kg/m^3 was used. The dosage of the air-entraining agent was kept constant but the dosage of each superplasticizer varied, being in accordance with the manufacturer's recommendations.

Properties of Fresh Concrete

The properties of the fresh concrete: temperature, slump, density and air content, were determined after the initial mixing time of 6 minutes and after the addition of the first dosage of a superplasticizer and further mixing of the concrete for two minutes. Following this, the concrete was left in the mixer undisturbed except for measurements of slump and air content which were continued at intervals. When the slump had reverted to the initial value of about 50 mm or after about one hour had elapsed, a second dosage of the superplasticizer was added and after the concrete was mixed for a further two minutes measurements were taken and again repeated frequently to determine the rate of loss of slump and air content. The procedure was repeated with a third dosage of the superplasticizer until the fresh concrete slump had, once again, reverted to the initial value.

The above procedure for mixing and testing of the fresh concrete was used with each of the remaining three superplasticizers, except in the case of superplasticizer D, in which case tests were terminated two hours after the addition of the third dosage. The test results are summarized in Table 1.

Table 1 Properties of the Fresh Concrete

SUPERPLASTICIZER		TEMPERATURE °C	SLUMP mm	DENSITY kg/m³	AIR CONTENT %
Type	Dosage % by mass of cement				
(a) After Initial Mixing of 6 minutes					
A	1.5	24	50	2377	4.9
B	2.0	23	50	2365	5.0
C	1.0	23	40	2377	4.5
D	2.0	23	50	2358	5.2
(b) After 1st Dosage of Superplasticizer					
A	1.5	24	>250	2409	3.8
B	2.0	23	230	2358	4.9
C	1.0	24	240	2384	4.2
D	2.0	24	220	2313	7.2
(c) After 2nd Dosage of Superplasticizer					
A	1.5	23	>250	2435	1.7
B	2.0	24	250	2409	3.2
C	1.0	23	240	2403	3.4
D	2.0	24	>250	2307	5.0
(d) After 3rd Dosage of Superplasticizer					
A	1.5	22	230	2448	1.5
B	2.0	24	230	2409	2.5
C	1.0	22	200	2416	3.2
D	2.0	22	250	2384	4.0

1st dose was added after about 30 minutes of mixing, during which time concrete had been left covered and undisturbed in the mixer.
2nd dose was added after slump had reverted to initial value.
3rd dose was added after slump had reverted to initial value.

PREPARATION AND TESTING OF SPECIMENS

Twelve 102 mm dia x 203 mm cylinders and six 89 mm x 102 mm x 406 mm prisms were cast from each of the four mixes, three cylinders after the completion of initial mixing and three after the addition of each of the three dosages of a superplasticizer. The six prisms were cast from each mix after the third dosage.

All specimens were compacted using a vibrating table. After casting, the specimens were covered with a water saturated burlap and were left in the casting room at 24 ± 1.3°C and 50 per cent relative humidity for 24 hours. They were then demoulded and transferred to the moist curing room until testing.

At 14 days, two prisms were removed from the moist-curing room and tested in flexure in accordance with ASTM Standard C78-75, using a third-point loading. The cylinders were capped with a sulphur and flint mixture and tested for compressive strength at 28 days, using a 2.7 MN machine.

Although durability cannot be measured directly, prolonged exposure of concrete to repeated cycles of freezing and thawing produces measurable changes in test specimens that may indicate deterioration. Measurements made on the test specimens after freeze-thaw cycling provide data that can be used to evaluate the relative frost resistance or durability.

In this investigation, test prisms were exposed to repeated cycles of freezing in air and thawing in water according to ASTM Standard C666-75, using an automatic freeze-thaw apparatus designed to perform eight cycles per day, with one complete cycle from 4.4 ± 1.7°C to -17.8 ± 1.7°C and back to 4.4 ± 1.7°C taking 3 hours. During this investigation, however, the freeze-thaw unit did not fully meet the above temperature requirements during freezing periods, with temperature fluctuating between -15 and -11.7°C.

At the end of the initial moist-curing period of 14 days, each set of four prisms was placed in the freeze-thaw cabinet at the thawing phase (4.4 ± 1.7°C) for one hour. The initial and all subsequent measurements of the freeze-thaw and reference test specimens were made at this temperature. After initial measurements of the test prisms were taken, two prisms were placed in the freeze-thaw cabinet and the two companion prisms were placed in the moist-curing room for reference purposes.

The freeze-thaw test specimens were visually examined at frequent intervals; in addition their length, density, resonant frequency and ultrasonic pulse velocity were determined at approximately every 100 cycle interval. The freeze-thaw tests were terminated when the test prisms had shown expansion in excess of 0.07 per cent or when about 800 freeze-thaw cycles had been completed.

RESULTS AND DISCUSSION

A total of 48 cylinders and 24 prisms were tested. The compressive strength results are summarized in Table 2 and also illustrated in Figure 1, together with the relevant data for slump and air content. The flexural strength and durability results are given in Table 3.

Effect of Repeated Dosages of Superplasticizers on Slump, Air Content and Compressive Strength of Concrete

Concrete Incorporating Superplasticizer A

The concrete reached a slump of about 250 mm immediately after the addition of the first dosage of the superplasticizer. Twenty five minutes later (1 hr after the commencement of initial mixing) the superplasticized concrete had lost considerable workability, with slump at 100 mm. Immediately following the addition of the second dosage, the slump once again reached 250 mm. This value was maintained for about 50 minutes when the concrete started to lose slump rapidly, reaching a value of 30 mm 3 1/2 hr later, see Figure 1. The addition of the third dosage increased the slump to 225 mm but the concrete lost slump rapidly thereafter, reaching a

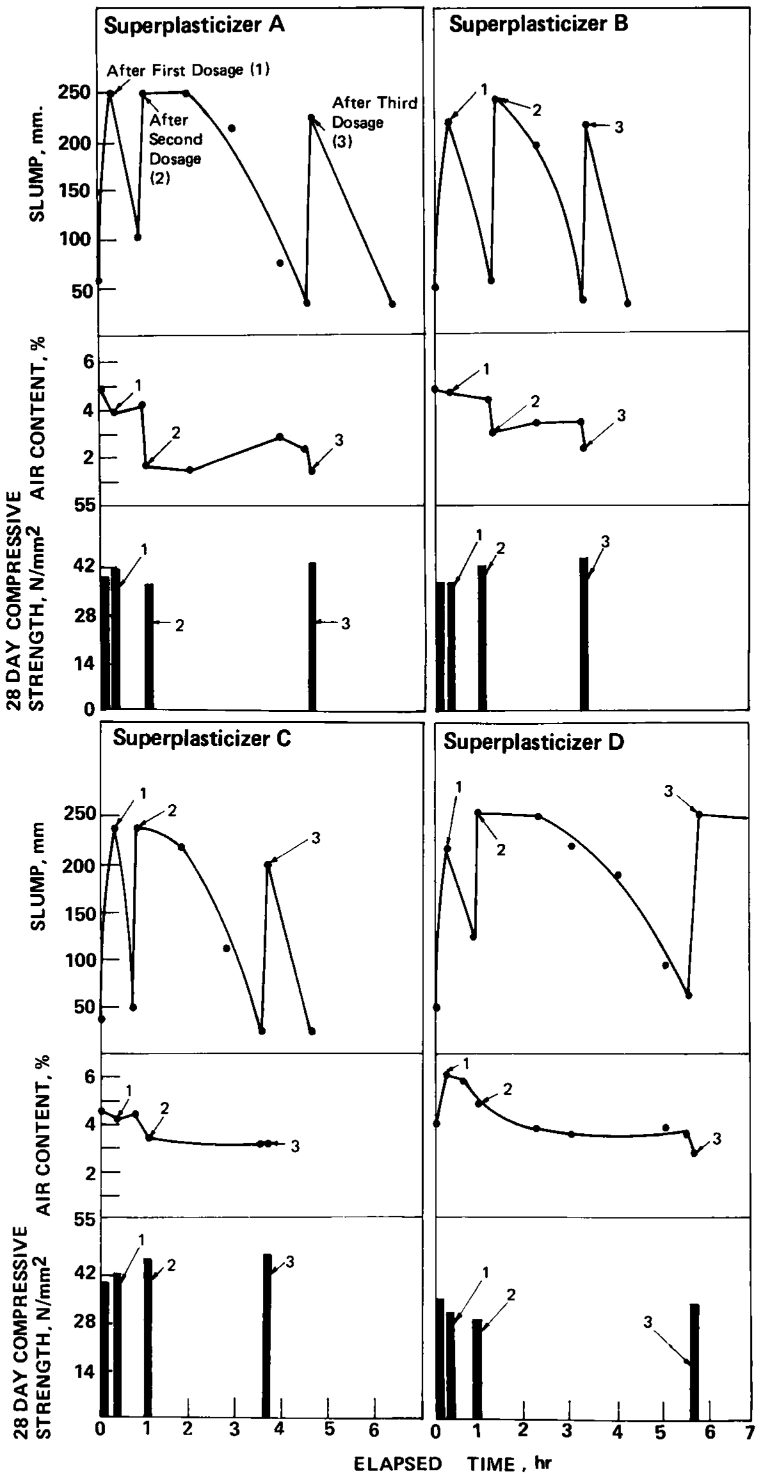

Figure 1 Elapsed Time from Initial Mixing vs Slump, Air Content and Strength

value of 30 mm in just 1 hour 35 minutes.

Table 2 Summary of Compressive Strength Results

SUPERPLASTICIZER		CYLINDER COMPRESSIVE STRENGTH AT 28 DAYS, N/mm^2			
Type	Dosage % by mass of cement	Cast Immediately after Mixing	Cast Immediately after the Addition of		
			1st Dosage	2nd Dosage	3rd Dosage
A	1.5	38.5	40.7	36.1	42.6
B	2.0	38.3	39.1	43.6	45.4
C	1.0	39.6	42.3	46.7	47.9
D	2.0	34.7	31.5	28.9	33.8

N.B. Each value is a mean of 3 test results.

Table 3 Summary of Flexural Strength Results at 14 Days and at End of Freeze-Thaw Cycling

SUPERPLASTICIZER		FREEZE-THAW TESTS		FLEXURAL STRENGTH*, N/mm^2			REMARKS
				Control Prisms			
Type	Dosage % by mass of cement	Number of Cycles	Age at End days	Start of Test	End of Test	Durability Prisms	
A	1.5	109	29	6.96	7.38	2.93†	Prisms exhibited considerable damage near the ends.
B	2.0	500	52	7.83	8.38	5.27†	Prisms exhibited considerable damage near the ends.
C	1.0	643	98	8.20	8.79	4.21†	Prisms exhibited considerable damage near the ends.
D	2.0	830	170	6.86	7.79	7.21	Prisms broke within the middle third.

*Mean of two results.
†Strength values calculated using $R = PL/bd^2$ even though the fracture occurred in the tension surface wall outside the middle third of the span length.

The repeated addition of the superplasticizer resulted in the loss of entrained air. From an initial value of 4.9 per cent, the air content of the superplasticized concrete was reduced to 3.9, 1.7 and 1.5 per cent after the addition of each successive dosage.

The repeated additions of the superplasticizer resulted in an increase in the compressive strength of the concrete. Test cylinders cast immediately after the initial mixing of the concrete had an average strength value of 38.6 N/mm^2; this value had increased to 42.6 N/mm^2 for the test cylinders cast immediately after addition of the third dosage. This increase in strength is primarily due to the loss of entrained air from the concrete. The loss in strength of test cylinders cast after the addition of the second dosage of the superplasticizer is thought to be anamalous as it cannot be explained satisfactorily.

Concrete Incorporating Superplasticizer B

The 50 mm slump concrete became flowing concrete immediately after addition of the first dosage of the superplasticizer, with the slump reaching a value of 225 mm. Thirty five minutes later (1 hr after the commencement of initial mixing) the concrete had lost most of the gain in slump, reverting to its original slump of 50 mm. The addition of the second dosage increased the slump to 250 mm, but the concrete started losing slump immediately thereafter and 2 hours 15 minutes later had reached a value of 40 mm. The addition of the third dosage, once again, increased the slump to 225 mm but slump loss was very rapid and only 65 minutes later a value of 30 mm was reached. In spite of the high slump after the addition of the third dosage, the concrete exhibited poor workability.

The repeated additions of the superplasticizer resulted in a steady loss of entrained air. From an initial value of 5 per cent, the entrained air content of the superplasticized concrete was found to be 4.9, 3.2 and 2.5 per cent after each successive dosage.

The loss of entrained air resulted in substantial increase in compressive strength of the test cylinders. The 28 day strength of test cylinders cast after the addition of the third dosage of the superplasticizer was 45.4 N/mm^2 compared with a strength value of 38.3 N/mm^2 for test cylinders cast immediately after the initial mixing, see Figure 1.

Concrete Incorporating Superplasticizer C

The addition of the first dosage of superplasticizer increased the slump of the concrete from 40 mm to 240 mm. However, slump loss was rapid; only 20 minutes later the slump had reverted back to about its original value. The addition of the second dosage again increased the slump to 240 mm. This value was maintained for about 20 minutes following which the concrete lost slump steadily but at a slower rate reaching 25 mm in 2 hours 45 minutes. The addition of the third dosage increased the slump of the concrete to only 200 mm, with rapid slump loss thereafter a value of 25 mm was reached in just over 1 hr, see Figure 1. Unlike concretes incorporating superplasticizers A and B, this concrete exhibited good workability between the second and third additions of the superplasticizer.

The repeated additions of the superplasticizer resulted in only 1.3 per cent loss of entrained air at the end of the third dosage, the initial and final values of air content being 4.5 and 3.2 per cent respectively. This is in sharp contrast with concretes incorporating superplasticizers A and B, which had lost substantial amounts of entrained air at the end of the third dosage.

The 28 day compressive strength of test cylinders cast immediately after the addition of the third dosage of the superplasticizer was 47.9 N/mm^2 compared with a strength value of 39.6 N/mm^2 for the cylinders cast immediately after the initial mixing of the concrete, Figure 1. This is an increase of 8.2 N/mm^2 and cannot be explained by the loss of 1.3 per cent of the entrained air. The exact mechanism of this gain in strength is not known, but may be due to some special properties imparted to the concrete by the superplasticizer.

Concrete Incorporating Superplasticizer D

The properties of the fresh concrete incorporating the superplasticizer were considerably different from those of concretes incorporating superplasticizers A, B, and C, see Figure 1. The addition of the first dosage increased the slump of concrete from 50 mm to 215 mm, and forty five minutes later, the concrete still had a slump of 125 mm. The addition of the second dosage increased the slump to 250 mm. This high slump was sustained for about 1 hr, following which there was a gradual loss, the slump reaching a value of 60 mm in 4 hours 30 minutes after the addition of the second dosage. The addition of the third dosage once again increased the slump to 250 mm and 2 1/2 hr later the concrete still had a slump of 215 mm.

The entrained air content of the concrete increased to 7.2 per cent from 5.2 per cent after the addition of the first dosage. This is in contrast to the concretes incorporating superplasticizers A, B and C, which had exhibited loss of entrained air. An air content of 6 per cent after the second dosage of the superplasticizer had been added, was still higher than the initial value of 5.2 per cent. After the addition of the third dosage of the superplasticizer, the concrete had an air content of 4 per cent, a loss of only 1.2 per cent from the initial value.

The compressive strength of test cylinders cast after the addition of the superplasticizer showed a reduction compared with the strength of the test cylinders cast immediately after initial mixing, see Figure 1. The strength loss ranging from 9.5 per cent for cylinders cast after the addition of the first dosage of the superplasticizer to 16.8 per cent for cylinders cast after the addition of the second dosage. The strength of concrete containing no superplasticizer was 34.7 N/mm^2. The above loss in strength of concrete incorporating superplasticizer D is in sharp contrast to the gain in strength for concretes incorporating superplasticizers A, B and C, and is thought to be due to the fact that the former concrete entrained higher amounts of air than the latter concretes.

Durability of Concrete Prisms Exposed to Repeated Cycles of Freezing and Thawing

The durability of concrete exposed to repeated cycles of freezing and thawing was determined by measuring change in density, length, resonant frequency and pulse velocity of test prisms after exposure to freeze-thaw cycling, with respect to the corresponding values of reference prisms. After the completion of the freeze-thaw test, the prisms were tested for flexural strength. The test data indicated that the relative performance of the prisms in the freeze-thaw test was a direct function of the residual entrained air content of the concrete, after the addition of the third dosage of a superplasticizer. The concrete incorporating superplasticizer A had a residual air content of only 1.5 per cent after the third dosage had been added and the prisms cast from this concrete performed very poorly in the freeze-thaw cycling and suffered severe damage after only 109 cycles. Concretes incorporating superplasticizers B and C had residual air contents of 2.5 and 3.2 per cent respectively and the test prisms cast from these concretes performed relatively well in the freeze-thaw test. The tests on these prisms had to be discontinued after 500 and 634 cycles respectively, when the prisms had shown excess length change and distress. Concrete incorporating superplasticizer D had a residual air content of 4.0 per cent and the test prisms cast from this concrete were still in excellent shape after 700 freeze-thaw cycles but started showing distress after 830 cycles when the test was terminated, see Table 3.

It should be pointed out that the freeze-thaw tests were performed using ASTM Standard C666-76 and employing Procedure B, rapid freezing in air and thawing in water. ASTM Standard C494-71, Chemical Admixtures, specifies the use of Procedure A, rapid freezing and thawing in water, for the evaluation of concrete incorporating chemical admixtures. Notwithstanding the above, it is considered that the reported freeze-thaw data are valid because the freeze-thaw test is a comparative test, the comparison being carried out with specimens cast from control mixes; for the investigation reported, the test prisms cast from a non air-entrained control mix had completely disintegrated at less than 100 freeze-thaw cycles. The air void system of the hardened concrete was not determined in this investigation.

CONCLUSIONS

Large increases in slumps of superplasticized concretes can be maintained for several hours by the addition of a second dosage of a superplasticizer. The addition of a third dosage was not considered desirable.

The repeated additions of sulphonated melamine (type B) and naphthalene (A and C) based superplasticizers caused substantial loss in entrained air content of the concrete; however for concrete incorporating the lignosulphonate based superplasticizer (type D), the reverse was true. The loss of entrained air adversely affected the performance of the concrete in freeze-thaw tests. More laboratory research data are needed before the use of repeated dosages of superplasticizers can be recommended for use in air-entrained concrete.

The compressive strength of test cylinders cast after the additions of the second and third dosages of sulphonated melamine and naphthalene based superplasticizer was generally higher than the strength of cylinders cast immediately after initial mixing. For concrete incorporating a lignosulphonate based superplasticizer, the reverse was true.

The use of repeated dosages of superplasticizers, particularly those based on lignosulphonates, may adversely affect the time of initial set of the concrete. This aspect also needs to be further investigated.

The results presented in this report were obtained for concrete having a water-cement ratio of 0.42 and made with CSA Type 10 (ASTM Type 1) cement. The superplasticizers may or may not perform as reported in concretes made with other water-cement ratios and with different types of cements, air-entraining agents and aggregates.

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FLOWING CONCRETE

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ABSTRACT Flowing concrete is a relatively new development. It has only been in use for about 10 years in Europe and Japan. The Swedish Cement and Concrete Research Institute has therefore conducted an investigation aimed at finding out more about the properties of flowing concrete in the fresh and hardened state and areas of application within which flowing concrete can be used to advantage have been explored.

INTRODUCTION

Research and development within the concrete industry is aimed primarily at developing more efficient and economical working methods and improving the quality of concrete in order to make it more competitive in relation to other building materials. In recent years, working environment and socio-economic considerations have also come to play an increasingly important role, which has resulted in an increased interest in energy conserving materials and safe, labour saving building techniques.

Concrete with as fluid a consistency as possible has always been desirable on building sites. Hard non-tensioned reinforced concrete structures as well as prestressed concrete structures, which require concrete with a fluid consistency from a production point of view are becoming increasingly common. The trend has been towards more fluid consistencies than are often desirable from the viewpoint of good practice.

The methods which have previously been used for making concrete with a fluid consistency were based on increasing the cement paste content of the concrete or, at worst, increasing the water content alone, leading to poorer concrete properties such as lower strength, poorer durability and greater shrinkage.

In recent years, admixtures known as superplasticizers have come into use. These admixtures can, strictly speaking, be most accurately classified as water-reducing agents, but with a more pronounced effect, and can be used to reduce the water content of the concrete without changing its consistency. However, superplasticizers are mainly used to produce the so-called flowing concrete which, when fresh, has a slump of 180 to 250 mm at the same water-cement ratio as the original concrete.

The use of flowing concrete permits improvements in production costs and the working environment on building sites and at factories manufacturing precast concrete elements, and hence superplasticizers are of great interest. The use of flowing concrete can be expected to increase with time and since flowing concrete is not covered by current concrete codes of practice it is, therefore, important that additional research be conducted in this area aimed at providing supplementary recommendations for industry.

The purpose of the CBI (Swedish Cement and Concrete Research Institute) study has been to investigate the properties of flowing concrete in the fresh and hardened state and to study the areas of application where flowing concrete can be used to advantage from considerations of production economy and improved environmental conditions.

SUPERPLASTICIZERS STUDIED

The chemical compositions of the superplasticizers (all obtained in the form of solutions) are not known, but based on the principal active components of the agents they may be divided into the following groups:

1. Sulphite waste liquor (or modifications thereof),
2. Formaldehyde resins, either sulphonated melamine formaldehyde resins, or sulphonated naphthalene formaldehyde resins,
3. Others.

The principal effect of the superplasticizers in the fresh concrete mix is to facilitate and promote the movement of particles in relation to each other, especially in the fine material fraction, the effect lasting only for a limited period of time (fluidity time).

In all six superplasticizers, belonging to the above three groups according to their composition, as shown below, were studied.

Group 1: Agents 3 and 6

Group 2: Agents 1 and 5

Group 3: Agents 2 and 4

PROPERTIES OF FRESH CONCRETE

Consistency Change and Water Reduction

The effect of the superplasticizers studied on the consistency of the fresh concrete was very marked. If a normal dose of the agent was added to a semi-fluid concrete (of slump equal to 50 to 100 mm), the consistency of the concrete was altered to approximately 200 mm on average (concrete designated as fluid has a slump equal to 100 to 150 mm). The actual magnitude of the consistency change obtained was dependent upon the type of admixture, the dose and the type of cement, as shown in Figure 1. Thus, aside from the amount of liquid in the admixture, a significant consistency change is obtained without the water content of the concrete being altered, leaving the water-cement ratio unchanged. Producing a concrete with a consistency equivalent to that of flowing concrete without the use of superplasticizer requires an increase in the water content of the mix alone (see Figure 2), which is detrimental to the properties, giving lower strength and

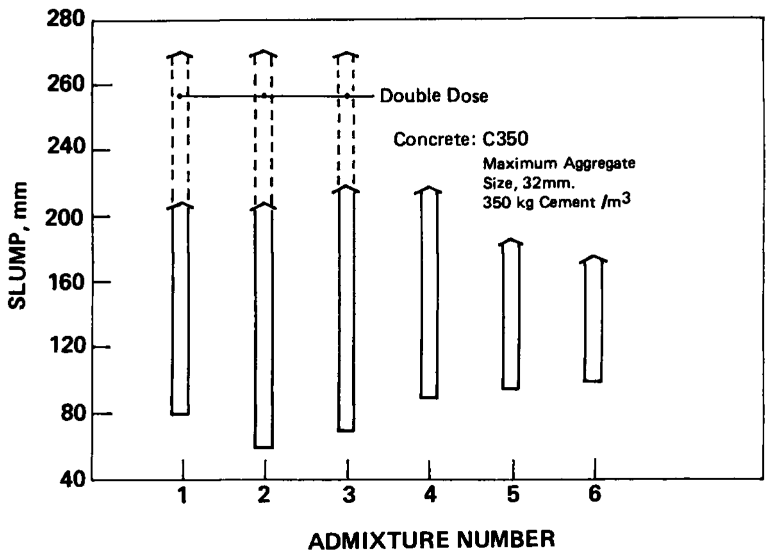

Figure 1 Consistency Change brought about by Different Superplasticizers at Normal Dose and at Double Dose

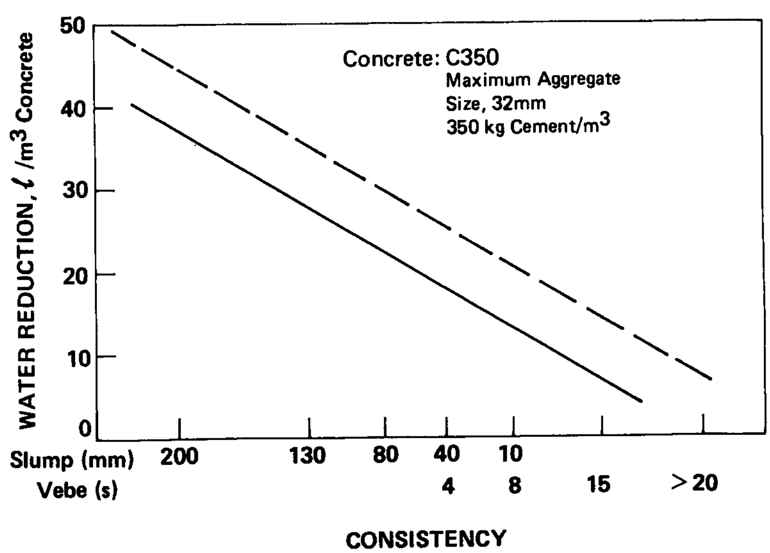

Figure 2 Reduction of Water Requirement of Fresh Concrete with Normal Dose of Superplasticizer within Different Consistency Ranges

greater shrinkage.

Fluidity Time

The more fluid consistency which is achieved in the concrete by the addition of the superplasticizer is maintained for only a certain duration, known as the fluidity time. The loss of fluidity occurs gradually and the fluidity time is defined here as the time from mixing until the consistency of the concrete has returned to its original value.

The fluidity time is temperature-dependent and, as is evident from Figure 3, varies with the type of agent, the different superplasticizing agents employed having markedly different fluidity times. In the case of most of the agents with short fluidity times, however, the time can be extended by the addition of other admixtures, such as retarders, to the mix.

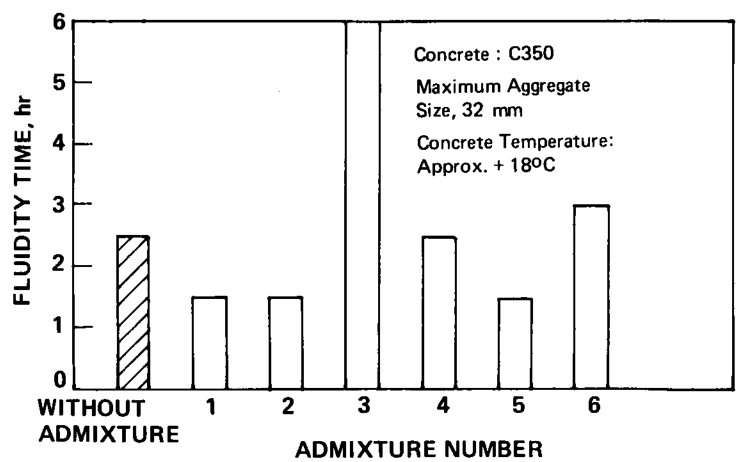

Figure 3 Fluidity Time with Use of Different Superplasticizing Admixtures (The time for concrete without any admixture has been determined from approximately 22 cm slump to about 8 cm.)

Water-Aggregate Separation

When compared to the original concrete, in most cases the flowing concretes had less water separation, see Figure 4.

The water-retaining capacity of the superplasticizers is even more striking if the comparison is made with a concrete of equally fluid consistency achieved without the use of a superplasticizer. The risk of aggregate segregation exists when compacting the concrete with vibration, but if the amount of vibration is adjusted to suit the consistency, flowing concrete possesses satisfactory stability. At slumps of more than about 260 to 270 mm, however, there is a great risk of aggregate segregation, even without vibration. In order for flowing concrete of this degree of fluidity to possess satisfactory stability the original concrete must be proportioned in accordance with normal rules for good stability, particularly with respect to the problem of a sufficient amount of fine material. The amount of fine

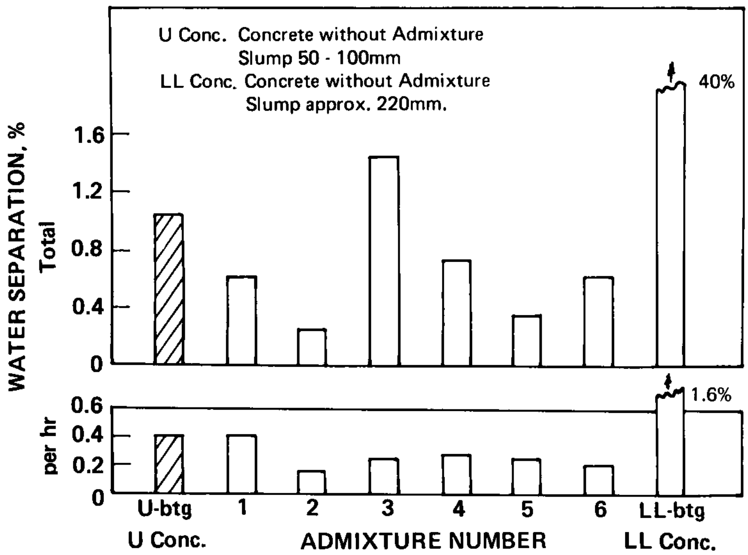

Figure 4 Water Separation of Fresh Concrete with use of Different Superplasticizers

material, defined as the cement + fines of size < 0.25 mm, should be at least 250 kg/m^3.

Hardening Time

Depending on the type of admixture and on the type of cement, the concrete's hardening time may be either shorter or longer than that of the original concrete. However, the most common effect is a lengthening of the hardening time. This is especially true of the sulphite waste liquor type of admixture, where the concrete may have a hardening time which is several hours longer than that of the normal concrete. This retardation may be difficult to control in the case of the sulphite waste liquor type of admixture.

PROPERTIES OF HARDENED CONCRETE

Strength

The use of superplasticizer results in improved or unchanged compressive strength, see Figure 5. Most marked is the increased short-term strength produced by certain agents which are used widely within the precast concrete industry, where short stripping and detensioning times are of the essence.

Shrinkage

The relatively few shrinkage tests which have been done in the study thus far indicate that 91-day-old flowing concrete exhibits somewhat greater shrinkage than the original concrete. The average values of drying shrinkage for the original

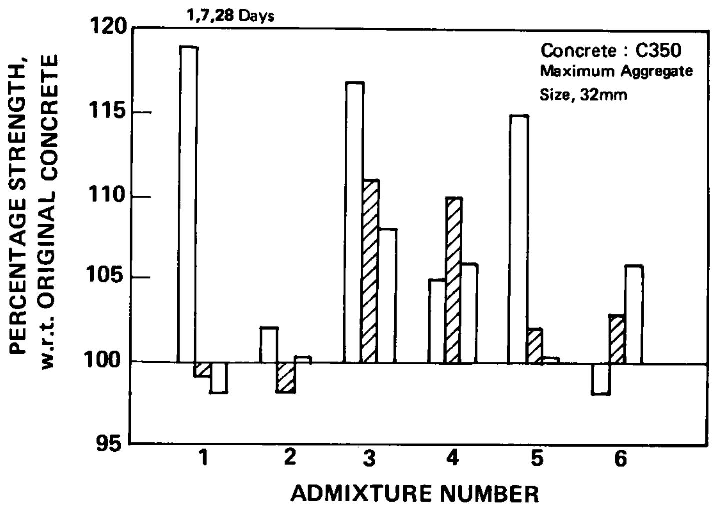

Figure 5 Strength of Flowing Concrete at Different Ages

concrete was found to be approximately 0.35 per cent while that for the flowing concrete was found to be approximately 0.40 per cent.

Air Content and Frost Resistance

The entrained air content of fresh concrete is generally adversely affected by the addition of superplasticizer, as shown in Table 1, although the effect does vary.

Table 1 Change in Air Content and Spacing Factor of Superplasticized Concrete

ADMIXTURE		ORIGINAL AIR CONTENT	CHANGE IN AIR CONTENT PERCENTAGE FOR CONCRETE OF SLUMP		CHANGE IN SPACING-FACTOR	
Number	Group	%	200 mm	260 mm	from	to
3	1	2	+0.5	-1.0		
		5		+3.0	0.18	0.13
1	2	2	-0.5	-1.5		
		5		-0.9	0.16	0.22
5	2	2	+0.5	-1.5		

The spacing factor of hardened concrete, i.e. the distribution of air pores resulting from the use of an air-entraining agent, was determined for agents 1 and 3 only. Agent 3 caused an improvement of the spacing factor, while agent 1 caused a reduction. These variations were achieved in addition to the changes caused by the change in the air content of the concretes.

TRANSPORT OF FLOWING CONCRETE

The transport facilities which are commonly used for normal concrete can also be used for flowing concrete, although owing to the risk of splatter associated with transport in a non-agitating truck some caution should be exercised. The advantages of flowing concrete are perhaps most manifest in connection with the movement by concrete pumps on site. Flowing concrete of the correct composition permits the use of a lower pumping pressure while the higher degree of piston fill produces a higher pumping capacity.

COMPACTION OF FLOWING CONCRETE

Certain superplasticizer manufacturers state in their literature that flowing concrete does not have to be vibrated. However, the studies carried out by the CBI in Sweden show that in order to obtain an acceptable result with regard to porosity, strength and surface appearance, the concrete must be compacted, normally by vibration, although the amount of compaction given can and should be much less than for the original concrete. Indeed, excessive compaction can lead to segregation.

As a result of its considerably lower compaction requirement, flowing concrete offers advantages with regard to both pouring convenience and working environment. Perhaps its greatest advantage from the viewpoint of improvement in the working environment is that flowing concrete can be used to reduce noise nuisance considerably. In principle, noise nuisance can be reduced by two means: shorter exposure time and reduced noise level. Both means can be employed simultaneously, since the formwork can be filled completely right from the start. The sound level is also reduced in that a shorter period of vibration is required for flowing concrete and in certain pilot tests with form-vibrated vertically cast wall elements (0.13 m x 1.0 m x 1.5 m) it was shown that the noise level could be reduced radically, see Figure 6.

APPLICATIONS OF FLOWING CONCRETE

Flowing concrete is particularly suited to slab construction and is generally suited to most of the areas of application of normal concete. Flowing concrete has been used both on construction sites, especially in Germany, and in precast element plants. In Sweden, flowing concrete has been tested at numerous precast element plants in an attempt to increase production. The conclusions drawn have varied; some plants have started to use flowing concrete extensively, others have not. No relationship between the conclusions reached and the type of product being produced has been found. Another area where the use of flowing concrete could be advantageous is in the casting of narrow, heavily reinforced structures.

ECONOMY

Flowing concrete offers numerous advantages purely from the viewpoint of production economy, mainly in respect of the reduced labour requirements, lower energy needs and lesser plant wear. Its higher early strength is also advantageous. However, the necessary admixtures are expensive. The price level has not yet stabilized, but an extra material cost of 20 to 30 Swedish Kr./m^3 (£2.3 to £3.4/m^3) can be expected, although in some cases the price has been kept down to only 10 Swedish Kr./m^3 (£1.15/m^3). The normal price of one cubic meter of concrete is about 230 Swedish Kr. (£26.44).

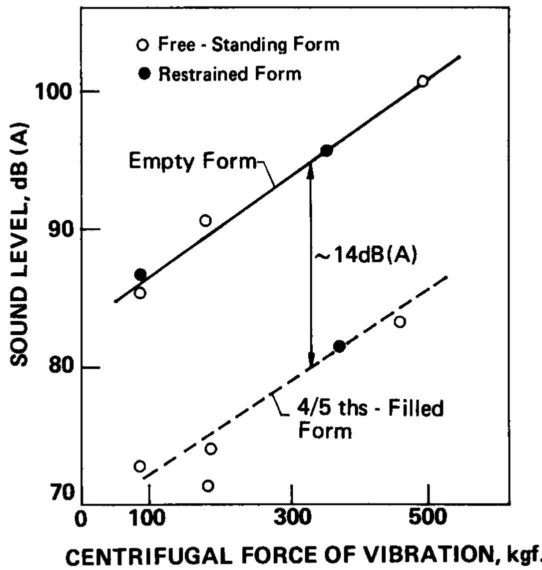

Figure 6 Pilot Test with Flowing Concrete
in Form-Vibrated Steel Wall Forms

It is difficult to make a general statement as to whether the improvement in production economy can cover the extra materials costs, although it can be postulated that the effect on the price of the final product, mainly owing to the fact that present machinery can be utilized to full capacity, must be extremely marginal.

UTILIZING THE ADVANTAGES OF TYPE-K SHRINKAGE-COMPENSATING CEMENT CONCRETE IN VARIOUS TYPES OF SLAB DESIGNS—A REPORT COVERING FOURTEEN YEARS OF U.S.A. USAGE

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ABSTRACT Shrinkage-compensating cement (type-K) has been commercially produced in the U.S. since 1965 and is used extensively to eliminate or minimize cracking caused by drying shrinkage in concrete. This paper outlines its history, field stability measurements, properties, structural design, placement methods and tips on placement-finishing and curing.

INTRODUCTION

Shrinkage-compensating cement concrete has been commercially produced in the United States since 1965 and is thoroughly described in American Concrete Institute publications (1, 2).

For years the construction industry has tolerated cracks in concrete. Whether you are an architect, an engineer, a contractor, or an owner, it is doubtful that you have not, at one time or another, suffered the unpleasant experience of having some sort of cracking occur on your projects.

There are many types of concrete cracks: structural cracks, cracks resulting from temperature changes, plastic shrinkage cracks and drying shrinkage cracks. Of all the various types, drying shrinkage cracking is one of the most common and troublesome forms.

Much has been done in the field of research and development toward elimination and/or control of drying shrinkage cracks. Laboratory and field tests have been exhaustive, resulting in numerous changes in building codes and design methods. This has helped to a degree, but is not adequate because standard Portland cement concrete always undergoes a negative volume change upon drying. Shrinkage under restraining conditions produces tensile stresses which frequently overcome the tensile strength of the concrete. Now a commercial method of compensating for stresses due to drying shrinkage makes it possible to produce concrete slabs virtually free of this type of cracking by using a modified Portland cement called type-K expansive cement.

This paper deals with applications of this type of cement for over fourteen years in large area heavy industrial concrete slabs on grade in which only construction joints are required and conventional control joints (sawn) are eliminated.

HISTORY

Type-K cement is a 'controlled expansive cement' designed to compensate for the tensile stresses which normally develop in concrete due to drying shrinkage and manufactured by conventional Portland cement processing methods under ASTM Specification C-845-76 and patents owned by the Chemically Prestressed Concrete Corporation. The cement was developed during the early 1960's by Professor Alexander Klien at the University of California (Berkeley) and is now covered by four U.S. patents which are registered in twenty other countries (3). It is commercially available at seventeen distribution centres throughout the U.S.A. A standard industrial test method, ASTM C-806-75, has been developed by the American Society for Testing and Materials for evaluating expansive cement mortars.

To outline the way in which type-K expansive cement works it is necessary to first consider the behaviour of standard Portland cement. A hardened standard Portland cement concrete shrinks as it dries, see Figure 1, and this drying shrinkage is conducive to cracking. In a reinforced standard Portland cement concrete slab the concrete is bonded to the reinforcing steel and when it tries to shrink, the restraint from the steel places the concrete into tension. Due to the low tensile strength of concrete, particularly at early ages, only a little restrained shrinkage may be sufficient to develop tensile stresses which exceed the tensile strength of the concrete. Thus the concrete cracks to relieve these tensile stresses.

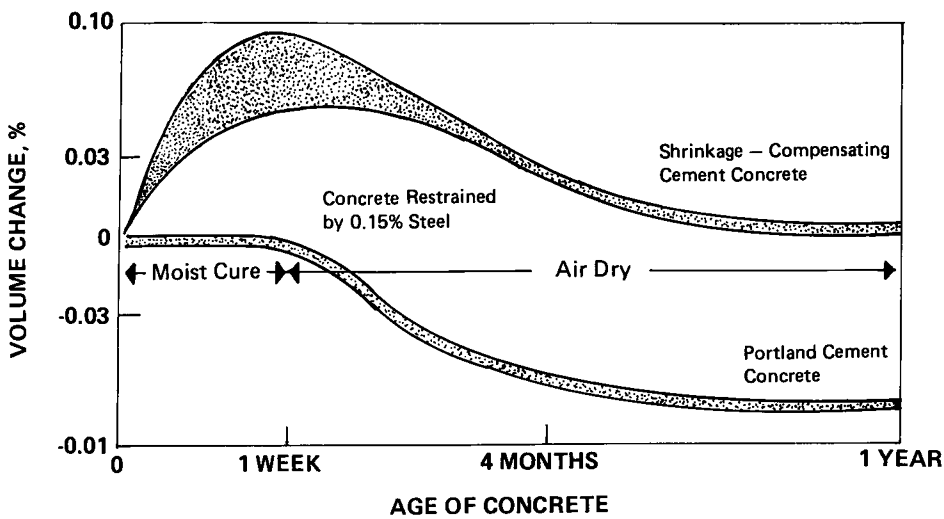

Figure 1 Typical Volume Changes of Two Kinds of Concrete

In a reinforced concrete slab incorporating type-K cement, as the concrete sets it bonds to the steel as normal. At the same time, however, the controlled expansion reaction causes a positive volumetric expansion of the reinforced concrete. Since the concrete is bonded to the steel, this expansion will place the steel in tension and the concrete in compression. The concrete is mildly precompressed, but at a level of magnitude which is much less than that of conventional prestressing. The expansion reaction is complete in the first few days of concrete curing. Later, when the type-K cement concrete is exposed to drying conditions, it will shrink just as standard Portland cement concrete. Unlike standard Portland cement concrete, however, the shrinkage simply relieves the slight precompression and does not build up tensile stresses.

FIELD DIMENSIONAL STABILITY TESTS

Field measurements (4) were undertaken in 1966 in order to record the actual on-site concrete expansion and subsequent net length changes in two municipal structures (coastal and inland) using type-K shrinkage-compensating cement. For the purpose of control, measurements were also taken on an industrial building constructed using a type II Portland cement. The three structures selected for this study were similar in all respects, i.e. other mix ingredients, structural design, general location, age, etc., except for the cement choice. From the results obtained, which are presented in Figure 2, it is apparent that the two shrinkage-compensating slabs offered two very important advantages: (i) a greatly minimized possibility of cracking in the early stages of curing because of the mild prestress induced by the expansive concrete, and (ii) a greatly reduced magnitude of final net contraction of the slab.

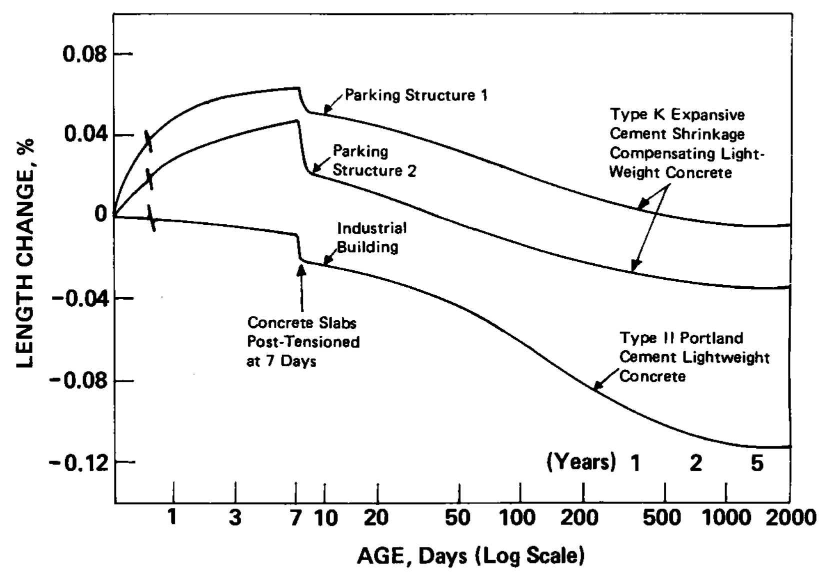

Figure 2 Five Year Length Change History of Lightweight Concrete Slabs

TYPE-K CEMENT CONCRETE PROPERTIES

This concrete tends to set somewhat faster than normal concrete in very hot weather, and somewhat slower in cold weather. The tensile, flexural and compressive strength development of shrinkage-compensating concretes after expansion is similar to that of Portland cement concretes under both moist and steam curing conditions.

The modulus of elasticity of shrinkage-compensating concretes is comparable to that of standard Portland cement concretes, and the creep characteristics of shrinkage-compensating concretes indicate that their creep coefficients are within the same range as those of Portland cement concretes of comparable quality. There has been no observed difference between Poisson's ratio of shrinkage-compensating and normal concretes. Tests have shown that the coefficient of thermal expansion is consistent with that of corresponding Portland cement concretes and that concretes made with type-K cement are comparably resistant to freezing and thawing, deicer scaling and abrasion as standard Portland cement concretes of the same water-cement ratio. The effects of air content and aggregates on these measures of durability are essentially the same for both types of concrete. Shrinkage-compensating cements made with type I Portland cement clinker may be undersulphated with respect to the aluminate available and are, therefore, susceptible to further expansion and possible disruption after hardening when exposed to an external source of additional sulphates. Type-K shrinkage-compensating cements made with type II or type V Portland cement clinker are adequately sulphated and produce concretes having sulphate resistance equal to or greater than that of standard Portland cement made from the same clinker type (5).

The restrained expansion of shrinkage-compensating concrete produces a dense matrix (characteristically non-bleeding) which serves to reduce the permeability as compared to corresponding concrete.

It is recommended that the amount of expansion achieved in a given mix, which is as important as strength in the performance of a type-K cement concrete, be determined by measuring the length change of restrained 76 mm x 76 mm x 254 mm (3 in x 3 in x 10 in) concrete prisms. This method has been used extensively in trial mixes (6).

STRUCTURAL DESIGN CONSIDERATIONS

General

The design of a reinforced concrete slab using a type-K cement concrete should naturally conform to applicable building standards. At the same time, adequate expansion must be provided to compensate for subsequent drying shrinkage to minimize cracking. Since the final net result of expansion and shrinkage is essentially zero, no structural considerations need normally be given to the stresses developed during this process (7).

Reinforcement and Restraint

In some non-load-bearing members, slabs on grade and lightly-reinforced structural members, the amount of steel reinforcement used may be less than the recommended minimum when using expansive concrete. For such designs, a minimum ratio of 0.15 per cent should be used in each direction, an amount which is approximately equal to that recommended by ACI 318 for temperature and shrinkage stresses. In 1968 the Portland Cement Association began a survey of over 100 shrinkage-compensating concrete structures. Their report, presented in 1977 (8), showed that many slabs were in excellent condition with even lesser amounts of steel.

Location of Reinforcement

In slabs on grade, where most of the drying occurs in the top portion, the reinforcement should be placed in the upper half of the slab (preferably one-third the distance from the top) while still allowing for adequate cover, see Figure 3. At re-entrant corners, at least one #4 bar approximately 0.9 m (3 ft) in length placed diagonally across the corner should be used, Figure 4.

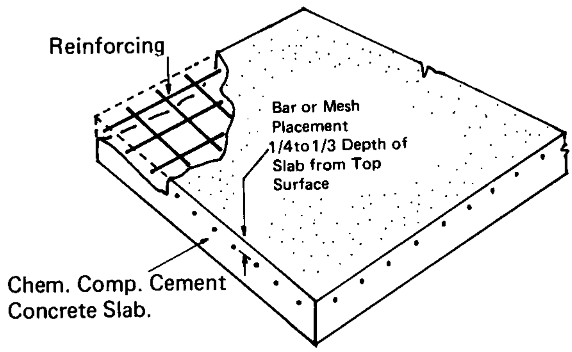

Figure 3 Reinforcement Position in Slab

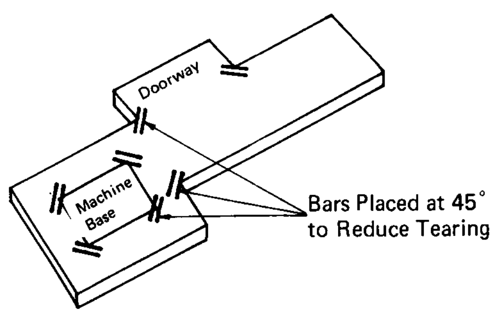

Figure 4 Reinforcement Arrangement at Corners

Joints

The three main types of joints used in concrete slab construction are expansion, construction and contraction (or control) joints.

> Expansion joints are provided in slabs to control movement due to thermal expansion or contraction forces caused by temperature changes. Type-K cement concrete is not intended to be used to effect any change in location of expansion joints.
>
> Construction joints are those joints formed at the beginning and end of a particular concrete placement.
>
> Contraction (or control) joints are joints which are sawed,

formed, or otherwise placed in slabs between construction and expansion joints. Their primary purpose is to induce drying shrinkage cracking to take place in a controlled fashion along these weakened planes or joints. One of the great advantages of shrinkage-compensating concrete is that large slab areas can be placed without the need for contraction joints and with fewer construction joints.

A comparison, carried out for a major soft drink bottling company, of joint layouts for a typical slab on grade designed using shrinkage-compensating concrete and conventional Portland cement concrete is shown in Figure 5. Each slab has the same area of 13790 m^2 (148400 ft^2) but the shrinkage-compensating concrete slab on the left of the Figure required 3455 m (11340 ft) less sawed contraction (or control) joints.

Normally, with standard Portland cement concrete, engineers use construction and/or contraction joints to break slab areas into 4.5 to 7.6 m (15 to 25 ft) squares. The use of type-K cement concrete generally permits significantly larger joint-free areas. Slabs located inside enclosed structures, where temperature changes are small, have been placed in areas as large as 1858 m^2 (20000 ft^2) without joints. The J.C. Penny Company in the U.S.A., for example, has established a standard slab design based on 12.19 m (40 ft) column spacing in their large, 185800 m^2 (2000000 ft^2), warehouses and using type-K cement concrete in pours of 1338 m^2 (14400 ft^2) or 892 m^2 (9600 ft^2) between construction joints with no intermediate sawn control joints. The slab is generally 152.4 mm (6 in) thick using natural aggregate in 28 N/mm^2 (4000 psi) concrete with 0.17 per cent steel and a trap rock hard trowelled surface with two directional spray-on curing compound. During construction of their last catalogue warehouse of this size in Reno, Nevada the contractor averaged two placements per day, each of 892 m^2 (9600 ft^2), giving a total daily concrete placement of 1784 m^2 (19200 ft^2). They have similar structures in Kansas City (Kansas), Manchester (Conn.) and Columbus (Ohio) and more are planned.

PLACEMENT METHODS

Modern warehouse floors are among the most demanding structures built with concrete. They must be extremely durable to stand up to the constant trafficking of heavily loaded wheeled vehicles while remaining virtually crack-free and they must be level within the very narrow tolerances (typically 3 mm in 3 m or 1/8 in. in 10 ft) demanded by modern automated material handling equipment.

The client's major concern is to avoid concrete maintenance problems and therefore minimization of construction joints and the elimination of sawed joints and random shrinkage cracks are among the prime design objectives.

Two high production methods of slab construction have been used very successfully, differing only in the selection, by the contractor, of the mechanical power screed and the placement method for the reinforcing steel.

The contractors will probably use power graders equipped with laser leveling equipment to prepare the subgrade and before placing any concrete will insist that the roof and walls form an enclosure. A key factor in getting the floor to perform as specified is placing the welded wire fabric (4 x 4 W4) in the correct position. In the first construction method, for a 152 mm (6 in) thick slab, 102 mm (4 in) of concrete is first placed, then the welded wire fabric is laid onto the concrete surface and the final 50 mm (2 in) is placed. A Clairy power screed with 7.9 m (26 ft) rollers, riding on screed rail supports spaced 6.1 m (20 ft) apart, is used to level and compact the concrete. To set the screed supports, the contractor

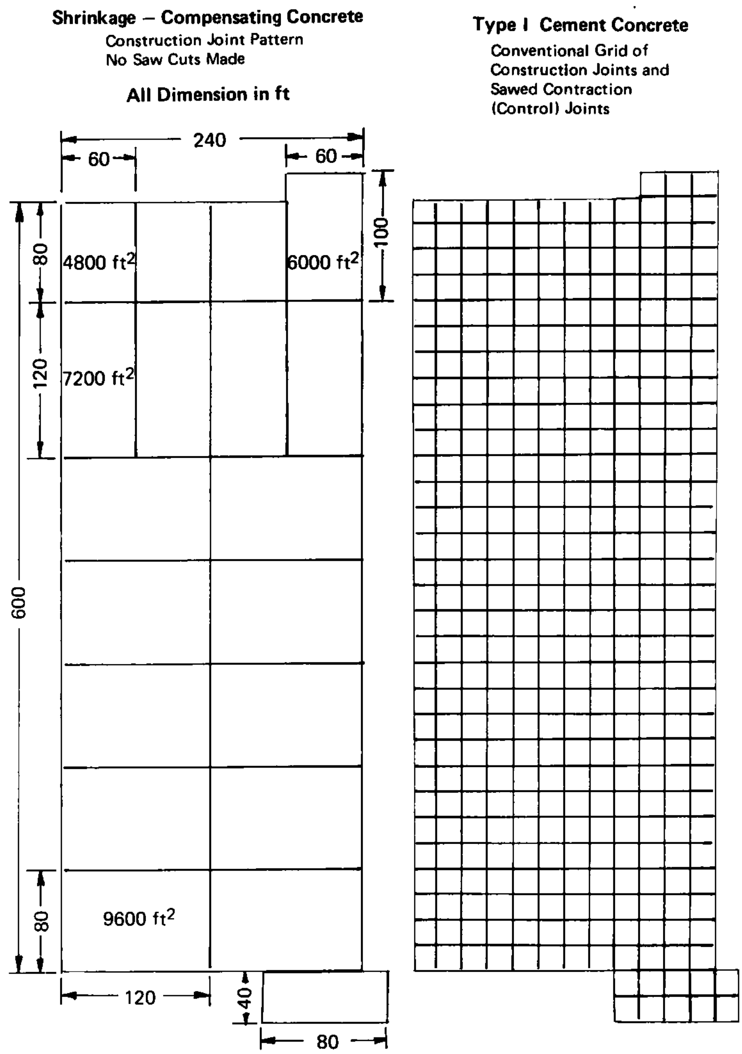

Figure 5 Comparison of Contraction Joint Requirements of Slabs With and Without Shrinkage-Compensating Cement

first drills 203 mm (8 in) deep holes in the compacted subgrade, inserts a paper sleeve and then fills the hole with concrete. The screed rail support is inserted into the sleeve so that it can be removed after serving its function.

To improve surface durability, trap rock mixed with shrinkage-compensating cement is spread over the plastic concrete surface at 5 kg/m^2 (1 lb/ft^2), then power trowels finish the surface.

In the second construction method the welded wire fabric is first placed on the surface of the compacted subgrade that has been thoroughly wet down the night before. The concrete trucks back down the lane between columns and begin discharging concrete as two men position the flat welded wire fabric, by means of support blocks, in the upper third of the slab. Depending on the column spacing, the contractor will use either a 15.2 m (50 ft) or 12.2 m (40 ft) wide pneumatic vibratory screed to compact the 152 mm (6 in) thickness of concrete. All other procedures remain similar. In both of these methods, concrete can be discharged in each lane at the rate of 4.6 to 5.6 m^2 per minute (50 to 60 ft^2/min) on these 152 mm (6 in) slabs. Both methods have been used very satisfactorily.

PLACEMENT TIPS FOR EXPANSIVE-CEMENT CONCRETE

For slabs on grade the prepared subgrade should be wetted thoroughly (not just sprinkled lightly) before placing this concrete. This reduces the rapid loss of mixing water by absorption since shrinkage-compensating concrete bleeds less than conventional Portland cement concrete.

In exposed areas, in hot, dry and windy weather, slabs placed over a vapour barrier tend to dry unevenly and sometimes crust on top. To avoid plastic shrinkage cracking, if a vapour barrier is required it is recommended that 25 to 75 mm (1 to 3 in) of sand be placed over the vapour barrier and prewetted. Hot weather concreting measures should be employed. In enclosed areas, normal good concreting practices used with Portland cement concrete are suitable.

It is not normally recommended to keep shrinkage-compensating cement concretes in transit mixers for more than 90 minutes as slump loss will require water additions at the unloading area which does reduce the expansive potential (10). Shrinkage-compensating concretes will generally exhibit more cohesiveness (or fat) than conventional concrete and will have less tendency to segregate. This makes it particularly adaptable to placement by pumping.

FINISHING TIPS

One notable difference between this type of concrete and conventional Portland cement concretes is the almost total absence of bleed water after screeding and floating. Even though a high slump is used, the excess mixing water is consumed at a fairly rapid rate in the early hydration process. Normally a mix design will require from 10 to 15 per cent more water without loss in strength when compared with normal concrete. This does allow the contractor to work with a greater slump than is normal in concreting practice.

Since bleed water is not normally present, finishers working with these concretes for the first time tend to start finishing operations too soon. Most experienced finishers like the finishing characteristics of this concrete, however, because they are 'fatter' and have more 'butter'.

CURING TIPS

It must be remembered that moisture remains an essential ingredient in obtaining proper hydration and required strength, and curing of concrete flatwork should commence immediately after final finishing. The normally accepted methods of curing are satisfactory for these concretes.

It is essential, because of the large area pours which are possible, that sprayed-on curing membranes be applied in two directions.

Normally forms are struck as soon as possible so that the controlled expansion process may freely place the steel in tension and the concrete in compression.

It is normal to have at least two sides of a large pour free to expand before placing new concrete against the previous pour, otherwise the general methods recommended in the ACI Manual of Concrete Practice, Part 1 (ACI 605 and ACI 306) are applicable.

CONCLUSION

The most suitable conclusion is to summarize the advantages to slab construction from the use of type-K shrinkage-compensating cement concrete, which have become apparent over the last 14 years.

It reduces the frequency of occurrence and size of concrete cracks due to drying shrinkage.

It provides a substantial reduction in the amount of sawn control joints necessary, so there is less sealing and caulking of joints. It also reduces the cost of long-term joint maintenance because there are fewer joints.

It allows the contractor to work with higher slumps without loss in compressive strength of the concrete.

Due to its greater 'butter' or 'fat' it has excellent pumping characteristics.

It has excellent resistance to abrasion because it is a non-bleeding type of concrete, which also reduces bleed problems in concrete making for easier finishing.

It provides dimensional stability.

It is ideally suited to post-tensioning operations.

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SULPHUR-TREATED CONCRETE SLABS

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ABSTRACT To enhance significantly both the strength and durability of concrete, the technique of polymer impregnation has received wide spread recognition in recent years. However, the price of monomers prevents a large scale use of this technique. The objective of this paper is to demonstrate the applicability of sulphur as an attractive substitute to monomers for impregnation. The sulphur impregnated concrete slabs could give an equivalent relative strength increase compared to PMMA-impregnated slabs. There is a substantial reduction in water absorption, permeability and acid attack as well as improvement in abrasion resistance for sulphur-treated slabs as compared to the corresponding untreated units.

INTRODUCTION

It is well known that both the mechanical and physical properties of Portland cement concrete can be improved by impregnating it with a liquid monomer and then thermal-catalytically polymerizing it to form a solid polymer throughout the concrete (1-3). Since polymer impregnated concrete (PIC) effectively resists penetration by water and salt solutions, significant research progress has been made in recent years to apply PIC technology to concrete slabs in which the corrosion of steel reinforcing rods is a pressing problem. The price of oil based monomers, however, prevents large scale use of this technique.

In recent years, interest in sulphur utilization in concrete (and in asphalt paving materials) has been renewed. One of the foremost reasons for this is the abundance of surplus sulphur in connection with meeting the environmental pollution standards. Sulphur melts at 115 to 120°C and the viscosity of molten sulphur remains relatively low, from about 12.5 cp at 120°C to 6.6 cp at 160°C. In this workable range, dewatered porous material can be impregnated with molten sulphur in the same manner as polymer impregnation, with sulphur solidifying on cooling. These characteristics make sulphur a very attractive substitute for polymers as impregnators as long as they are able to transform from the liquid to the solid state. There is the additional reduction in cost with the elimination of the polymerization step required in the polymer impregnated concretes.

Application of sulphur impregnation to the Portland cement concretes (and other building materials such as masonry blocks and clay bricks) has shown that the strength, modulus of elasticity and durability can be increased significantly (4-8). The main aims of this investigation were (1) to study quantitatively the water absorption, acid resistance, permeability and abrasion resistance of sulphur impregnated concrete slabs, and (2) to evaluate the strength and modulus of elasticity of sulphur impregnated concrete slab specimens.

EXPERIMENTAL PROGRAMME

Materials

The experimental investigation included two types of concrete specimens: concrete slabs, with water-cement ratios of 0.40 and 0.70, having nominal dimensions of 203 mm x 203 mm with a thickness of 25.4 mm and concrete cylinders with the same water-cement ratios having nominal dimensions of 76 mm dia. x 152 mm. Six samples of each type were tested to obtain average values. The results of the experiments were analyzed and compared with the performance of corresponding untreated control units.

The commercial grade flour sulphur (99.5 per cent purity) was obtained in 23 kg bags. Sulphur has a specific gravity of 2.07 in the solid form and 1.79 in the liquid molten form, with viscosity decreasing from 12.5 cp at 120°C to 6.6 cp at 160°C. The temperature of the molten sulphur was therefore kept between 122 and 160°C throughout the impregnation process.

Impregnation Procedure

To study the characteristics of drying and moisture loss, the concrete slabs and concrete cylinders were fully saturated with water and placed in an electric oven at approximately 145°C, the optimum temperature for impregnation with sulphur. The results are presented in Figure 1, from which it can be seen that concrete slabs with water-cement ratio of 0.70 lost all their surplus water in 16 hr and slabs with water-cement ratio of 0.40 in 20 hr; the corresponding values for cylinders with water-cement ratios of 0.70 and 0.40 were found to be 22 and 32 hr respectively.

In the laboratory, all specimens were dried in an oven at 145°C for approximately 24 hr and then immersed hot in a molten sulphur bath, Figure 2, heated by two line gas burners and maintained at between 122 and 160°C. The specimens were then removed from the vessel and immediately immersed in water for a few seconds to prevent rapid drainage of sulphur from the surface pores, and subsequently were left at room temperature to cool in the air. The sulphur, on cooling, crystallizes rapidly to form a hard, shiny yellow coating on the surface of the specimen. The samples were weighed before and after impregnation and the sulphur loadings were calculated.

All the specimens were cured initially for three days in a standard fog room. Each mix specimen was then dried for 24 hr and impregnated with sulphur for 1/2, 1, 2, 4, 6, 8 and 10 hr respectively.

TEST RESULTS

The sulphur loading generally increased with increasing impregnation time, see Figure 3. For a given water-cement ratio and a given time the percentage of sulphur loading is a function of surface to volume ratio of the specimen. For a given surface to volume ratio and a given time, the percentage of sulphur loading depends on the water-cement ratio.

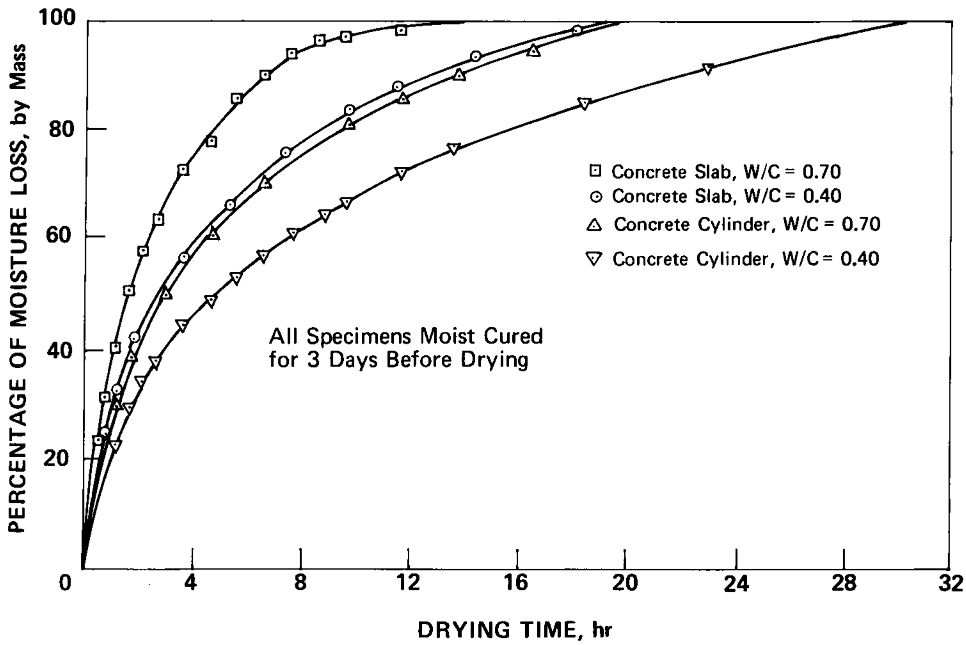

Figure 1 Moisture Removal Time Curves for Concrete Slabs and Cylinders Dried at 145°C in Oven

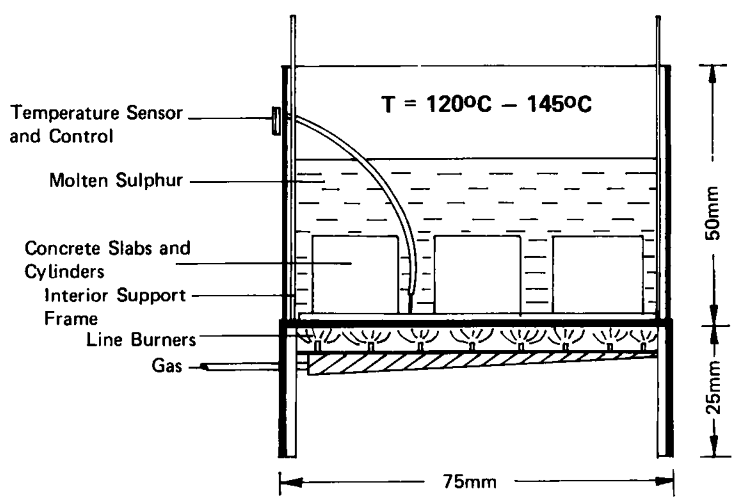

Figure 2 Steel Bath Used for Impregnating Concrete Slabs and Cylinders with Molten Sulphur

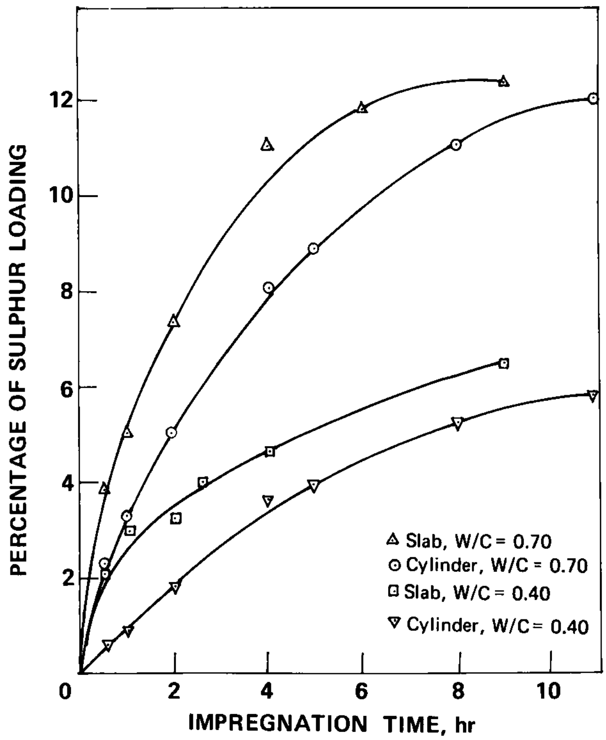

Figure 3 Percentage of Sulphur Loading for Concrete Slabs and Cylinders After 3 Days of Moist Curing

It was found that the sulphur infiltration into the concrete specimen was closely related to the drying characteristics and porosity of the specimen. The presence of internal pores in the concrete generally influences the specific gravity of the specimen and is important in the study of sulphur impregnation of concrete. The term specific gravity also refers to the density of the specimen. Figure 4 shows the bulk specific gravity of concrete slabs and cylinders as a function of the percentage of sulphur loading. The values of bulk specific gravity for each type of specimen were obtained according to ASTM Standard Method of Test for Specific Gravity, Absorption and Voids in Hardened Concrete (C 642). There seems to be a linear relationship between the bulk specific gravity and the percentage of sulphur loading.

Effect of Sulphur Impregnation on Water Absorption

The effect of sulphur impregnation on absorption was found by subjecting the control specimens and sulphur treated specimens to both the ASTM C67 24-hour cold water immersion test and 5-hour boiling water test. Tap water was used in lieu of distilled water. The results of the absorption tests are shown in Figure 5. For the 24-hour cold water immersion test, concrete slabs with the same percentage of sulphur impregnation showed a higher reduction in water absorption with a lower water-cement ratio. The same phenomena were observed for the 5-hour boiling water immersion test. The maximum reduction in water absorption recorded was 90 to 95 per cent, corresponding to concrete slabs with a water-cement ratio of 0.70 at a sulphur loading 10 per cent. Figure 6 shows the relationship between volume of permeable voids and sulphur loading. It appears that for a given water-cement ratio the volume of permeable voids decreases with an increasing percentage of sulphur loading. In general, there was a substantial reduction in water absorption for the sulphur impregnated concrete slabs.

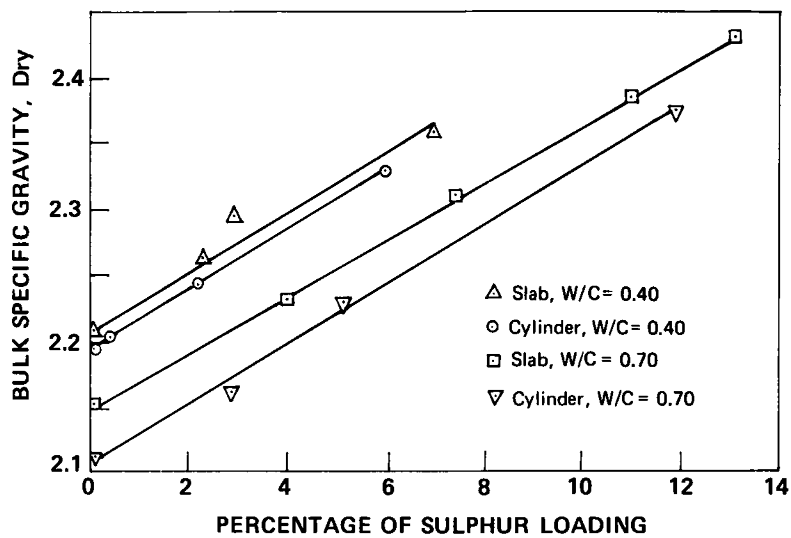

Figure 4 Bulk Specific Gravity of Sulphur Impregnated Concrete

Effect of Sulphur Impregnation on Abrasion Resistance

The abrasion test machine used in this investigation was a three-cone rock bit type. The rate of rotation can be controlled by the operator and was 250 rev/min for the test. The geometry of the three-cone bit provides both a well balanced cutting structure and efficient use of all available spaces. This not only allows a more even distribution of weight but also permits the individual parts to be large enough to meet strength and durability requirements. Six abrasion specimens of each water-cement ratio were tested for 12 minutes and wearing was recorded at 2, 4 and 8 minute intervals. The difference between the original mass and the final mass of the test specimen was expressed as a percentage of the original mass of the test sample. The test results are shown in Figure 7. For a water-cement ratio of 0.40 there was only a little improvement in the abrasion resistance of the sulphur impregnated concrete, but for a water-cement ratio of 0.70 a substantial improvement in abrasion resistance was observed.

Effect of Sulphur Impregnation on Acid Resistance

The sulphur impregnated concrete slabs and the corresponding untreated control units were immersed in 2½, 6 and 12 per cent sulphuric acid solution.

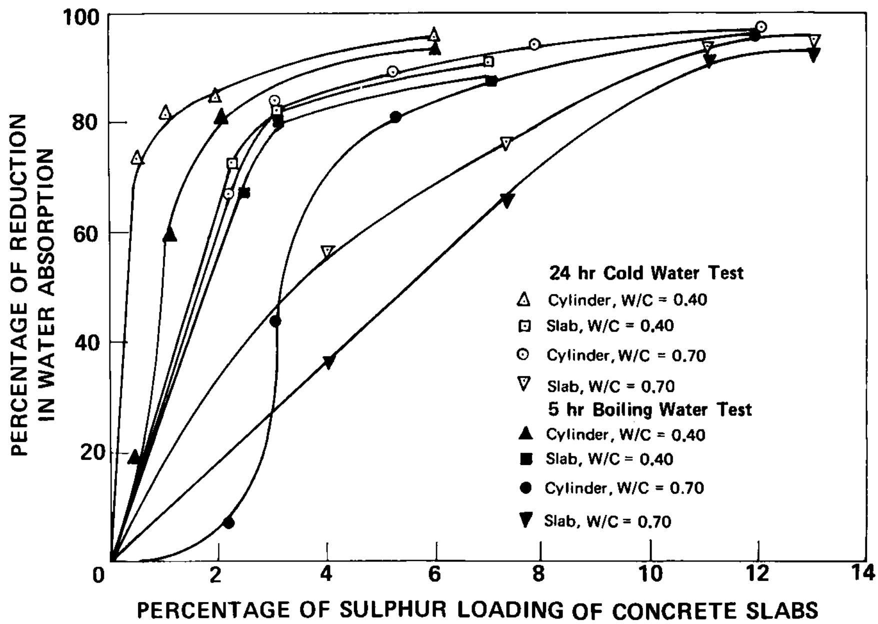

Figure 5 Percentage of Reduction in Water Absorption of Concrete Slabs and Cylinders

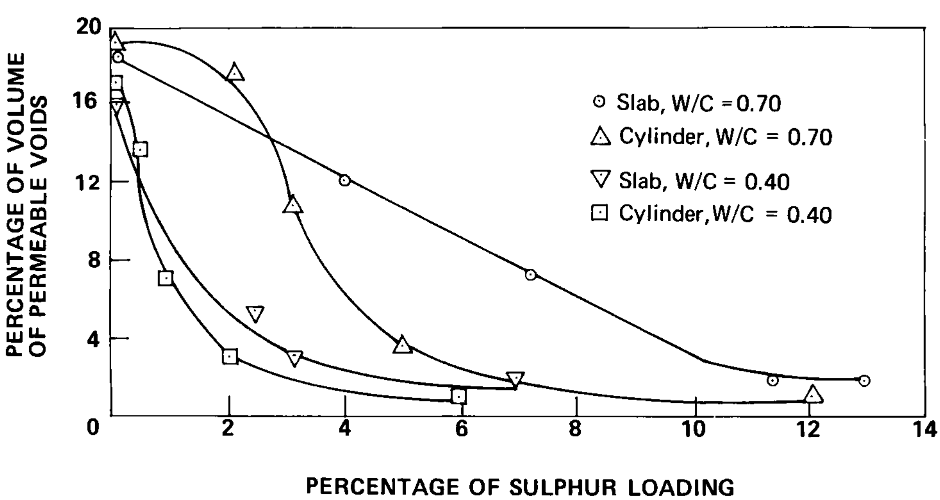

Figure 6 Percentage of Volume of Permeable Voids of Sulphur Impregnated Concrete

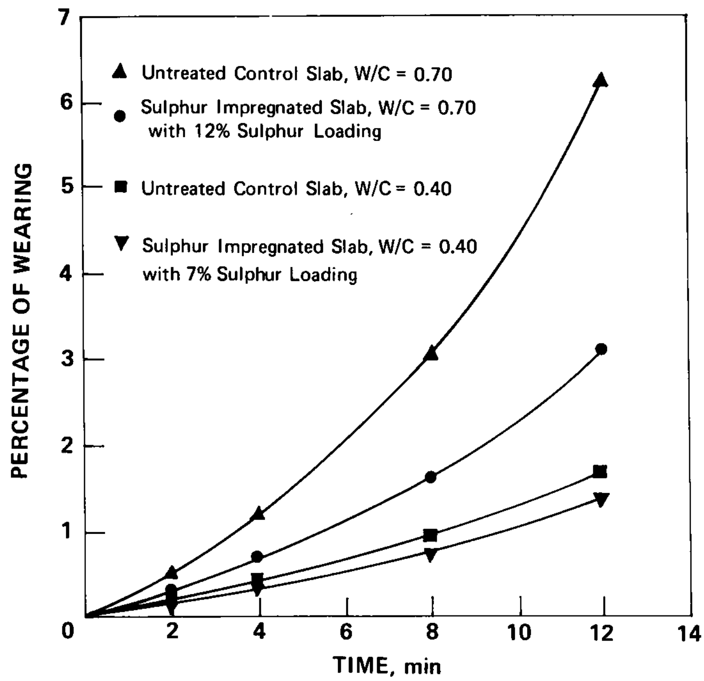

Figure 7 Percentage of Wearing as a Function of Abrasion Time

The comparative tests of the rate of digestion showed that:

i) both the sulphur treated slabs and the corresponding untreated control units were unaffected by weak acid solution (2.5 per cent or less), there being little change in mass of the specimens,

ii) for the concrete slabs in 6 and 12 per cent acid solutions, the sulphur treated specimens were more resistant to acid attack than the control specimens. It was observed in both cases that the lower the water-cement ratio of the sulphur treated concrete specimens the more effective the resistance to acid attack. The present investigations are being extended to determine the long term stability of sulphur impregnated concrete slabs in acid.

Effect of Sulphur Impregnation on Strength and Modulus

There is a large increase in the strength and modulus of elasticity of sulphur impregnated concrete. This results mainly from the filling of capillary pores in the hydrated cement paste with sulphur. Both the improvement in strength and modulus of elasticity are proportional to the percentage of sulphur loading, see Figure 8.

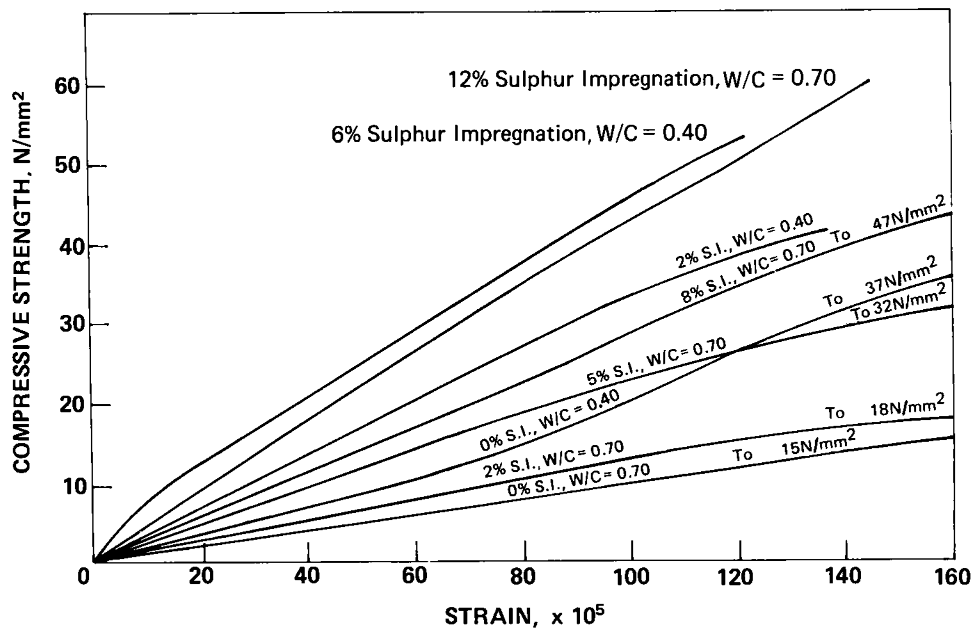

Figure 8 Compressive Stress-Strain Curves for Sulphur Impregnated Concrete

For a water-cement ratio of 0.70, the modulii of elasticity of sulphur impregnated concretes were 43, 29, 21 and 11 kN/mm^2 for 12, 8, 5 and 2 per cent sulphur loading respectively. The modulus of elasticity of the control specimens was 9 kN/mm^2. For a water-cement ratio of 0.40, the modulii of elasticity of the sulphur impregnated concretes were 46 and 35 kN/mm^2 for 6 and 2 per cent sulphur loading respectively. The modulus of elasticity of the control specimens was 17 kN/mm^2.

The test results showed that the flexural strength increased by 75 per cent for the sulphur impregnated concrete with a water-cement ratio of 0.70 and 15 per cent for sulphur impregnated concrete with a water-cement ratio of 0.40.

CONCLUSIONS

The percentage of sulphur loading is a function of the impregnation time. Concrete with a high water-cement ratio shows a higher initial rate of sulphur loading than concrete with a low water-cement ratio.

There is a substantial reduction in water absorption for sulphur treated units as compared to untreated units. The data indicates that low water-cement ratio concretes are more effective in reduction of water absorption than high water-cement ratio concrete for the same percentage of sulphur loading.

Sulphur impregnation of a concrete slab increases its abrasion resistance. The date indicates that the high water-cement ratio concrete exhibits a better improvement than the low water-cement ratio.

Sulphur impregnation of a concrete slab increases its resistance to acid attack. The data indicates that the low water-cement ratio concretes are more effective than the high water-cement ratio.

There is a significant increase in strength and modulus of elasticity for sulphur treated units as compared to untreated units. The increase is a function of the percentage of sulphur loading.

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DISCUSSION

Alan A. Lilley. I wish to ask Mr. Beckett a question, which I suspect Dr. Swamy may also be able to answer. I would firstly like to congratulate Mr. Beckett on his paper in which he outlines briefly and very clearly some of the merits and some of the problems in using steel fibre reinforcement in slabs. As he points out, the basic weakness of all concrete is its low tensile strength compared with its compressive strength. He also points out that the use of fibres is not new. The Romans were using them some years ago in making bricks, and the only new recent use of fibres is, in fact, in asbestos cement, which has proved a very functional material with a good performance and price record. I was very disappointed then to learn from Mr. Beckett's paper just how little benefit is gained, in flexural strength, by the addition of steel fibres to concrete. This small gain implies to me that fibres are not being used very efficiently. On page 3 of his paper he shows that the addition of 5 per cent of fibres, by mass, produces concrete with a flexural strength in the order of 7 N/mm^2. I would suggest that this is only marginally better than the flexural strength of many unreinforced precast concrete products. So I come to my question: does Mr. Beckett agree that the fibres are not being efficiently used and therefore are not being economically used? Would it not be better to direct research to improve the orientation of the fibres, both in their direction and their location within a slab? With this point in mind has any consideration been given to the use of fibrelated steel fibre nets? On page 15 of his paper, Mr. Beckett claims that the thickness of reinforced slabs can be reduced by half compared with plain slabs. I must say that I find this very difficult to understand. All the classic slab design theories, Westergard, Teller and Sutherland, and Kelly, and also practice, suggest that if the slab thickness is reduced, the tensile stress is markedly increased. Halving a slab thickness would in fact put up the tensile stress four-fold. From the results given in Mr. Beckett's paper, I do not think a slab would perform as well as he hopes.

R. Narayan Swamy. Thank you Mr. Lilley. I am quite sure that Mr. Beckett would like to answer the points that you have raised, but maybe I can just put forward a few answers first. Firstly you questioned the fact that fibre reinforcement does not substantially increase the flexural strength; this is probably true. I think that the most that one can expect is an increase of the order of, say, 100 per cent in flexural strength with a fibre volume of, say, 1 to 2 per cent. However, one must not think therefore that the fibres are being used inefficiently. There are two points here. Firstly when you have no fibres you cannot utilize the flexural strength of the concrete at all in design; when you have fibre reinforcement you can use a certain proportion of that flexural strength in design. Secondly, the effect of the fibres is not only just to increase the flexural strength, and in fact I think papers talking about increase in flexural strength because of fibre reinforcement are really misleading. The main purpose of fibre reinforcement is to prevent the propagation of cracks, both in length and in width. It controls the cracking and the crack width and therefore the tensile stiffening effect of the tensile concrete, in a beam or a slab, is substantially increased. We have made extensive calculations on the cracked EI value of beams and slabs and you will find that there is a substantial increase in the cracked EI value of a loaded structural member. So I think that if you just look at flexural strength alone it could be misleading.

To come now to the point raised about alignment and orientation of fibres to resist the stresses in the most obvious direction, which is certainly a valid idea, but remember we cannot always utilize this. Very few structural members are subjected to uniaxial stresses. In a slab, therefore, there is the possibility that one could orientate the fibres in the tension zone, but in areas where multiaxial stresses act, orientated fibres may not be fully effective. Quite a lot of work has been done in Sweden in orienting fibres and I think perhaps Dr. Petersons might care to comment. I know Dr. Skarendal of the Swedish Cement and Concrete Research Institute has done quite a lot of work in orientating the fibres and there is quite a lot of published data available; for example, see the proceedings of the symposia held in London* in 1975 and in Sheffield† in 1978. The point that I want to make is this: in a structural member we have bi-axial and tri-axial stresses and not just pure flexural stresses arising from constant moment, and fibre orientation probably is not the best or most effective means of counteracting these stresses. You made the point about fibrelated fibre nets being more effective. Well there have been several attempts at producing not only fibres of different kinds: for example, in Australia they have fibres with enlarged ends. We have the paddle fibres which are produced in the Netherlands, we have a Japanese fibre which is again enlarged at the ends, hooked fibres from Belgium, crimped fibres and other fibres are also being developed. Fibre nets are also not new, and their action may be very similar to ferrocement or members with mesh reinforcement. On the question of slab thickness which you raised, I must confess that I am also rather surprised at the comment made. The fibres cannot be utilized to carry heavy loads, the maximum increase in the moment capacity due to fibres is only of the order of 10 to 20 per cent; so I do not think one can use fibre reinforcement to reduce substantially the slab thickness in suspended slabs. The way to tackle this is to reduce the slab thickness where punching shear comes in and one can also reduce the amount of other reinforcement that one puts in. However, on the other hand, it may be possible to reduce the thickness of slabs on grade when fibre reinforcement is used compared to unreinforced slabs, but here slab thickness is not decided by flexural strength alone. One may have to consider fatigue resistance or impact resistance for example.

Richard E. Beckett. In making the proposal that the concrete slab thickness be reduced, it is not intended that the overall depth of construction above the sub-grade be reduced, i.e. any reduction in concrete thickness is compensated for by increasing the thickness of hardcore. Concentrated loads

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* Proceedings, RILEM Symposium on Fibre Reinforced Cement and Concrete, London, September 1975, The Construction Press Ltd., Hornby, Lancs., 1975, pp 459.

† Proceedings, RILEM Symposium on Testing and Test Methods of Fibre Cement Composites, Sheffield, April 1978, The Construction Press Ltd., Hornby, Lancs., 1978, pp 545.

applied at the surface of the concrete will therefore be distributed approximately over the same ground area at the formation level. The effect of using a predictable high strength thin slab has in practice already been demonstrated in the field. Results giving satisfactory performance over a period of between 10 to 15 years are not uncommon. A reinforcing system which operates from the surface has the advantage of enhancing other characteristics of the concrete. This is referred to by Dr. Swamy, and requires the designer to relate slab thicknesses to factors other than just flexural strength.

Barry P. Hughes. Gentlemen, may I confine myself to only one contribution, but for separate papers! First the keynote paper: Mr. Hobbs had the very unenviable task of crystal ball gazing for part of his paper. I would like to comment on one aspect which I think can be misunderstood and could lead to problems in the future. Although we can design concrete not to crack, I think in many cases it is simpler to accept the fact that concrete is going to crack and design accordingly. If, for example, appearance is important, then we should design for crack widths which are acceptable for appearance, which generally means crack widths which are considerably finer than those which normally we have had in the past. Again I think that concrete is often blamed for material deficiencies when in fact it is detailing of structures which can be at fault and in this respect I am thinking especially of the interface between architect and engineer. For centuries facades of buildings have been designed with lintels under windows, and so on, to throw water off the face of the building. Furthermore brickwork has the advantage that its small units, with the joints, can tolerate significant staining due to weathering. With concrete we expect large facades to weather uniformly when water is not thrown off the face and when the weathering characteristics of the concrete are not improved with, for example, air entrainment. So whilst I accept everything that Mr. Hobbs said, I would add these comments on aspects where I think that concrete can be blamed unjustifiably.

Dr. Swamy stated for his paper that the same mix design can be used with and without fibres. I think that that can be misinterpreted because if fibres are added to concrete, it is well known that the workability is greatly reduced. Would Dr. Swamy elaborate on that particular comment? Mr. Liljestrom showed us a number of slides referring to type-k cement. Now, shrinkage as far as I am concerned is much less of a practical problem than early thermal contraction. The drying shrinkage strains are generally much less than the early thermal contraction strains, and he did not appear to refer in any of his slides to the early thermal contraction strains that can be anticipated with type-k cement. Certainly in our experience long-term drying shrinkage strains under exposed site conditions are almost negligible. When he showed the continuous casting of slabs, I was pleased to see that generally he was using square mesh reinforcement and advocating that it is placed near the top surface of the slab. I would merely comment, therefore, that while that was doing a good job, the main problem that it was solving was early thermal contraction rather than long term drying shrinkage. Finally I would just like to ask one quick question on Professor Yuan's paper, on sulphur impregnated concrete. Sulphur, of course, is much cheaper than the normal polymers used for impregnation. Does he foresee any possibility of this being used for in-situ impregnation of slabs to achieve some surface impregnation with sulphur? Is this a practical proposition in the fairly near future?

William P. Liljestrom. I really did not have enough time, Professor Hughes, to properly define the points that you have raised. Today, when we are designing slabs for large industrial warehouses, where joints are a problem due to the wheel loadings imposed, we definitely insist that the walls and roof are in place before we move in and place these large slabs, which are dimensioned at, say, 120 ft by 80 ft with no sawcuts whatsoever. We do protect those slabs from thermal contraction. Even in areas like Reno, Nevada, where you could have a difference of 50°F from the morning temperature to the afternoon temperature, within the warehouse we have kept the temperature in a range of, say, ± 10°F. I do not know whether I have completely answered your question or not, but I would emphasise that we are very sensitive to the problem of thermal contraction. That is why we will not take on large projects in areas where there is low humidity and high winds unless we can have the walls and roof up on a warehouse first, because we do subject ourselves to plastic shrinkage cracking more easily with type-k cement than with its comparable Portland cement.

Chairman. Yes, I would very much agree with that comment. Observation of industrial floor construction shows that with the sort of slab thicknesses that we are talking about in the average industrial floor, temperature effects are not as great as they are externally, although the need to put a roof over the floor before it is constructed is most important.

R.L. Yuan. The question you ask, Professor Hughes, is a good question, because if you do research the objective is how to apply it to the field. To answer your question, yes, some work has been done to develop the procedure to spray sulphur on the surface of vertical walls and horizontal pavements, but as far as the highway pavement or bridge deck is concrened, the in-situ impregnation of slabs encounters certain obstacles. It is difficult to get deep impregnation because of temperature problems. You have to have the temperature in a particular range in order to obtain this impregnation of concrete. The first thing you have to do, therefore, is to dry out the surface of the bridge deck or pavement, then heat it up to the required temperature, if possible, and then impregnate the concrete.

R. Narayan Swamy. I would like to make one point: we actually use a water reducing agent to increase the workability of concrete. With this agent we get a slump of between 100 and 125 mm for plain concrete and there may be some problems with bleeding of these slabs. With concrete with fibres the slump was between 40 to 50 mm. The water reducing agent has been used in all slabs, in general at 1.6 per cent of the cement mass.

Barry P. Hughes. The point I was making was not about daily or monthly temperature variations but the early heat of hydration, an effect which also occurs with all slabs. Assuming that it is placed under good conditions and there is no question of plastic drying shrinkage, in which case the early thermal contraction strain, due to the heat of hydration, is of the order of three times the long term shrinkage strain. When I saw some of the slides it appeared to me that what was being shown in one case, the sudden change in strain, could have been an early thermal effect rather than actually a drying shrinkage effect. Long-term drying shrinkage can be only of the order of 50 microstrain compared with an early thermal contraction strain for the first few days of 100 to 200 microstrain.

Peter C. Hewlett. I would like to make one point just to amplify something which was mentioned in Dr. Peterson's paper concerning high flow, high workability concrete, that I think you might find of interest. We have been doing some work, which is still current, endeavouring to make a very fluid grout for the purpose of making in-situ concrete by grouting very large aggregate that was preplaced. I am not at liberty to disclose the reason we are interested in that, but nevertheless I think you might be interested in one of the trends. Figure 1 shows the workability loss of a superplasticized grout with time, and the steep line, A, relates to Colcrete trough measurement of flow. As the Colcrete flow value decreases so the workability decreases and you can see that within 60 minutes or so, at ambient temperature, the workability that has initially been given to the grout is fairly rapidly lost. That is a trend that is current in the use of these materials for normal concreting when obtaining high flow concrete. There is a high workability period that has to be utilized properly. We were concerned with endeavouring to extend that period, not by a few minutes but by several hours at ambient temperature, by careful selection of a particular type of chemical retarder. I am emphasizing that point, because we have found that, for reasons we are not at all clear about, the extended workability trend is not common to all retarder/superplasticizer combinations. If you make a combination of a particular type of retarder and a superplasticizer then line A can be replaced by curve B, and the high workability period is now extended from some 60 minutes to something approaching 6 hours. The two lower curves (C and D) relate to the yield value and the plastic viscosity of the same grout and the loss at point E is followed by a steep rise in yield value and plastic viscosity as you would expect. It's my view that this general trend of workability extension can be extended to concreting practice, although this data was obtained on a grout.

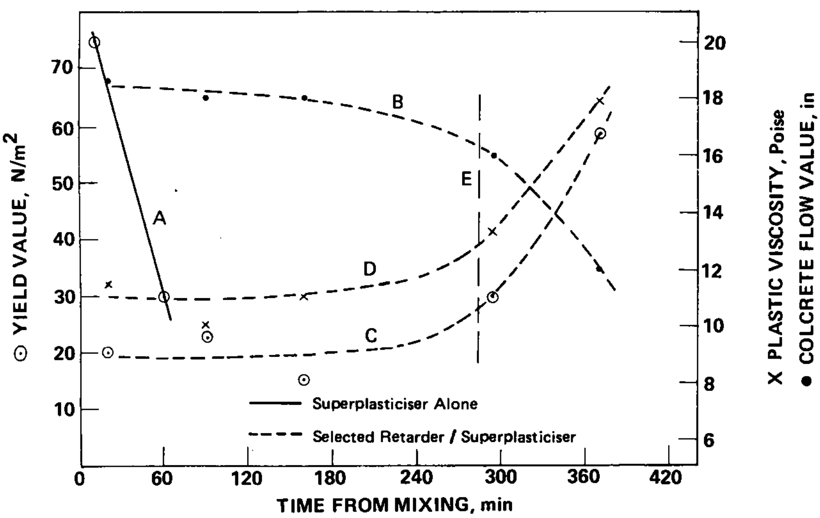

Figure 1 Retained Workability using Retarded Superplasticizer in Cement Grout

Finally, I have two questions, one directed to Mr. Liljestrom and one to Dr. Yuan. Firstly, could Mr. Liljestrom give some comparative cost indicators of the benefits or otherwise of using a type-k cement as opposed to other ways and means of obtaining low water-cement ratio concretes. Secondly, before we get too enthusiastic about the possibilities of a cheap polymer such as sulphur, would Dr. Yuan like to comment on the consequences of fire damage?

William P. Liljestrom. We are very sensitive about the cost of this cement since we are a licensor. Generally, concrete made with this cement will carry a premium of from $5 to $7 per cubic yard over comparable type 1 or type 2 Portland cement concrete. Thus a cubic yard of this type of concrete in, say, the Southern Californian area would be priced at around $32, whereas a cubic yard of normal Portland cement concrete would be priced at around $25 to $27.

R.L. Yuan. We are not very enthusiastic about selling the sulphur impregnated concrete, we just do the research. But to answer your question, we do realise that the fire hazard is a problem for sulphur impregnated concrete. However, in common applications, the sulphur is applied to external structures such as pavements, highway bridge decks and parking garage decks where the fire hazard is not really a problem. We have also done some study on sulphur impregnated concrete blocks and bricks, and sewage pipes, which is another good application because in the ground fire is no problem. We have also found that sulphur impregnation of sewage pipes can not only increase their life but also impart a high resistance to certain types of acid effect.

John D. Peacock. There has been quite a lot said about the weathering of concrete and how in many cases it has not been terribly good. I do not personally agree with this view; I think there are some very good examples of exposed aggregate concrete that weather quite well. In fact you can see some of them close to this building. However, I have not got an answer yet to the question of how steel fibre reinforced concrete weathers. Now Mr. Beckett says at the end of his paper that the internal fibres do not corrode, but surely the external fibres do corrode. Can we be sure that we are not going to get a progressive collapse of the external surface of fibre-reinforced concrete or can you tell me if it just ends up looking like a Corten steel structure?

R. Narayan Swamy. Just to correct the final point, Corten structures do look attractive provided that they are correctly made so I think it is not quite true to say that all Corten structures are unattractive. However, to be fair it is a very valid point, to what extent corrosion of the steel fibres endangers the stability of the structure or of the external surface is indeed very important, although I do not think we are talking in terms of progressive collapse whether of the structure or of the facade. I think you have to be very careful what terms are used because I do not think the load is being carried by the fibres totally, thus there is no question of progressive collapse at all, so let us get that out of the way, although there is the possibility of steel fibre corrosion. How much and how to deal with it is very difficult to say. One way is as was done in one tunnel where the steel fibre concrete was sprayed on the underside of the archway and subsequently a covering layer was done without the fibre, so you get a thin layer of mortar to cover the steel fibre; we are engineers and there are very many ways of tackling this problem. Now the other point, in internal structures the corrosion is very, very negligible, the crack widths are substantially less than 0.1 mm at the same load or at an increased load, so in internal structures again there is no possibility of corrosion. For external structures, the problem needs careful consideration. We have got about half a dozen beams which were loaded and cracked and which have been exposed to the outside atmosphere for the last six years. I will have the answer, I hope, within the next few months to see to what extent corrosion has been penetrating into the members.

Richard E. Beckett. Incorporation of steel fibres which are of small diameter, between 0.25 mm and 0.65 mm, at a volume concentration of between 1 to 2 per cent will result in a surface exposure of very few fibres, which will never produce the overall appearance of Corten steel. Within the concrete, which is generally cement rich, the alkaline environment protects the steel and long-term studies carried out in various environments, and by different researchers, have demonstrated quite conclusively that rust penetration ceases below the depth to which carbonation of the concrete has taken place, this has not resulted in an increase in the depth of rust penetration. Aesthetically, there may be locations where the exposure of any steel would not be tolerated. However, in the application being considered by the conference, that of ground slabs, any exposed wire will in fact be worn away and it will therefore be very difficult to differentiate between a conventional concrete slab and one containing steel fibre when viewed at, say, normal walking height above the surface of the concrete.

George Barnbrook. I would ask a question of Mr. Liljestrom concerning expansive cements and the subject of expansion joints in slabs on the ground. Could he say whether compared with normal ordinary Portland cements, there would be an additional requirement for expansion joints in slabs on the ground? We try to persuade people to avoid their use, as they are very often unnecessary in many cases with ordinary Portland cement concretes. What would be the case with the expansive cement concretes?

William P. Liljestrom. The normal procedure that we use today is quite simple. Say, for example, we were starting in a corner of a warehouse, we will put a compression strip along the footing section, maybe 15 mm of felt material, which will take the compression caused when the first slab section begins to move due to its controlled expansion. We tell the contractor that as soon as the structural engineer feels that the forms can be relieved, we want at least two sides of the slab left open for three days so that movement can occur on the subgrade. After that period we pour right up against the key-ways formed in the first section, with a bond breaker. We have found that in this way the compression remains for a good number of years in those slabs. A very good example is given in the March issue of Concrete International, which is a new publication from the American Concrete Institute. This issue includes a very detailed structural engineering evaluation of a 2 million square foot warehouse, four of which have been constructed by the same company. It might be an item that you would like to review.

Adrian E. Long. I would like to ask Dr. Swamy a question with regard to the shear capacity at columns. A few years ago in Belfast we did some work on fibre reinforcement in round columns. Largely because we were doing it on models we did not have success so far as compaction of the fibre reinforced concrete was concerned and we got very, very modest increases in strength because of this. I would like to ask a practical question which is how did you go about vibrating the concrete in and around the column/slab junction so that the fibres did not segregate down to the bottom of the slab? My second question is basically economic: is there an advantage in using fibres when you could use a comparable amount of reinforcing steel placed in the normal way? If you increase your percentage steel from about ½ per cent, which you have used, up to 1 per cent, you probably get 100 per cent increase in strength as opposed to the 50 per cent increase in strength that you have found in your test; you would also probably get improved serviceability.

R. Narayan Swamy. I think that compaction has never been a serious problem. Obviously in the laboratory we have tried both internal vibration and table vibration. I think our general experience is that probably table vibration gives you a much better compaction and uniform distribution of fibres. We have looked at the fibre distribution by sawing through the specimen after compacting by both internal and external vibration and if the mix is designed properly, if you accept that fibres behave as aggregate and therefore design your mix accordingly, there should be no problem with compaction. One of the reasons that we use fly ash is because it increases the workability, it increases the cohesiveness of the matrix, it decreases the frictional characteristics between the fibres, and so on. So basically workability is not a problem.

The second point that you raise is very valuable indeed. I think if we replaced the fibres by an equivalent amount of steel reinforcement, then yes, there will be substantial increase in strength. I think the point that I want to emphasise, however, is that you cannot avoid punching shear failure unless either you increase the thickness of your slab or provide shear reinforcement. Now what we have found is that with the addition of fibres you can reduce the total amount of tension and compression steel, i.e. longitudinal steel, by about 25 per cent and, for the same load capacity, change the mode of failure from shear to flexure.

What I am really trying to say is that you can change the mode of failure from a sudden brittle type of failure into a progressive ductile type of failure. You rearrange your tension and compression steel and for the same punching shear load as will be taken by the reinforced concrete slab without fibres, you can achieve the same load capacity in flexure with a reduction in the tension steel.

John F. Dixon. I would like to keep Dr. Swamy busy and ask a question about high early strength concrete, and the values of strength he referred to. First of all the cement that was used, could you tell us a little more about it and do you know when, if ever, it will be commercially available? Secondly, the rather interesting 12 and 24 hour strengths you showed, I assume they were achieved in a rather cosy laboratory environment at 20°C. Since we have just experienced in these islands for five months temperatures which seem disinclined to exceed 5°C, could you please comment on the possible performance of that concrete at lower temperatures?

R. Narayan Swamy. The high early strength cement we referred to is not commercially manufactured at this moment. I understand that the decision as to what price it should be allowed to sell for would decide whether the cement is going to be manufactured or not. All I could say is it has got a C_3S content of 74 per cent, C_2S round about 6 or 7 per cent, C_3A round about 4 or 5 per cent and C_4AF round about 2 to 3 per cent, roughly. It has an SO_3 content which is almost right up to the limit so far as B.S. 12 is concerned. Its specific surface is 4,500 cm^2/g, it has got better setting characteristics, both initial and final time of setting, than rapid hardening Portland cement and at 24 hours it will have double the strength of rapid hardening Portland cement, but it is not commerically available as yet. As to your second point, I am sorry but I could not really tell you anything because we have only got a very small amount of this cement. All our tests were carried out in an internal uncontrolled environment so that we did not give it any special treatment at all, but there are some specimens which are exposed to outside elements which we have not tested yet, so perhaps in the next few months I may be able to give you some answer to that question.

Session 2

Structural Design

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STRUCTURAL DESIGN

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INTRODUCTION

In this, the first of two sessions on Structural Design, I propose to concentrate upon the development of the methods of analysis currently available to designers and their validation by laboratory and other tests.

For the purpose of this paper I have divided the analytical methods in terms of the basic relationship between load and deformation: (a) elastic, (b) plastic, and (c) non-linear.

The main methods are summarised in Table 1, with a corresponding selection of key references (which are by no means exhaustive); the distribution of relevant papers in the two Sessions on Structural Design is shown separately in Table 2.

Table 1 Main Methods of Slab Analysis

BASIC LOAD-DEFORMATION CHARACTERISTIC	ANALYTICAL METHOD	KEY REFERENCES
Elastic	Plate Theory	Timoshenko, 1959
	Finite Differences	Schleicher and Wegener, 1968
	Finite Elements	Zienkiewicz, 1967
	Grillage	West, 1973
	Experimental	Rüsch and Hergenröder, 1961
Plastic	Yield Lines	Johansen, 1968
	Strip	Hillerborg, 1975
	Optimal	Rozvany, 1966; Morley, 1968
Non-linear	Finite Elements	Jofriet and McNeice, 1971; Hand et al, 1973
	Simplified Equations	Rao and Subrahmanyam, 1973

Table 2 Conference Papers on Slab Analysis

BASIC LOAD-DEFORMATION CHARACTERISTIC	ANALYTICAL METHOD	PAPERS IN SESSION 2 (and 3)
Elastic	Plate Theory Finite Differences Finite Elements Grillage	
Plastic	Yield Lines Hillerborg Strip Optimal	Goli and Gesund; Hindson and Chronowicz Kemp and Fernando; Attard and Base Melchers and Lowe
Non-linear	Finite Elements Simplified Equations	Jain et al; Sinisalo et al Clark and Cranston; Desayi and Muthu

ELASTIC METHODS OF ANALYSIS

If the behaviour of a concrete slab under load is considered as linear elastic, it may be treated as an elastic plate. The flexural and shear stresses and deflections may be found by solution of the fourth order differential equation relating load to deflection for thin plates with small displacements. Classical plate solutions using series techniques are confined to the simplest of plate plan forms and boundary conditions, see Timoshenko and Woinowsky-Krieger (1), Rowe (2), and, although the advent of the computer has extended these methods, Cusens and Pama (3), they remain essentially limited to rectangular and circular plan forms.

Finite difference solutions of the fourth order differential equation have also been proposed for simple boundary conditions, e.g. Marcus (4) and Robinson (5), and used to produce tabulated data, Schleicher and Wegener (6), Javor (7). Wood (8) has extended the method to cases of edges supported on beams. The further development of the finite difference method using the technique of dynamic relaxation has led to several applications to slabs and plates e.g. Cassett et al (9).

Engineers have long sought the simplification of two and three-dimensional problems by dividing them into discrete pieces. In the analysis of bridge decks Lazarides (10) in 1952 was one of the first to divide the structure into a grid of individual longitudinal and transverse beams; he set up and solved a set of simultaneous equations from equations of compatibility of deflection and slope at grid points. With the advent of electronic computation this technique became more practicable and was developed as the grillage method by Lightfoot and Sawko (11) and by West (12). It is a method of elastic design which is economical and popular with structural engineers for both slab and beam-and-slab decks.

The other discrete technique of analysis for slabs is the finite element method which involves the division of the slab into a mesh of triangles or quadrilaterals. Displacement functions of the intersecting mesh points (nodes) are usually characterised by polynomial functions. Stiffness matrices have been developed for common shapes of element and with knowledge of the nodal forces it is possible for the computer to solve the displacements of the assemblage of elements which make up the structure. Zienkiewicz (13) describes the method and its applications. Numerous computer programs are generally available for elastic analyses of slabs e.g. PSALM in the Department of Transport STRAND 2 suite (14). This program employs triangular elements which assume a linear variation of moment within the element giving good accuracy for both shear forces and bending moments.

The STRAND suite also incorporates a program OPUT 2 for calculating the moments of resistance required in reinforcement that is placed in any two directions in an elastic moment field. This is based on the Wood-Armer plastic equations (15), but is generally applicable to any of the elastic methods of analysis.

PLASTIC METHODS OF ANALYSIS

The yield line analysis of concrete slabs developed by Johansen (16) is a method of determining the limit state of collapse by considering the yield lines or hinges which occur in the slab as a mechanism forms. Elastic displacements are neglected. The method is an upper bound approach and thus will give optimistic estimates of the collapse load unless the correct mechanism is found. Experimental evidence (8) has shown that the method gives a safe estimate of the collapse load; because it neglects the membrane action which occurs at large inelastic deformations the experimental values can be much higher than calculated values. However, Clark (17) in tests on skew slabs has shown that where design of reinforcement is based on yield-line theory the limit state of serviceability (i.e. cracking) is unlikely to be satisfied.

The strip method attracted little attention when first presented by Hillerborg in the late 1950's. The publication in 1969 by Wood and Armer (18) of a paper, which critically examined the original theory and suggested some modifications, was followed by others. Hillerborg's authoritative book (19) was published in 1975. The principal aim of Hillerborg's method was to formulate a safe lower bound solution and the basic approach neglects slab twisting moments and distributes the load in two orthogonal directions; the problem is so reduced to the simple analysis of beam strips. Recent generalisation of the method by Kemp and Fernando, which more readily enables the consideration of concentrated forces, is described in their paper in Session 3 of the conference. In their conclusions they suggest that a torsion-free grillage program would be a convenient procedure; Gurley (20) has recently put forward such a procedure employing a bi-moment approach.

There has been considerable research into optimal solutions. Rozvany's method (21) for the minimization of reinforcement is based on a lower-bound plastic solution assuming zero torsional moments and no membrane action. The equilibrium equation has an infinite number of solutions which are optimised by considering that all portions of the slab fail simultaneously. Shear stresses are distributed by concentrated bands of reinforcement. Morley (22) has developed an optimum solution on a similar basis but shear stresses are distributed by negative reinforcement. Lowe and Melchers (23) have investigated the range of optimal solutions to slabs supported by edge beams.

The vigour apparent in the discussion of papers on optimal solutions does not seem to have spread to design offices as there is no evidence of practical adoption of the complex reinforcement patterns which result.

NON-LINEAR METHODS

In the finite element method, the stress-strain characteristics of individual elements may also be non-linear, making the method completely general in application. However, the application of the method to simulate the true load-deformation characteristics of a reinforced concrete section is a complex process, expensive enough in terms of computer time to make it too extragavant for other than research use. Jofriet and McNiece (24) used rectangular elements with empirical moment-curvature relationships based on the well-known equations of Branson and of Beeby. Hand et al (25) used a layer technique assuming the reinforcing steel to be elastic-plastic and the concrete to possess a limited linear tensile stress-strain curve and a tri-linear compressive stress-strain curve. Schnobrich (26) has reviewed work in this field and indicated research needs. A multiplicity of empirical relationships has been advanced to represent the stress-strain properties of concrete in compression and the moment-rotation (M-ϕ) curves for reinforced concrete. Bi-linear and tri-linear curves have been suggested and Rao and Subrahmanyam (28) presented an improved tri-segmental M-ϕ relationship for beams and slabs which included the contribution of concrete in tension. In their paper in this session Clark and Cranston have further developed this work.

EXPERIMENTAL EVIDENCE

The first paper in this session by Lowe and Melchers examines some of the test evidence available on slabs but their primary emphasis is on anisotropic arrangements of reinforcement. Early experimental work (prior to 1961) on the collapse loads of rectangular slabs is summarised by Wood (8); this work includes some tests of a full-sized building in South Africa by Ockleston (29). In general, however, recent work has been on laboratory scale models and many of the better-known results are quoted in the papers of this Session. The common finding has been that yield line theory provides a conservative estimate of the limit state of collapse for slabs and orthotropic or isotropic arrangements of reinforcement. The load factor is frequently enhanced by membrane action in the slabs at the post-cracking stage. However, the findings of Clark (17) of cracking problems in slabs designed by yield lines must be re-emphasised.

In a Conference paper, which is not to be presented by the authors, Attard and Base use the Hillerborg strip method for a simple form of optimal design within the provisions of the Australian Code of Practice and give test results showing satisfactory limit states of serviceability and of collapse. However, yield-line theory overestimates the collapse load of at least one of their three test slabs. Earlier tests by Armer on half-scale slabs designed by the Hillerborg method also found satisfactory results for both limit states.

The use of elastic moment fields remains a viable basis for design and Clark (31) has found that slabs so designated behave satisfactorily in terms of both serviceability and collapse. It is of interest to note that writers on the Hillerborg strip method urge the necessity of understanding the general form of the elastic moment field. Thus the original approaches to design of concrete slabs have not been superseded by recent advances in plastic methods of analysis.

CONCLUSIONS

In brief, my views on the various methods discussed may be summarised as follows:

The determination of the elastic moment field is useful, primarily as a computer-based technique, and may conveniently be linked with the Wood-Armer plastic equations for design of reinforcement.

The Hillerborg strip method is gaining acceptance and, if used with care, is a good, simple, overall design technique.

The yield line method is a reliable technique for checking the limit state of collapse.

Non-linear finite element methods are convenient in research computation.

Non-linear empirical equations are useful for checking deflections and the limit state of serviceability.

Experimental work on slabs has not been extensive and more research is needed.

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EXPERIMENTAL BEHAVIOUR OF REINFORCED CONCRETE SLABS, WITH PARTICULAR REFERENCE TO ANISOTROPIC SLABS

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ABSTRACT A number of alternatives are available for the design of slab structures. The resulting structures have a greater or lesser degree of anisotropy of reinforcement, which in some cases may be quite extreme. The available experimental evidence is reviewed in order to clarify the role of such anisotropy.

INTRODUCTION

Interest in the theory of design of reinforced concrete slabs extends at least from the beginning of this century to the present day. Early efforts to rationalize experimentally observed behaviour against theory rested naturally on linear elastic fundamentals, with only modest success (7). Renewed interest in slab problems became apparent in the 1940's. It seems clear that the search for what came to be called yield line theory (11) began prewar and was vitally influenced by the classical results of plasticity theory. In some sense it may be argued that the dominance of isotropic reinforcement layouts in yield line theory springs, in part, from the influence of classical plasticity theory. In part too, the isotropy results from the assumed idealized rigid-perfectly plastic behaviour of slabs and the resulting pyramid type yield patterns separating rigid elements. For these, banded or anisotropic reinforcement does not produce a more favourable estimate of load carrying capacity. Of course, isotropic reinforcement obviously provides benefits during construction and this must have had considerable bearing on the apparent attractiveness of yield line theory.

Despite the greater rationality of the yield method for slab design (in terms of relating design approach to observed behaviour), it has not apparently found favour with practising designers. In practical application yield line theory offered little new; it requires tabulated or handbook solutions to all but the simplest cases, and leaves an uncertainty on the bounds on load carrying capacity. Being a method of analysis, it is unable to generate design decisions. Theoretically, too, it presented difficulties. There is uncertainty as to whether it should lie within the theory of limit analysis, and even if this is accepted, complete moment fields cannot be found for the bulk of practical cases, nor do orthotropically reinforced slabs necessarily lie within the strict confines of limit analysis.

These difficulties were largely responsible (10) for the development of the strip method of slab design (rather than analysis). However, strip methods, because they concentrate attention on exploring certain types of equilibrium moment fields, suffer from the apparently undesirable feature of continuously variable moment fields. Once a complete equilibrium moment field is generated by the strip method then it can be argued that only the reinforcement needed to cope with this moment field need be provided and as a result complex patterns of reinforcement, in non-isotropic arrays, seem to be inevitable. However, it should be observed that the same situation exists in the conventional elastic based design methods and in yield line theory. The only practical simplicity which these methods have is that endowed on them by custom, namely that isotropic or orthotropic reinforcement systems are generally used. Thus, in both cases, the intractability of moment-field analysis (and practical considerations) has led to the moment variation within the slab being ignored when detailing reinforcement.

In its simplest (and most useful) form, the strip method leaves the designer with considerable freedom to decide how the slab shall support the applied load. While in practical situations there may be restraints imposed by Codes of Practice for reason of serviceability, in theory this freedom can be utilized to allow the imposition of other objectives, such as material minimization. This led originally to attempts to optimize lower bound designs, but has now become a separate approach to slab design.

The optimal design formulation of particular interest here is that which seeks to minimize the amount of reinforcement required for the slab to support a given loading. In practical applications of these results such designs are not fully realizable (15), though many of the optimal features can be incorporated into real slabs.

In contrast to yield line analysis where few fully rigorous boundary value problems have been solved, in optimal design a comparatively wide range of boundary value problems have been solved, including such problems as edge beam supported slabs (12). This is not to say, however, that some interesting unsolved problems do not remain.

Below is given a brief review of slab design criteria and their relation to the four design procedures: Elastic Moments, Yield Line, Hillerborg Strip and Optimal Design. An examination of relevant experimental studies follows. Finally, conclusions are drawn regarding the feasibility, safety and utility of the various approaches, but with particular emphasis on the anisotropy which seems to be an inherent common feature of the recent theoretical developments.

SLAB DESIGN CRITERIA

In the context of limit state design, the two main criteria for design are ultimate strength and serviceability. The latter may, as a first approximation, be regarded as related to the stiffness of a slab. Slab stiffness against load can be derived easily from experimental work. Comparison of crack patterns, crack distribution and crack size is much less quantifiable and hence rather more subjective in interpretation, although crack patterns, occurrence of the first crack and crack widths can all be determined experimentally and compared.

The slabs with which we are concerned are all uniform thickness and have a relatively low percentage of reinforcement. For such slabs it is generally expected that prior to initial cracking, the steel reinforcement present has little effect on the slab stiffness or cracking load. Only subsequent to cracking does the steel reinforcement layout begin to influence the stiffness and deflection pattern. Hence, in a sense, a serviceability requirement on cracking will be independent of steel layout and will depend only on slab thickness and the concrete properties. As will be seen, the slabs tested to date largely bear out this argument, at least at low loads.

Of the methods of slab design outlined in the introduction, the yield line method (11) and (Hillerborg's) strip method (10) clearly emphasize the criterion of strength. The elastic moment field method (23), while primarily strength oriented, does offer some scope for stiffness specification. In principle, the optimum method (12, 20) also offers scope to specify both strength and stiffness, in the sense that minimum weight systems are also maximum stiffness systems. Each of these methods of slab design are considered below in relation to the criteria of strength and serviceability.

Yield Line Theory

As indicated earlier, in some respects the initial promise of yield line theory has not been fulfilled. Consideration of collapse modes, particularly, has not proceeded much beyond the rudimentary pyramid-shaped surfaces, surfaces which are a valid approximation to behaviour at collapse, but which do not necessarily reflect behaviour at service load. For serviceability, the yield line method must therefore, rely almost exclusively on the relative independence of service load behaviour compared to reinforcement layout. In this case, deformation of the slab provides only a qualitative guide to serviceability requirements.

Strip Method

At the risk of over-simplifying the situation, it can be said that the excellent developments associated with the strip method of design (10) have provided a much-needed alternative and have stimulated renewed interest in the whole field of slab study. Yet the strip method leaves something to be desired on two counts. First, the largely arbitrary decisions which need to be made to determine load distribution and second, the lack of specific regard for the deflected shape of the slab. Questions of load distribution rely upon an independent appreciation of the problem for successful resolution. Disregard of the deflected shape may particularly affect the serviceability of the resulting structure.

The concern for serviceability has led to the suggestion of a "generalized strip deflection" method in which the load distribution between orthogonal directions (x, y) is determined on the basis of matching the elastic deformations of orthogonal strips (8), it being assumed that reference (even approximately) to an elastic plate solution will be beneficial in ensuring adequate serviceability. The suggested approach appears unnecessarily restrictive in terms of slab shapes that can be considered and requires considerable extra computation. As a result the essential simplicity of the strip method, as well as its wide applicability, is under threat.

Elastic Solutions

Underlying much thinking about serviceability is the concept that the theory of elasticity somehow predicts this stage fairly well. Perhaps a better statement is simply to note that the theory of elasticity is the best indicator we have of service load behaviour, recognizing that the effects of reinforcement arrangement, prior loading, internal cracking, creep and shrinkage, etc. are all ignored. Within these limitations, the elastic procedures do at least explicitly consider the deflected shape of the slab.

Optimal Design

Optimal design of slabs based on an optimality criterion (12) requires the deflection surface of the slab to be considered. While this deflected shape does not necessarily represent the slab shape at collapse, it does represent the deflected shape that would be expected at lower loads, all reinforcement being strained equally (ignoring the tensile strength of concrete). Hence, it is apparent that both elastic and optimal procedures consider the deflected shape in an intimate manner. Neither solution can be completed until the deflected shape has been established and in each case, smooth deflected shapes are in principle required. In this and other respects the elastic and optimal methods have features in common.

From a practical point of view, iso- or ortho-tropic rectangular reinforcement layouts are, of course, strongly preferred, if only for the need to save labour in reinforcement placing. It has been argued here that this preference does not stem from any quality inherent in any of the design approaches considered above. In theory, the elastic, strip and optimum designs all require highly complex anisotropic reinforcement layouts to satisfy the calculated state of internal stress. Even in the yield line method, the use of iso- or ortho-tropic reinforcement is not inherent in the method (which does not, however, readily permit the calculation of internal actions), but merely a device for making it tractable in practical calculation.

It follows, therefore, that given sufficient repetition in construction, or incentive from enhanced properties, anisotropic arrays of reinforcement might be justified in at least some classes of slab construction, irrespective of the design method used.

It is further worth noting that virtually all real slabs, especially when considered in conjunction with associated edge beams, are essentially anisotropic structural elements. Viewed in this light there is a case for considering the role of anisotropy in the behaviour of actual structures in general, and slabs in particular. The rest of this paper considers the experimental behaviour of slabs reinforced in an anisotropic manner.

EXPERIMENTAL INVESTIGATIONS

Perhaps the most surprising aspect of the experimental investigation of slabs is the relative neglect of this field as compared with theoretical studies. The list of references is short since in only few studies can confident statements be made about the role of anisotropy.

The earliest comparative studies were those of Taylor *et al.* (22, 9). The studies pre-date the more important findings of optimal theory, and were made specifically to test the use of banded reinforcement, such as might result from the application of strip theory. Two series of ten slabs, each 1.83 m square and with various thicknesses, reinforcement arrangements and edge support conditions, were tested. The first series (22) showed that banded (anisotropic) reinforcement arrangements produced somewhat stiffer slabs at loads below ultimate than did equivalent isotropic reinforcement arrangements. However, the results are not directly comparable in terms of efficiency compared to the degree of anisotropy, since the criterion for design was that of equal ultimate loads and as a result unequal weights of reinforcement were used in the various slabs.

All slabs exceeded the design ultimate load. In the corners of slabs with a high degree of anisotropy rather greater cracking at working loads than in slabs with isotropic reinforcement was observed, although the crack widths were within allowable limits.

Sharpe and Clyde (21) reported the results of tests on rectangular (2.13 m x 1.52 m) slabs, some designed by the strip method and others designed using the yield line method.

Various boundary conditions were considered and although details of reinforcement and details of cracking behaviour are scant, it is evident that the slabs all behaved satisfactorily; all exceeding the design ultimate load and having adequate deflection stiffness. It was stated that the slabs designed by the strip method tended to be stiffer than the yield line slabs in the working load range. In the post yield range no significant difference in behaviour was found.

Another series of seven slab tests is reported by Armer (2). The purpose of these tests was to explore applications of the strip method, and some attempt was made to explore reinforcement minimization through considerations of the moment volume (23). Some of the specimens were cast with integral edge beams while others were simply supported in the classical fashion. Some had sizeable openings cast non-centrally, or were provided with a central column support.

Despite the relatively large departures from isotropic reinforcement resulting from the application of the strip method, the slab specimens were found in all cases to considerably exceed the design ultimate load, to show only very fine cracking at working load and to have acceptably small working load deflections for the slab (as distinct from the beam slab system *in toto*).

Four large (3.05 m x 3.05 m) square simply supported slabs were tested by Rozvany (19). The reinforcement layout for three of these slabs was based on the optimal (minimum reinforcement) design and was highly anisotropic in character, see Figure 1. The fourth slab contained isotropic reinforcement. In all four cases cracking under load did not occur until after the design ultimate load had been exceeded. The tests have been criticized (19) on the grounds that the concrete tensile strength was sufficiently high to mask expected working load behaviour in real slabs. Nevertheless, cracking, when it did occur, was acceptable in pattern and crack size. For moderate loads the optimal (anisotropic) slabs were marginally stiffer than the isotropic slab. At higher loading (beyond ultimate design capacity), the roles reversed, with only the isotropic slab showing development of tensile membrane action. This is a trend noticed in other series of tests also.

Related to the previous series of tests is the series of four slab tests reported by Muspratt (16, 17). Once again some quite extreme patterns of reinforcement were used in square, simply supported slabs with two different versions of optimal layouts, one strip design and one isotropic design, being tested. The geometry of the slabs was otherwise identical to Rozvany's, although the design loading was set 70 per cent higher in an attempt to overcome the effect of concrete tensile strength. Experimental observations show initial cracking at 70 to 100 per cent of design ultimate load, indicating a rather high concrete tensile strength (14), see Figure 2. Apart from this, the results again indicate some advantage for banded (anisotropic) reinforcement arrangements, such as are derived from a strip method calculation, but possibly less strong support for the optimal reinforcement patterns.

In this context, it should be observed that for the case of a square, simply supported slab, the reinforcement based on the optimal calculation can, and probably should, be interpreted as main reinforcement being banded (13). This is an example of a type of slab where the strip and optimal approaches, when fully exploited, produce near identical recommendations for design. Tests on four

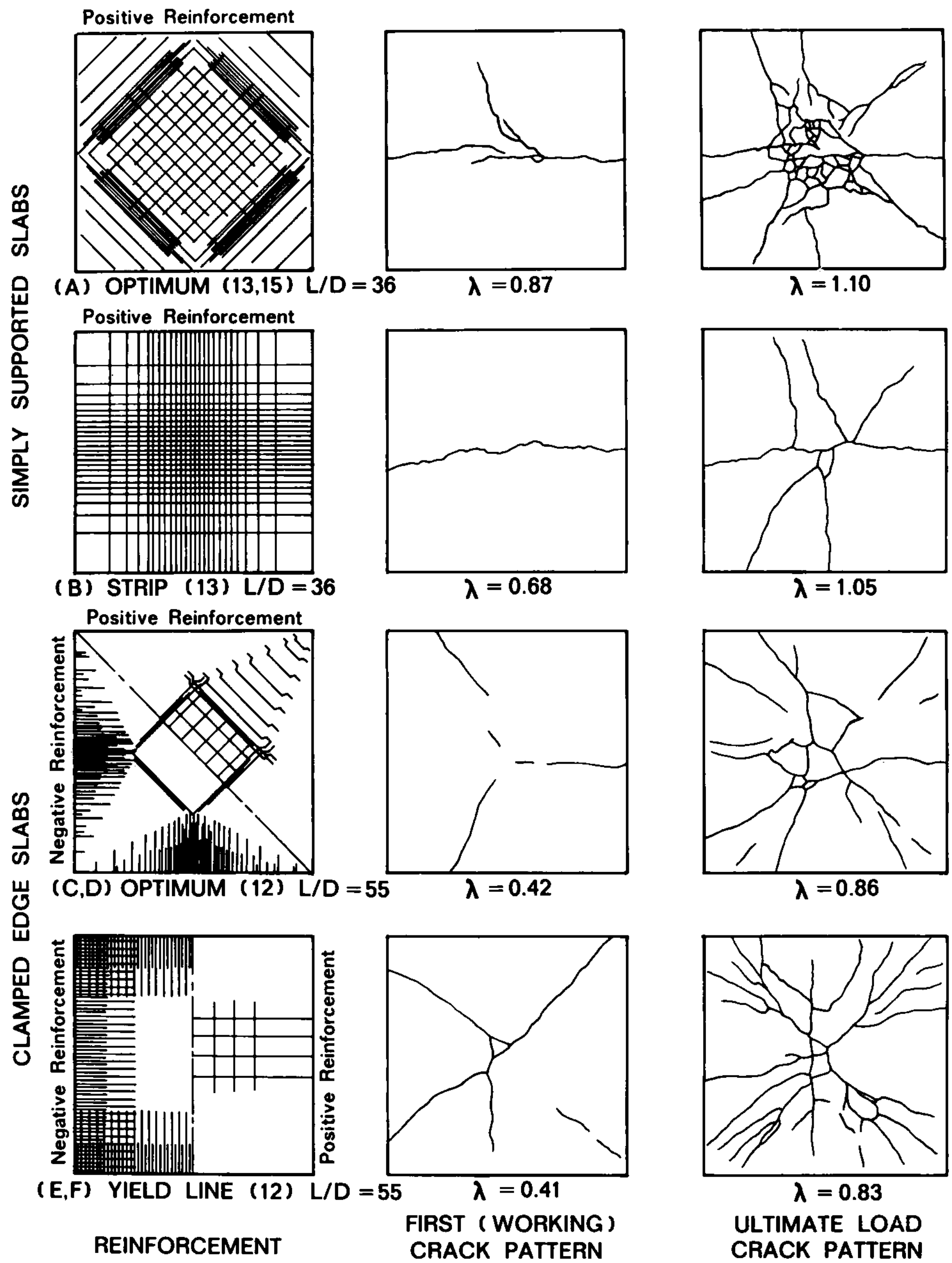

Figure 1 Anisotropic Layouts of Reinforcement and Crack Patterns at Working and Ultimate Loads

(Note: λ values corrected for effect of self weight; λ = 1 = design ultimate load)

0.81 m square, simply supported model slabs designed according to strip method rules but approximating in a reasonably practical fashion to optimal layouts, also confirm the above findings when compared to a paired isotropic (yield line) reinforcement model slab (13).

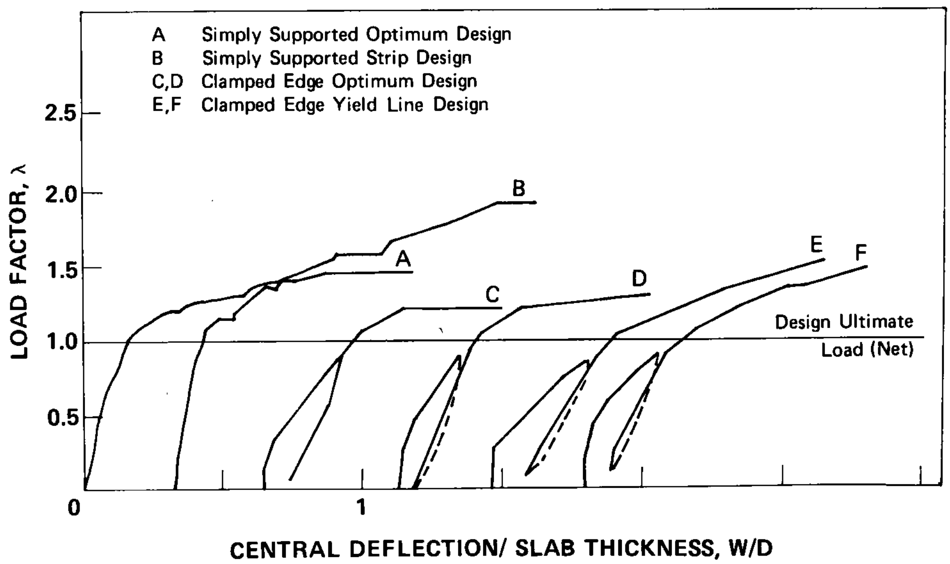

Figure 2 Load-Deflection Curves

The experimental investigations referred to thus far have been concerned with simply supported, or edge beam supported, slabs. Tests on fixed edge slabs have been reported by Charrett (6) and by Melchers (15). Charrett tested three 3.66 m square clamped slabs (L/D = 57) designed on the basis of (a) a simplified yield line method; (b) the strip method; and (c) a field of elastic moment (Poisson's ratio = 0). All three slabs behaved in a reasonable manner, despite the fact that the simplified yield line design was theoretically unsafe. All slabs cracked at loads between 55 and 60 per cent of the design ultimate load, with negligible crack size at that load level. No details of the initial crack patterns are available. The reinforcement in slab (b) was closely curtailed to the design moment field and considered to approximate closely the minimum that could be provided using an orthogonal reinforcement layout based on the strip method. At higher loads this slab developed a poorly spaced crack pattern of relatively large cracks. A factor not ascertainable from the reported work is the influence of bond loss in the development of this crack pattern. In all three slabs, cardboard packers placed in the slab periphery to reduce the development of a ring of outer compression with central tensile membrane action were found to act as crack initiators. The development of finely spaced crack patterns at ultimate load for slabs (a) and (c) is possibly attributable to their reinforcement layouts. The positive reinforcement in slab (a) was isotropic and uncurtailed; that in slab (c) was orthogonal and curtailed but with some bars at least extending across the slab. Due to various problems encountered in loading the slabs, it is not possible to validly compare their stiffness.

In the tests reported by Melchers (15), two slabs designed on the basis of optimal reinforcement and two on the basis of the yield line method were tested. The slabs were 2.8 m square between clamped supports. Particular care was taken to model the effects due to the tensile strength of the concrete. The L/D ratio for the slabs was kept high (= 55) to reduce the effect of concrete tensile strength and to reduce compressive membrane action. Also, although the slabs were clamped against moment, they were unrestrained in-plane in order again to reduce the effect of compressive membrane action. Cracking commenced below working loads, but did not become significant until mugh higher loadings had been applied, see Figure 1. As in the earlier tests on simply supported square slabs, the anisotropic (optimal) slabs exhibited greater stiffness at loads below ultimate but less stiffness at greater loads due to their inability to develop tensile membrane action, see Figure 2.

It might be noted here that the lack of development of tensile membrane action observed in optimally reinforced slabs (as distinct from strip method designs) is due to the reinforcement being discontinuous across the slabs in the optimal layouts, see Figure 1.

Beranek (4) recently reported some elegantly conducted tests on both isotropic and anisotropic slabs, supported in various manners. While the motives for the series were not to explore the question of performance of highly anisotropic slabs, some of the results do relate to the present discussion and suggest that anisotropy can be beneficial.

The behaviour characteristics of circular (1, 5, 18) and annular (6) slabs under various loadings have also been reported. The influence of the ratio of radial to circumferential reinforcement on the corresponding load capacity and cracking pattern was studied and compared to the behaviour of slabs reinforced with isotropic mesh. In general, in the cases reported, working load behaviour appeared satisfactory and design capacities were reached. In some cases, however, the cracking loads exceeded the design ultimateload (1, 6). In testing annular slabs with cracking loads of approximately 80 per cent of the design ultimate load (6), it was found that highly orthotropic (in the radial/circumferential sense) reinforcement designs were able to produce excellent behaviour when compared to more conventional designs. The tests on circular (and annular) slabs therefore supply some additional support for the positive role which anisotropy can play in slab design.

CONCLUSIONS

A number of studies are available to support the view that anisotropic reinforcement layouts can produce practical benefits of enhanced stiffness and strength compared with equal weight isotropic layouts. A notable feature of the experimental results is the lack of any violent difference in the real slab behaviour as compared with the quite violent difference in the concept of design. However, it is also clear that a number of points remain to be investigated.

Firstly, apart from Armer's tests on rectangular slabs (2), virtually all the available experimental results relate to circular and to square slabs. Other shapes need investigation. Indeed, it can be argued that the square shape (and certainly the circular shape) is so special due to its multiple symmetry allowing simplifications both in theory and experiment, that it should be avoided in order not to obscure the real practical utility of some of the novel concepts arising from both the strip method and optimal theory.

The second major point for investigation relates to interpreting the results of testing at what amount to model scale, while most often still using full size bars but in a restricted range of sizes. On a large scale, with more bar sizes and less tight dimensional tolerances there would appear to be significant scope for exploiting more fully the essential aspects of anisotropy which are inherently present in all the available methods of design for slabs.

While drawing attention to possible benefits from anisotropy, it should not be forgotten that there are practical prices to pay if these benefits are to be had. In the long run, a balance will need to be struck between possible savings in design time, increased steel fixing costs, possible savings in materials and an evaluation of benefit for enhanced structural response.

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SHORT-TIME DEFLECTIONS OF RECTANGULAR SIMPLY SUPPORTED RC SLABS

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ABSTRACT A method is proposed for the estimation of short-time deflections of two-way rectangular simply supported slabs for loads up to Johansen's load. The effect of cracking is included by choosing a decreasing moment of inertia function after cracking. Two coefficients, which have been determined from nineteen test results reported in the literature, are used in formulating the function. The variations of these coefficients are studied with respect to a non-dimensional parameter derived from the sectional properties of the slab and the strengths of the materials used. Using the proposed method, deflections have been predicted for comparison with experimental test values, and a satisfactory agreement obtained.

INTRODUCTION

Deflection is one of the serviceability requirements to be satisfied in the design of reinforced concrete slabs, and hence the estimation of deflection prior to Johansen's load and at working load is important to complement the strength criterion in the limit state analysis of structures. As an indirect means of restricting deflections, some of the codes of practice (1) specify the limiting span to thickness ratio. This specification is conservative since the influence of all parameters on deflection is included in a single parameter 'thickness'. The A.C.I. manual of concrete practice (2) recommends Branson's method, which is used for beam deflection calculations, for slabs also but a further verification of this is essential due to the difference in the structural behaviour of beams and slabs. In a recent state of the art paper (3) some methods to predict the deflection of two way reinforced concrete floor systems have been reviewed. Methods based on nonlinear finite element analysis have also been developed for predicting the load deflection behaviour of slabs in some of the investigations reported (4,5). Since this method of analysis needs a large capacity computer, it cannot readily be used in all design offices. Work has also been carried out by Herzog (6), who presented a method to determine the deflection of reinforced concrete slabs, Shukla and Mittal (7), who proposed a semi-empirical method for computing the short time deflection of reinforced concrete slabs based on their test results, and Desayi and Kulkarni (8), who reported on a determination in stages of a load-deflection curve for restrained reinforced concrete slabs.

In the present study an attempt has been made to propose a method which is simple from the computational point of view and which satisfactorily predicts deflections upto Johansen's load in two way simply supported slabs under distributed loading. The load-deflection curve is predicted in two stages. In the first stage, the deflection upto cracking load is estimated, assuming that the section is uncracked. In the second stage, i.e. from cracking load to Johansen's load, the effect of cracking is included in the analysis and thus the load-deflection curve upto Johansen's load is established.

NOTATION

A_{sx}	Area of steel in section 1-1
A_{sy}	Area of steel in section 2-2
d_x	Effective depth of steel in section 1-1
d_y	Effective depth of steel in section 2-2
E_c	Modulus of elasticity of concrete
f'_c	Cylinder strength of concrete
f_y	Yield stress of steel
h	Thickness of slab
I_g	Gross moment of inertia
I_{eff}	Effective moment of inertia
k_1	Proportional coefficient
k_2	Power coefficient
L_x	Length of shorter span of the slab
L_y	Length of longer span of the slab
q	Intensity of uniformly distributed load
q_{cr}	Intensity of cracking load at cracking moment
q_W	Intensity of working load taken equal to $^2/_3\, q_J$
X	$(\rho_x + \rho_y)(f_y/f'_c)(L_y/L_x)(L_x/h)^2$
β	Constant depending on appropriate span ratio
Δ	Deflection in stage 2 corresponding to the intensity of uniformly distributed load q
δ_{cr}	Deflection at cracking load
δ_I	$\Delta - \delta_{cr}$
δ_J	Deflection at Johansen's load
δ_W	Deflection at working load
ρ_x	Percentage of steel in section 1-1 = A_{sx}/bd_x
ρ_y	Percentage of steel in section 2-2 = A_{sy}/bd_y

PROPOSED METHOD

An idealised load-deflection curve OAB of a two way slab is shown in Figure 1 with two stages OA and AB indicated. In the first stage, the deflection is calculated using classical theory of plates upto cracking load. In the second stage, i.e. from cracking load to Johansen's load, the flexural rigidity gradually decreases, this being effected by modifying the moment of inertia to take into account the effects of cracking and the decrease in the modulus of elasticity of concrete.

First Stage

In the idealised load-deflection curve, the deflection upto point A in Figure 1 is estimated using classical plate theory. Thus:

$$\delta = \frac{\beta\ q\ L_x^4}{E_c\ I_g} \tag{1}$$

in which β is a constant for the simply supported plate and the appropriate span ratio (9). The value of E_c is determined from $4781 \sqrt{f'_c}$ N/mm^2 ($57600 \sqrt{f'_c}$ psi) where f'_c is in N/mm^2 or its equivalent (2).

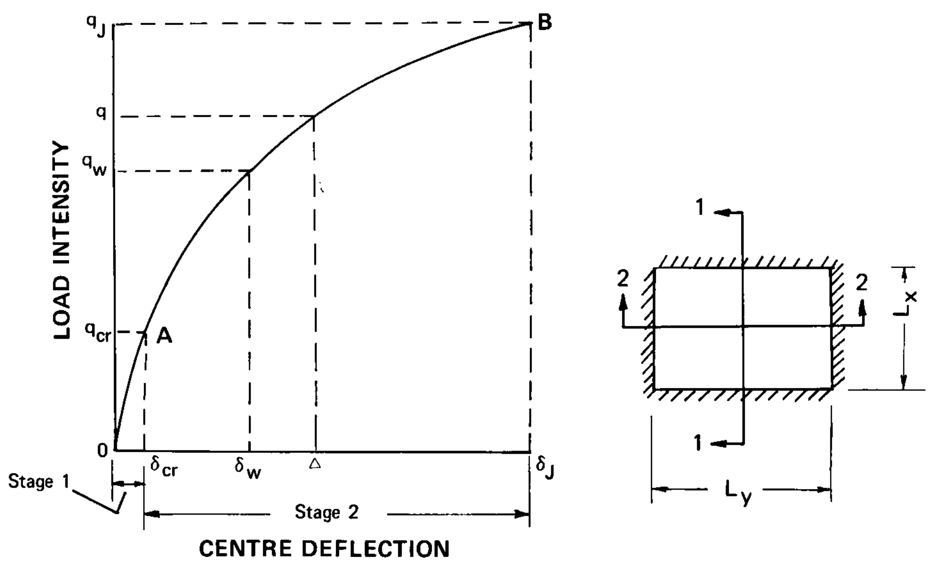

Figure 1 Idealised Load-Deflection Curve for a Two-Way Slab

At point A in Figure 1, $\delta = \delta_{cr}$ and from equation (1):

$$\delta_{cr} = \frac{\beta\, q_{cr}\, L_x^4}{E_c\, I_g} \tag{2}$$

The intensity of the cracking load corresponds to the moment at cracking which is determined using a modulus of rupture value of $0.6225 \sqrt{f'_c}$ N/mm^2 ($7.5 \sqrt{f'_c}$ psi) or its equivalent (2).

Second Stage

After cracking, the flexural rigidity reduces as the load is increased. A representation of the flexural rigidity of the slab after cracking should thus be of a deteriorating nature, such that it satisfies the value of I_g at cracking load and a fraction of I_g at Johansen's load. Hence it is assumed that the effective moment of inertia in the second stage is of the form:

$$I_{eff} = I_g \left[1 - k_1 \left(\frac{q - q_{cr}}{q_J - q_{cr}}\right)^{k_2}\right] \tag{3}$$

Equation (3) satisfies the required two conditions, namely that when $q = q_{cr}$,

$I_{eff} = I_g$, and when $q = q_J$, $I_{eff} = I_g(1 - k_1)$.

Thus the deflection in the second stage, AB in Figure 1, is computed from the expressions:

$$(\Delta - \delta_{cr}) = \delta_I = \frac{\beta\ (q - q_{cr})L_x^4}{E_c\ I_{eff}} \tag{4}$$

In equation (4), δ_I can be determined only if I_{eff} is known. To estimate I_{eff}, the value of the two coefficients k_1 and k_2 have to be evaluated.

Evaluation of Coefficients

Proportional Coefficient (k_1)

Substituting equation (3) in (4) and simplifying:

$$k_1 \left(\frac{q - q_{cr}}{q_J - q_{cr}}\right)^{k_2} = 1 - \frac{\beta\ (q - q_{cr})L_x^4}{E_c\ I_g(\Delta - \delta_{cr})} \tag{5}$$

In equation (5), if $q = q_J$, $\Delta = \delta_J$ and so:

$$k_1 = 1 - \frac{\beta\ (q_J - q_{cr})L_x^4}{E_c\ I_g(\delta_J - \delta_{cr})} \tag{6}$$

From equation (6) the values of k_1 for the reported test results can be evaluated.

Power Coefficient (k_2)

If it is assumed that the working load q_w is equal to $^2/_3\ q_J$, then q_w and the corresponding Δ (= δ_w) can be obtained from plots of experimental results. Substituting q_w and δ_w in equation (5), since q_w is usually > q_{cr}, and simplifying gives:

$$k_2 = \frac{\log \left[\frac{1}{k_1}\ \left\{1 - \frac{\beta\ (q_w - q_{cr})L_x^4}{E_c\ I_g(\delta_w - \delta_{cr})}\right\}\right]}{\log \left(\frac{q_w - q_{cr}}{q_J - q_{cr}}\right)} \tag{7}$$

The value of k_2 can be determined for any reported test results from equation (7) since the value of k_1 has already been determined from equation (6).

In this manner the values of k_1 and k_2 have been determined from a total of nineteen test results from four investigations reported in literature. Of these, 12 slab tests were carried out by Desayi and Kulkarni (10, 11) and 4 sets of test results were reported by Shukla and Mittal (7). Of the test results reported by Taylor et al (12) only two corresponding slabs, S_1 and S_7 with reinforcement parallel to the sides have been considered. Of the two tests carried out by

Sawzuck and Winnicki (13), only one (Type II, $\alpha = 2$) was considered as the other one (Type I, $\alpha = 1.45$) gave the value of $k_2 > 1$ which is unacceptable. The values of k_1 and k_2 so determined are shown in Table 1 for various test results.

Table 1 Values of X, k_1, k_2 for Nineteen Test Results

INVESTIGATORS	SLAB NO.	X	k_1	k_2
Desayi and Kulkarni (10, 11)	S_1	38.186	0.907	0.115
	T_1	101.443	0.785	0.198
	S_2	40.454	0.903	0.069
	T_2	107.591	0.819	0.139
	S_3	52.067	0.799	0.088
	T_3	123.684	0.744	0.159
	S_4	46.203	0.880	0.045
	T_4	121.18	0.845	0.077
	S_5	48.975	0.858	0.064
	T_5	129.61	0.819	0.190
	S_6	75.649	0.843	0.068
	T_6	179.6	0.698	0.143
Shukla and Mittal (7)	S_8	237.926	0.789	0.206
	S_{10}	31.482	0.926	0.489
	S_{11}	274.714	0.814	0.201
	S_{12}	226.780	0.845	0.343
Taylor, Maher and Hayes (12)	S_1	210.296	0.810	0.091
	S_7	455.136	0.826	0.035
Sawzuck and Winnicki (13)	Type II ($\alpha = 2$)	259.16	0.953	0.171

The variation of the coefficients with respect to the physical parameters of the system, such as the percentage of steel in two directions, the ratio of length of shorter span to the thickness of the slab, the ratio of yield stress of steel to cylinder strength of concrete, and the aspect ratio, has been individually examined. Then the variation of coefficients with the group of these parameters was also studied. Statistical best fitting straight lines have been drawn for those lumped parameters which yield the best correlation coefficient.

The variation of the coefficients with the non-dimensional parameter X is shown in Figure 2, where:

$$X = (\rho_x + \rho_y)\left(\frac{f_y}{f'_c}\right)\left(\frac{L_x}{L_y}\right)\left(\frac{L_x}{h}\right)^2$$

Values of X for different test results are given in Table 1. The equations for the best fitting straight lines of Figure 2 are:

$$k_1 = 0.87761 - 4.1604 \times 10^{-4} X \tag{8}$$

$$k_2 = 0.025227 + 0.000828\ X \tag{9}$$

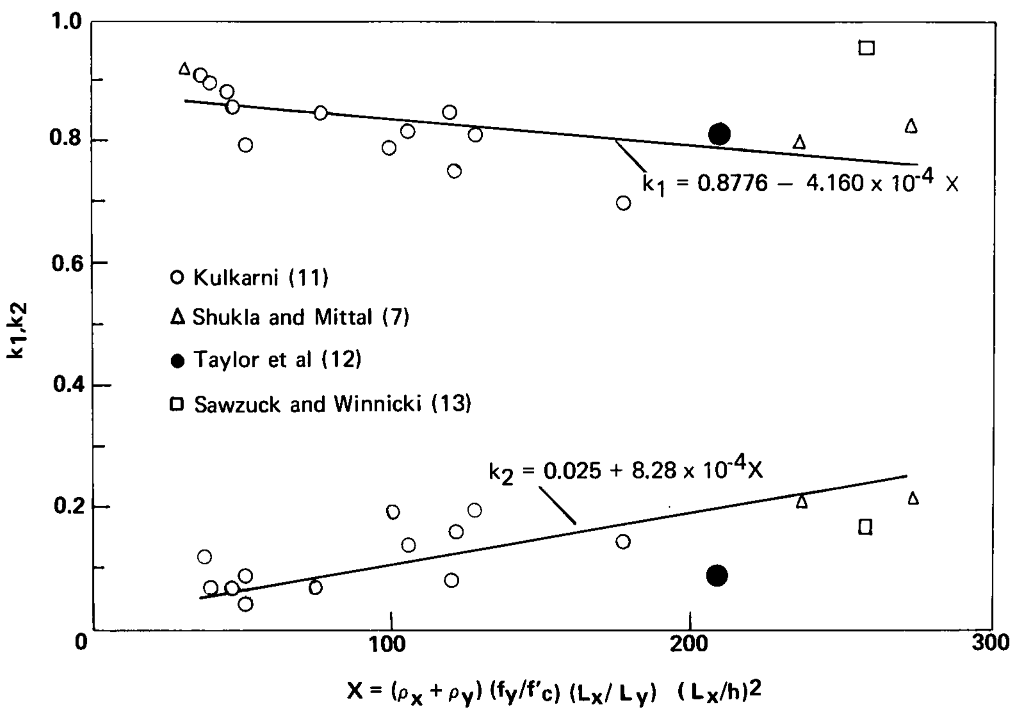

Figure 2 Variation of K_1 and K_2 with X

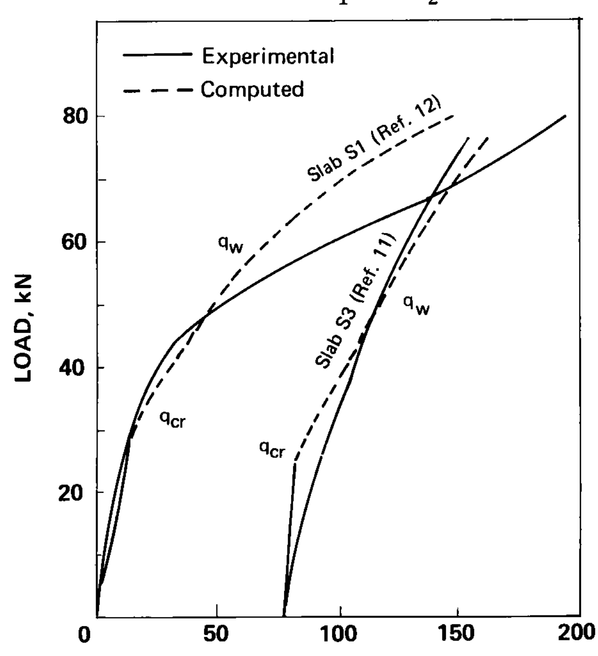

Figure 3 Comparison of Experimental and Computed Load Deflection Curves

Table 2 Comparison of Deflections at Working Load and at Johansen's Load

INVESTIGATORS	SLABS TESTED		RATIO OF EXPERIMENTAL TO COMPUTED DEFLECTION	
	Condition	Slab Number	At working load	At Johansen's load
Desayi and Kulkarni (10,11)	Rectangular (isotropic and orthotropic)	S_1	1.059	1.462
		T_1	0.751	0.782
		S_2	1.236	1.417
		T_2	0.912	0.925
		S_3	0.755	0.727
		T_3	0.742	0.693
		S_4	1.184	1.174
		T_4	1.175	1.103
		S_5	1.014	1.008
		T_5	0.903	0.976
		S_6	1.03	0.985
		T_6	0.628	0.665
Shukla and Mittal (7)	Square (isotropic and orthotropic, corners held down)	S_8	1.048	1.05
		S_{10}	0.538	0.605
		S_{11}	1.209	1.178
		S_{12}	0.992	1.025
Taylor, Malur and Hayes (12)	Rectangular orthotropic	S_1	1.182	1.457
		S_7	2.043	1.700
Sawzuck and Winnicki (13)	Rectangular isotropic	Type II (α = 2)	0.626	0.955
			mean = 1.001	mean = 1.047
			c.v. = 0.323	c.v. = 0.278

with corresponding correlation coefficients of -0.579 and 0.664 respectively. Equations (8) and (9) are applicable in the region $40 \leqslant X \leqslant 270$.

PREDICTION OF LOAD-DEFLECTION CURVES

For a given slab, deflections are determined for $0 < q < q_{cr}$ using equation (1), then δ_{cr} is determined from equation (2). The values of k_1 and k_2 are evaluated from equations (8) and (9). Substituting these coefficients in equation (3), I_{eff} is computed, and using equation (4), the value of Δ is determined for a particular intensity of load $q > q_{cr}$. The process is repeated at different values of q in the region $q_{cr} < q < q_J$ and the load-deflection curves thus obtained.

RESULTS AND COMPARISON

The coefficients k_1 and k_2 having been determined from equations (8) and (9), load-deflection curves have been computed using the proposed method and compared with available test results. Table 2 shows the comparison of the computed and experimental deflections at working load and at Johansen's load for all nineteen of the experimental investigations considered, while Figure 3 illustrates the comparison in two particular cases.

CONCLUSIONS

In this paper a method is presented for prediction of the load-deflection curve from zero load to Johansen's load for simply supported rectangular slabs in which the reinforcement is parallel to the span and which are subjected to uniformly distributed loading. The method requires a knowledge of two coefficients k_1 and k_2, values of which have been determined from available test data using equations (8) and (9). The deflection calculated using this method at working load and at Johansen's load were compared with the experimental test results and it was found that at working load, the average ratio of experimental to calculated deflections is 1.001 and the coefficient of variation is 0.323, while at Johansen's load the corresponding values are 1.047 and 0.278 respectively. Thus the proposed method is able to predict satisfactorily the deflections of simply supported slabs for loads up to the Johansen's load.

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THE INFLUENCE OF BAR SPACING ON TENSION STIFFENING IN REINFORCED CONCRETE SLABS

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ABSTRACT The concrete between the cracks in the tension zone of a cracked flexural member makes a significant contribution to the flexural stiffness of the member. This paper describes tests which provide data on the tension stiffening effect and, in particular, the influence of bar spacing on tension stiffening in slabs. It is concluded that the tension stiffening can be calculated on the basis of an average tensile stress equal to a fraction of the tensile strength of the concrete acting over an effective area of concrete surrounding the bars in the tension zone. The tension stiffening reduces, with an increase in strain, at a rate which depends upon the bar spacing.

INTRODUCTION

The trend in the recent past for economy in the use of materials has led to a requirement on the part of the designers for long spans of minimum depth, with the consequence that deflections are now often the governing design criterion for reinforced concrete slabs. This has resulted in a necessity to calculate accurately the flexural stiffnesses of cracked reinforced concrete flexural members.

The traditional concentration on strength rather than stiffness in the design of reinforced concrete has meant that it has been usual to ignore the tensile strength of the concrete. As a result, early theoretical studies of deflection used the classical elastic equations to calculate curvatures. These equations may be modified by including a triangular distribution of tensile stress in the concrete below the neutral axis with the maximum stress equal to the tensile strength of the concrete; this gives the result shown in Figure 1(a). It can be seen that once the point of cracking is passed the effect of including this tensile stress rapidly becomes very small. When, however, experimental data for a reinforced concrete flexural member are compared with theoretical calculations the typical result is as shown in Figure 1(b); a similar graph is obtained if, instead of curvatures, the strains at the tension steel level are considered. It can thus be seen that the average steel strain is reduced even at advanced levels of strain several times the cracking strain of the concrete. It follows that the average force in the tension steel is significantly reduced from the calculated theoretical level assuming no tensile strength of the concrete. In order to satisfy equilibrium there must be a compensating force in the concrete surrounding the steel. This tensile force in the concrete is generally known as the tension stiffening effect.

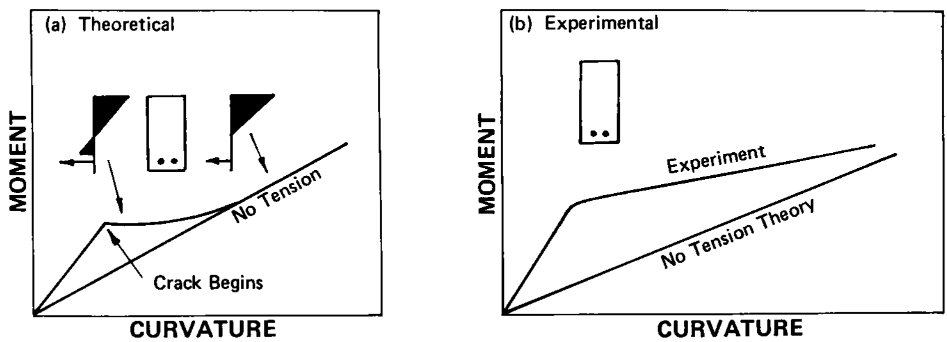


Figure 1 Moment-Curvature Relationships, (a) Theoretical and (b) Experimental

In this paper the tension stiffening effect is considered and an approximate theory, which takes account of the most recent research on cracking, is put forward. In addition, tests of one-way spanning slabs under short-term loading are described; the results of these tests are consistent with the proposed theory and enable the evaluation of empirical constants in the theoretical equations.

NOTATION

A_c	Effective area of concrete in tension
A_s	Area of tension steel
b	Overall breadth of section
b_c	Breadth of effective area of concrete in tension
c	Cover to reinforcement
d	Effective depth of reinforcement
E_s	Modulus of elasticity of steel
f_t	Tensile strength of concrete
f_y	Characteristic strength of steel
F_t	Tension stiffening force in concrete
F_{tp}	Maximum value of F_t
h	Overall depth of section
h_c	Depth of effective area of concrete in tension
I_c	Second moment of area of cracked transformed section
I_u	Second moment of area of uncracked section
n	Number of bars or bundles of bars
x	Neutral axis depth
β	Factor defining average tensile stress over A_c
β_2	Value of β when F_t is a maximum
γ	Factor defining the rate of decrease of F_t with respect to strain
ε_s	Strain in the tension steel ignoring tension stiffening
ε_{sm}	Average steel strain allowing for tension stiffening
ε_{sp}	Value of ε_s at which F_t is a maximum
σ_s	Steel stress in cracked section
σ_{sr}	Steel stress in cracked section at the cracking moment
ϕ	Bar diameter.

THEORY

The theory developed is based largely on suggestions made by Rao and Subrahamanyam (1). It is assumed that an area, A_c, of concrete around the reinforcement is effective in providing stiffening. If at any stage, ε_{sm} is defined as the actual average steel strain then, assuming that the lever arm with tension stiffening taken into account is not significantly different from that calculated from classical theory:

$$\varepsilon_{sm} = \varepsilon_s - \frac{\beta f_t A_c}{A_s E_s} \tag{1}$$

where ε_s is the strain in the tension steel calculated ignoring tension stiffening, A_s is the area of tension steel, E_s is the modulus of elasticity of the steel, f_t is the tensile strength of the concrete and β is a factor defining the average tensile stress over the effective area.

The experimental evidence shows a gradual reduction in tension stiffening with an increase in strain and Rao and Subrahmanyam (1) suggest that the ratio of the steel stress just after cracking to the steel stress under consideration is a suitable parameter to describe the reduction in tension stiffening. In the present approach this parameter is altered slightly to the ratio of the calculated steel strain, ε_{sp}, when the tension stiffening is a maximum, at which point $\beta = \beta_2$, to the calculated steel strain under consideration. Equation (1) is then rewritten as:

$$\varepsilon_{sm} = \varepsilon_s - \frac{\beta_2 f_t A_c}{A_s E_s}\left(\frac{\varepsilon_{sp}}{\varepsilon_s}\right)^{\gamma} \tag{2}$$

where γ is an index to be determined.

The effective area is now considered. After cracking the tension stiffening effect arises from two sources. Firstly, there is the contribution of the tension induced in the regions of concrete between the cracks (the concrete teeth) by the flexure of the compression zone; this contribution occurs even with zero bond between the steel and concrete. Secondly, there is the contribution of the tension transferred to the concrete from the steel by bond. The latter contribution is now considered in the light of recent research on cracking (2).

In flexural members the major cracks are at a spacing of around 1.3 times their height. If a simple 45° spread of tensile force is assumed from the point where a reinforcing bar enters a major crack interface then, at a point along the bar midway between major cracks, tensile stress will have spread out from the bar by 0.65 times the height of the crack. A reasonable hypothesis might be to assume an average spread of 0.325 (h-x) above the bar where h is the depth of the member and x is the depth of neutral axis calculated ignoring tension stiffening. Thus the height, h_c, of the effective area is given by:

$$h_c = 0.325\ (h-x) + c + \phi \tag{3}$$

where c is the cover and ϕ the bar diameter. This is a pessimistic estimate of h_c because the effect of the flexure of the concrete teeth has been ignored and it is to be expected that h_c will exceed the value implied by equation (3) but is unlikely to exceed (h-x).

If the 45° spread is also considered laterally then the width, b_c, of the effective area is given by:

$$b_c = n[0.65\ (h-x) + \phi] \quad \text{for} \quad s \geqslant 1.3\ (h-x) + \phi \tag{4}$$

where n is the number of bars or bundles of bars and s is their spacing. It can be seen that if the bar spacing is less than [1.3 (h-x) + ϕ] then the effective areas of adjacent bars overlap and the net effective area is less than that given by equation (4). Thus the effective area, expressed as a percentage of the area of the concrete tensile zone, is predicted to be a maximum when the bar spacing is of the order of 1.3 (h-x).

From the above discussions it is evident that the proposed theory can only indicate trends in the values of A_c and tests are required to quantify A_c.

TESTS PREVIOUSLY REPORTED

Clark and Speirs (3) have reported tests on fourteen beams and nine slabs to examine the hypotheses described above.

In the case of a beam, with closely spaced bars, it is reasonable to assume that b_c = b (the overall breadth) and, on this basis, it was found that h_c = h-x (i.e. for a beam the complete tensile zone contributes to the effective area) and that β_2 had a mean value of 0.293. The strain at which the tension stiffening was a maximum was found to be given by 200 x $10^{-6} I_u/I_c$ where I_u and I_c are respectively the uncracked and cracked second moments of area. The rate of decay of tension stiffening was predicted adequately by $\gamma = 1$, as was also found to be the case by Rao and Subrahmanyam (1).

When the slab data were analysed it was found that there was considerable scatter of the results but there was a tendency for the tension stiffening force to decrease with an increase in bar spacing. It was proposed that A_c should be taken as the complete tensile zone and that β_2 should be considered to decrease with an increase in bar spacing, as:

$$\beta_2 = 0.4 - s/3b \tag{5}$$

It was further proposed that:

$$\varepsilon_{sp} = 670 \times 10^{-6} \beta_2 I_u/I_c \tag{6}$$

and it was found that the rate of decay of tension stiffening increased with an increase in bar spacing and could be predicted reasonably by:

$$\gamma = 3s/b \quad \text{but} \nless 1 \tag{7}$$

In view of the large scatter of the slab data it was decided to test a further eight slabs, which are described in this paper. One further beam was also tested but this is not discussed other than to say that the data obtained confirmed the above observations on tension stiffening in beams.

Equations (5) to (7) are reconsidered in this paper since the more general parameter s/(h-x) has been introduced in place of s/b which is not appropriate since the width of the specimen is an arbitrary quantity.

DETAILS OF SLABS

Details of the slabs are given in Table 1. Slabs 1 to 9 are the first series which have already been reported (3) and 10 to 17 are the second series. The bar spacing was varied by grouping the bars into bundles of one, two and three bars. Each slab was nominally 3.5 m long, 900 mm wide, 200 mm deep and 165 mm effective depth. The reinforcement was GK Torbar deformed steel which is cold-worked and had a stress-strain curve which became non-linear at about 1500 microstrain. Four 8 mm bars were provided as top steel and nominal links were provided in the shear spans. The concrete mix had a maximum aggregate size of 19 mm.

The slabs were tested under a four-point loading system which gave a constant moment zone of 1.2 m and two shear spans of 1.0 m each.

Table 1 Slab Details

SLAB	NOTES	WIDTH (b) mm	DEPTH (h) mm	MAIN STEEL, 6No. Bars					INDIRECT TENSILE STRENGTH (f_t) N/mm^2
				Number of Groups	Group Spacing (s) mm	Diameter (ϕ) mm	Depth (d) mm	Amount %	
1	-	902	204	6	150	20	169	1.24	2.65
2	-	904	204	3	301	20	169	1.24	2.74
3	-	902	205	2	451	20	169	1.24	2.92
4	-	901	204	6	150	16	169	0.80	3.26
5	-	901	203	3	300	16	169	0.79	2.26
6	-	903	203	2	452	16	171	0.78	2.22
7	-	900	201	6	150	12	170	0.44	2.04
8	-	901	203	3	300	12	168	0.45	2.65
9	-	901	204	2	451	12	172	0.44	2.72
10	Repeat of 1	902	203	6	150	20	168	1.22	1.96
11	Repeat of 2	901	203	3	300	20	168	1.22	2.06
12	Repeat of 3	902	204	2	451	20	166	1.24	1.88
13	Repeat of 5	903	202	3	301	16	168	0.77	2.50
14	Unbonded steel	903	203	3	301	16	168	0.79	2.46
15	Repeat of 9	904	204	2	452	20	165	0.41	3.01
16	Repeat of 4	902	204	6	150	16	169	0.79	2.04
17	Repeat of 6	901	203	2	451	16	168	0.80	2.42

Concrete surface strains were measured on a 200 mm gauge length using a Demec gauge on the top and bottom surface along lines parallel to the span and situated at ¼, ½ and ¾ of the breadth of each slab. Reinforcement strains were measured by means of electrical resistance strain gauges on two bars of each of slabs 10, 11, 12, 13 and 15.

EXPERIMENTAL RESULTS

From equation (2) the tension stiffening force, F_t, can be written as:

$$F_t = \beta_2 f_t A_c (\varepsilon_{sp}/\varepsilon_s)^{\gamma} = A_s E_s (\varepsilon_s - \varepsilon_{sm}) \qquad (8)$$

The actual average strain at the steel level, ε_{sm}, was obtained from the concrete surface strain measurements and ε_s was obtained from an analysis which ignored the tensile strength of the concrete but which used the experimentally determined steel and concrete non-linear stress-strain curves. The tension stiffening force at any stage was then determined by multiplying the difference between these strains by the steel area and its modulus of elasticity. In Figure 2 the

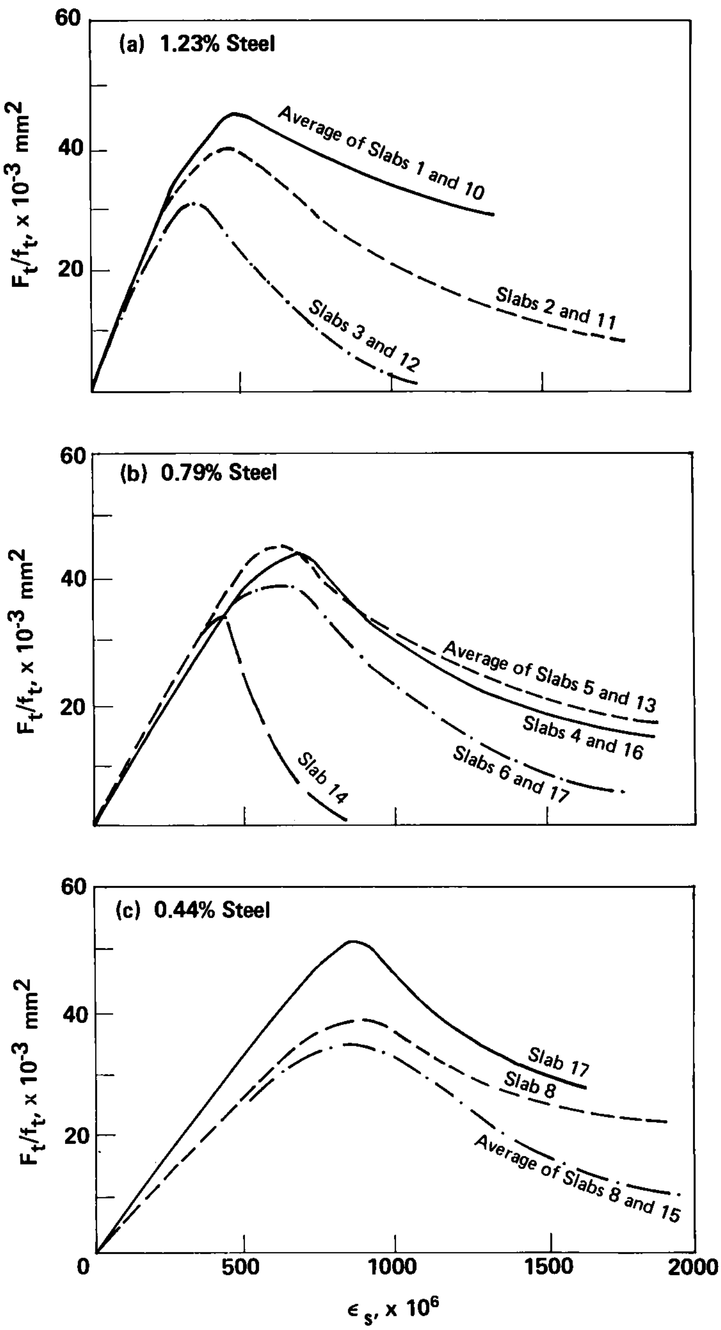


Figure 2 Tension Stiffening Forces

tension stiffening forces divided by the concrete tensile strength are plotted against calculated steel strain. It can be seen that there is a tendency for the peak values of F_t/f_t to decrease with an increase in bar spacing and for the decay to be more rapid for the largest bar spacing; the decay is extremely rapid for slab 14 with unbonded bars.

Maximum Values of Tension Stiffening Forces

The maximum values of tension stiffening force (given in Table 2) divided by concrete tensile strength and by the area of the complete concrete tensile zone are plotted against s/(h-x) in Figure 3 together with the beam data. There is

Table 2 Principal Results of Experimental Investigation

SLAB	MAXIMUM VALUE OF F_t kN	ε_{sp} x10^6
1	125	550
2	106	490
3	90	370
4	128	750
5	114	700
6	98	610
7	104	900
8	102	950
9	95	900
10	89	460
11	83	460
12	68	270
13	98	540
14	88	400
15	99	900
16	89	660
17	91	660

considerable scatter of the data but there is a tendency for F_{tp}/f_t b(h-x) to be a maximum at an s/(h-x) value of the order of 1.3, as predicted by the theory. However, for design purposes, it is not considered justifiable to allow for the small variation in maximum tension stiffening force with bar spacing and it is thus suggested that the effective area of the tensile concrete be taken as its actual area and that β_2 be taken as 0.3 for all bar arrangements (the mean value of β_2, calculated on the basis of A_c = b(h-x), for all beams and slabs is 0.295).

The strains, ε_{sp}, at which the tension stiffening forces were maxima are given in Table 2 and it can be seen that they increase with a decrease in steel percentage. This is to be expected since, if all the major cracks formed instantaneously, the value of ε_{sp} would be approximately equal to the concrete cracking strain multiplied by the ratio of the uncracked to cracked stiffness of the member. It is thus to be expected that the values of ε_{sp} should be approximately proportional to the ratio of uncracked to cracked second moments of area (I_u/I_c). That this is so can be seen from Figure 4, the data of which are fitted by the relationship:

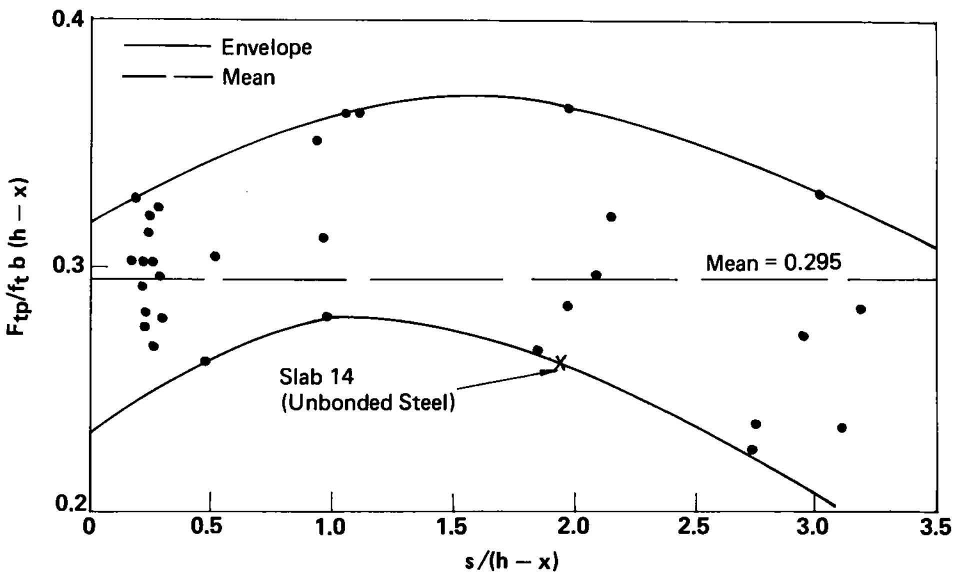


Figure 3 Influence of Bar Spacing on Maximum Tension Stiffening.

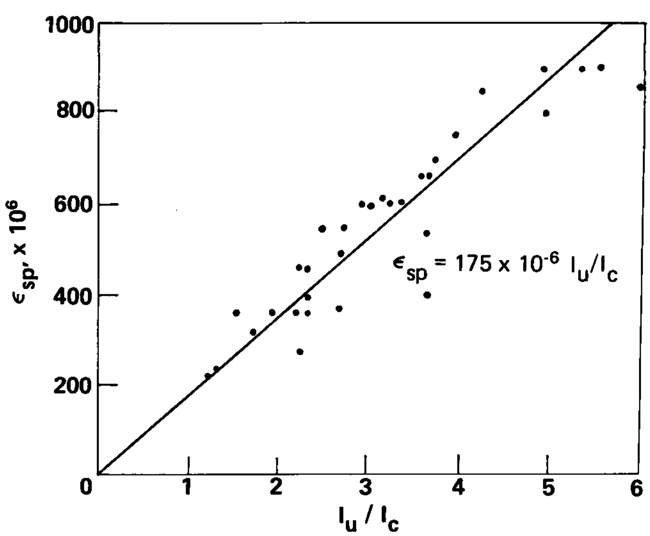


Figure 4 Steel Strain at $F_t = F_{tp}$

$$\varepsilon_{sp} = 175 \times 10^{-6} \, I_u/I_c \tag{9}$$

Decay of Tension Stiffening

It is to be expected that the index γ, which determines the rate of decay of tension stiffening, should be a function of $s/(h-x)$. A reasonable fit to the experimental data is obtained if γ is taken to be:

$$\gamma = s/2(h-x) \quad \text{but} \ngtr 1 \tag{10}$$

Thus the proposed tension stiffening formula is:

$$F_t = 0.3 \, f_t \, b(h-x) \, (\varepsilon_{sp}/\varepsilon_s)^{\gamma} \tag{11}$$

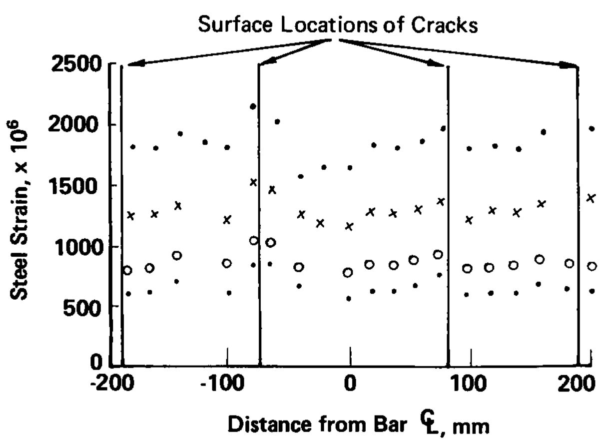
where ε_{sp} and γ are given by equations (9) and (10) respectively.

It should be noted that for $s \leqslant 2(h-x)$, $\gamma = 1$ and since, for a slab, x is in the approximate range of 0.2h to 0.3h the critical value of s below which bar spacing has no influence on tension stiffening is about 1.4h to 1.6h. As a design rule, it is suggested that for beams and slabs with a bar spacing not greater than 1½ times the slab depth the tension stiffening force can be calculated from:

$$F_t = 0.3 \, f_t \, b(h-x) \, (\varepsilon_{sp}/\varepsilon_s) \tag{12}$$

Measured Steel Strains

A typical plot of steel strain against distance along the bar is shown in Figure 5 for one bar of slab 11. The locations of the surface cracks are indicated and it can be seen that the steel strain is greater than average at these points. It can also be seen that the average steel strains from the electrical resistance strain gauges on the reinforcement agree very closely with the strains calculated from the Demec gauges on the concrete surface.



Mean Strain x 10^6

Steel	Concrete
2690	2630
1840	1790
880	850
670	720

Figure 5 Distribution of Steel Strain

COMPARISONS WITH CODES OF PRACTICE

In the following the various code equations are written in the notation used in this paper. In the latest version of the British Code (4) the steel strain allowing for tension stiffening is obtained from:

$$\varepsilon_{sm} = \varepsilon_s - \frac{0.7\ bh(d-x)\ 10^{-3}}{A_s\ (h-x)\ \sigma_s} \quad \text{but} \nless 0 \tag{13}$$

where d is the effective depth of the tension steel and σ_s its stress calculated on the basis of the cracked section. The equivalent equation in the European Code (5), for ribbed bars and for short-term loading is:

$$\varepsilon_{sm} = \varepsilon_s \left[1 - \left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right] \quad \text{but} \nless 0.4\ \varepsilon_s \tag{14}$$

In Figure 6 the tension stiffening values from equations (13), (14) and the proposed theory, equations (9) to (11) are compared for a section having a concrete tensile strength of 3N/mm^2, an h/d ratio of 1.1 and a steel percentage of 0.5 per cent. The tension stiffening is presented in terms of $(\varepsilon_s - \varepsilon_{sm})/\varepsilon_s$. It can be seen that the tension stiffening predicted by the CP110 formula exceeds that predicted by the CEB formula and that the proposed formula, which is dependent upon

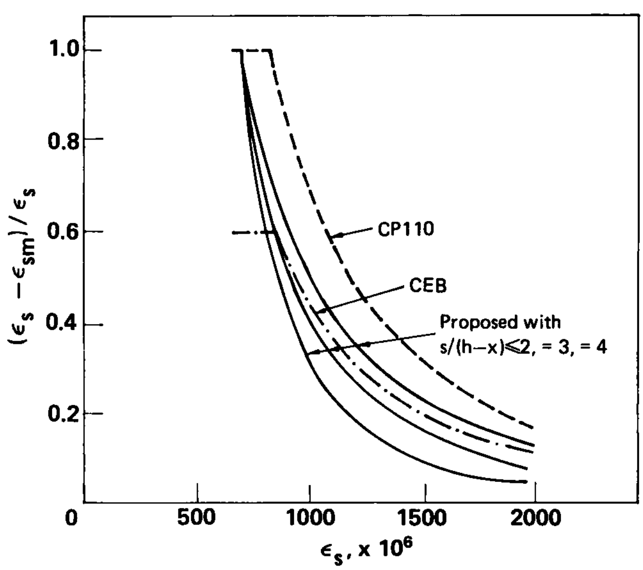


Figure 6 Comparison of Tension Stiffening Formulae

bar spacing, agrees closely with the CEB formula for bar spacing of the order of 1.5 times the slab depth but predicts less tension stiffening at larger bar spacings. At 1200 microstrain, which is the approximate service load strain for a design in accordance with CP110 using high strength steel, the CP110 formula predicts about 50 per cent more tension stiffening than do the CEB formula and proposed formula for the case of small bar spacings; when the bar spacing increases the above difference increases. The differences between the CP110 formula and the CEB and the proposed formulae decrease with an increase in h/d and increase with an increase in steel percentage, but in the latter case the tension stiffening becomes less significant in design terms.

CONCLUSIONS

The maximum tension stiffening force is a function of the tensile strength of the concrete and the area of concrete below the neutral axis and, for design purposes, can be considered to be independent of bar spacing.

The calculated steel strain at which the tension stiffening force is a maximum can be expressed in terms of the ratio of the uncracked to cracked moments of inertia.

The tension stiffening force decays with strain at a rate which increases with bar spacing when the latter exceeds about 1½ times the slab depth.

For small bar spacings, the proposed formula for tension stiffening agrees with the CEB formula but predicts less tension stiffening than does the CP110 formula.

ACKNOWLEDGEMENT

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A FINITE ELEMENT SOLUTION OF POST CRACKING BEHAVIOUR OF R.C. SLABS

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ABSTRACT The paper proposes a finite element incremental iterative procedure to predict the behaviour of reinforced concrete slabs in the pre and post cracking ranges. Numerically integrated higher order elements are used for discretization so that variability of material properties due to cracking and stress state changes under monotonically increasing load is conveniently simulated by assigning varied rigidities at different Gauss points within the element. Realistic moment curvature relations are developed for the purpose. The accuracy and efficiency of the proposed procedure are established by comparing predicted response with the experimental results of tests on three typical slabs reported in literature.

INTRODUCTION

With the increasing preference for ultimate load methods of analysis and design for reinforced concrete slabs as opposed to elastic methods, it becomes necessary to trace the complete response history of the structure through pre and post cracking ranges until failure so that the true safety against failure is assessed and serviceability requirements as regards deformations and cracking are ensured. Such a response can be correctly predicted only if due cognizance is taken of (i) the composite nature of reinforced concrete, (ii) non-linear material characteristics, (iii) anisotropy arising from unequal Young's modulii in principal directions due to different stress levels reached under increasing load and (iv) topological changes caused by the development of cracks in a random fashion. Yield line theory for slabs estimates only the ultimate load and it is too simplistic to cope with all the above complexities. The finite element method probably provides the only technique for such an analysis.

In the past, two distinct approaches have been adopted to obtain the necessary constitutive relations for use in the finite element method. The modified EI approach, adopted by Jofriet and McNeice (1) and applied within the working load range, utilizes a semi-empirical moment curvature relationship based on an empirical expression for the effective moment of inertia (Branson) or for the effective flexural rigidity (Beeby) for a cracked section. The second approach, known as the layered element approach, imagines every element to consist of a number of concrete and steel layers in plane state of stress so that the material property matrix can be written easily for any stress or cracked state. The entire element stiffness is obtained by summing up the stiffnesses of the layers. Many investigators (2) have adopted this approach to analyse reinforced concrete slabs and shells. Since concrete cracks randomly, and since it is not practical to redefine the finite element mesh at the appearance of every crack, the mesh grading has to be sufficiently fine in both these approaches. This in turn requires large computer storage and considerable manual and computational effort.

The present paper describes an automatic incremental iterative procedure which is shown to predict deflections, crack pattern and steel yielding in the case of reinforced concrete slabs even with a coarse mesh. This has been possible by using higher order numerically integrated elements which allow variation of material properties within the element. Realistic moment curvature relations using non-linear material properties have been developed and it has thus been possible to account for the effects of random cracking and crack penetration in reinforced concrete slabs. Evidence of the applicability and accuracy of the proposed procedure is given by reference to three examples.

PROPOSED FINITE ELEMENT FORMULATION

It is proposed that a fully conforming and extensively tested rectangular plate element (3) (with 16 degrees of freedom) be adopted,with vertical displacement, slopes in the x and y directions and twist as the four nodal displacements, as shown in Figure 1.

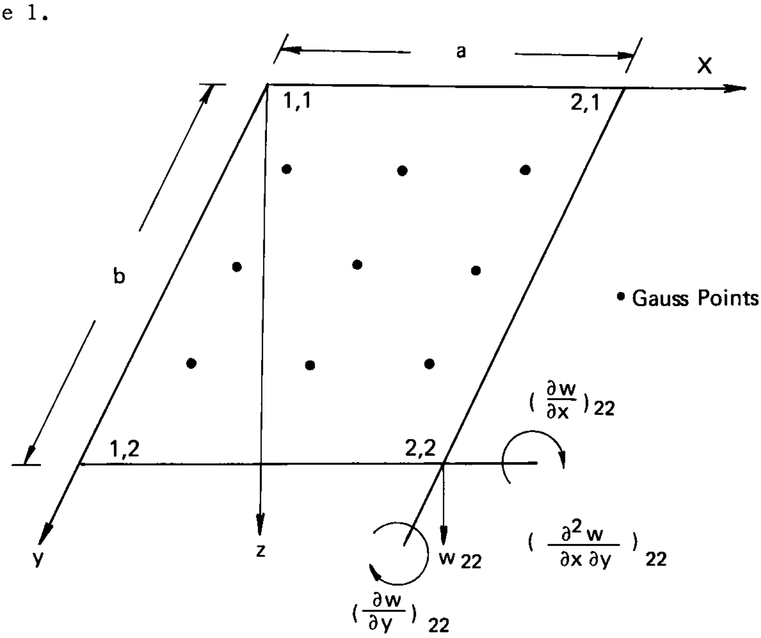

Figure 1 Plate Element with 16 Degrees of Freedom

The vertical displacement, ω, anywhere within the element is defined in terms of the nodal displacements with the help of first order Hermite interpolation polynomials, H_{oi}, H_{oj}, H_{1i} and H_{1j}, as given below:

$$\omega(x,y) = \sum_{i=1}^{2} \sum_{j=1}^{2} H_{oi}(x)\, H_{oj}(y)\, \omega_{ij} + H_{1i}(x)\, H_{oj}(y)\, \omega,_{xij} + H_{oi}(x)\, H_{1j}(y)\, \omega,_{yij} + H_{1i}(x)\, H_{1j}(y)\, \omega,_{xyij} \quad (1)$$

where $\omega,_{xij} = (\frac{\partial\omega}{\partial x})_{ij}$, etc.

The equations of equilibrium in incremental form are given by:

$$[K_T]\ d\ \{\delta'\} = d\ \{F\} \tag{2}$$

where $[K_T]$ is the tangent stiffness matrix of the structure and d $\{\delta'\}$ are increments in the nodal displacements due to an increment d $\{F\}$ in the applied nodal loads. The incremental iterative procedure is explained in Figure 2.

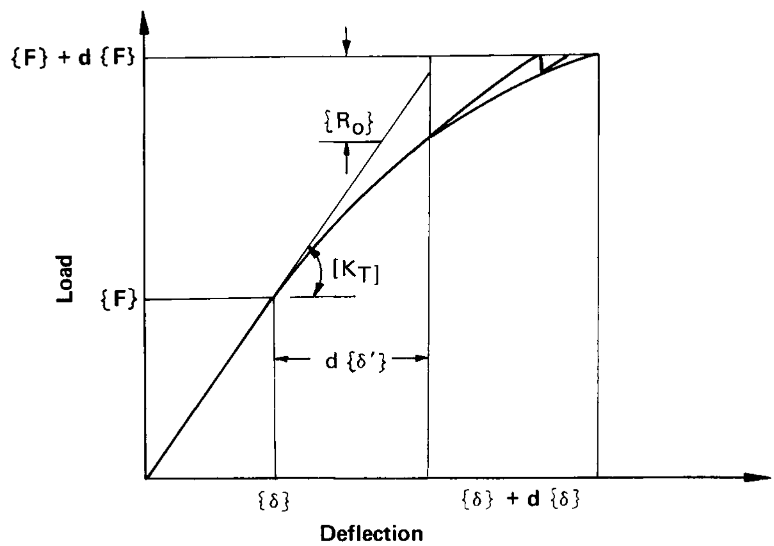

Figure 2 Incremental Iterative Procedure

Any residual force, $\{R_o\}$, which remains unbalanced in a given load increment after a prescribed number of iterations is added to the next load step. Failure of the structure is indicated by divergence of the unbalanced nodal forces or nonpositive definiteness of the structure stiffness. The element tangent stiffness, $[k_T]$, which add to form $[K_T]$, is given by:

$$[k_T] = \int [B]^T\ [D_T]\ [B]\ dA \tag{3}$$

in which [B] and $[D_T]$ are defined as:

$$d\ \{\chi'\} = [B]\ d\ \{\delta'\}^e$$

$$d\ \{\sigma'\} = [D_T]\ d\ \{\chi'\}^e$$

where $\{\chi\} = \{-\omega,_{xx}\ -\omega,_{yy} + 2\omega,_{xy}\}$ is a vector of curvatures, $\{\delta\}^e$ is the vector of element nodal displacements and $\{\sigma\} = \{M_x M_y M_{xy}\}$. The explicit evaluation of

$[k_T]$ is not possible, but using the 9-point Gauss integration formula $[k_T]$ is evaluated as:

$$[k_T] = \sum_{i=1}^{3} \sum_{j=1}^{3} [B]_{ij}^T [D_T]_{ij} [B]_{ij} C_i C_j \text{ a b} \tag{4}$$

where C_i, C_j are the weighting functions associated with the Gauss point ij. It is thus necessary to determine the following at all the Gauss points:

$$[\bar{k}]_{ij} = [B_{ij}]^T [D_T]_{ij} [B]_{ij}$$

$[B]_{ij}$ is independent of material properties and easily determined. $[D_T]_{ij}$ must, however, be determined in such a way that the state of stress, yielding of steel and concrete cracking are accounted for.

The material property matrix, $[D_T']$, for an anisotropic material is assumed to be defined as:

$$[D_T'] = \begin{bmatrix} D_{x'} & D_1 & 0 \\ D_1 & D_{y'} & 0 \\ 0 & 0 & D_{x'y'} \end{bmatrix} \tag{5}$$

where

$$D_{x'} = \frac{E_{cx'} . I_{cx'}}{1 - \nu_c^2} = \frac{R_{x'}}{1 - \nu_c^2} ,$$

$$D_{y'} = \frac{E_{cy'} . I_{cy'}}{1 - \nu_c^2} = \frac{R_{y'}}{1 - \nu_c^2} ,$$

$$D_1 = \nu_c \sqrt{D_{x'} . D_{y'}} ,$$

$$D_{x'y'} = \tfrac{1}{2}(1 + \nu_c) \sqrt{D_{x'} . D_{y'}} ,$$

and in which $R_{x'} = \partial M_{x'}/\partial \chi_{x'}$, and $R_{y'} = \partial M_{y'}/\partial \chi_{y'}$, are tangent flexural rigidities in the x' and y' directions of orthotropy which have to be defined at each Gauss point.

The slab behaves anisotropically either because of the non-linear stress-strain relationship for concrete giving unequal Young's modulii in the two principal directions or when concrete cracks after tensile strain reaches a certain limiting value. The directions of orthotropy have been assumed to lie along the steel directions in the case of uncracked sections and along and normal to the cracks for cracked sections. The effective area of steel, A_{st}, in the directions of orthotropy has been found using expressions given by Lenshow (4) and the matrix

$[D'_T]$ can easily be transformed to $[D_T]$ in the x,y directions by a simple congruent transformation.

MOMENT CURVATURE RELATIONSHIP

In the absence of any precise data, the material properties have been assumed to be direction independent. The moment curvature relationship for a reinforced section has been developed, from considerations of strain compatibility and equilibrium of internal forces, by assuming a suitable stress-strain curve for concrete (5), a cracking strain, e_{cr}, for concrete and a yield strain for steel. A reinforced section may be in any one of the four strain states as shown in Figure 3. The expressions for d_n, the depth of the neutral axis, the moment of resistance, M, and the tangent rigidity, $R_T = \partial M/\partial X$ as developed by one of the authors (6) in terms of the curvature, are summarised below.

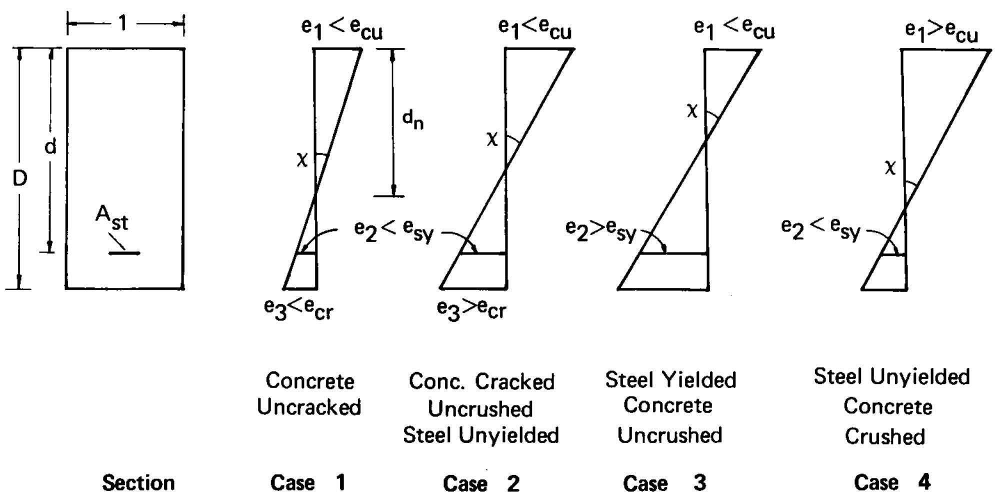

Figure 3 Strain Distribution at Various Stages

Case 1, Section Uncracked and Uncrushed

$$d_n = [- Dx + \{D^2x^2 - (1 - x)(\frac{e_o^2}{2} (1 - x) - xD^2)\}^{\frac{1}{2}}]/(1 - x) \tag{6}$$

where $\log_e x = 2(d - d_n)X^2 E_sA_{st}/E e_o^2$

$$R_T = \frac{E e_o^2}{X^3}(- DX - \frac{e_o^2 e_1}{e_{o1}} - \frac{e_o^2 e_3}{e_{o3}} + 2e_o\tan^{-1}\frac{e_oDX}{e'_{o13}}) + (d - d_n)^2 E_sA_{st}$$

$$+ [\frac{E e_o^4 e'_{13}}{X e_{o3} e_{o1}} - 2e_2 E_sA_{st}] \times [\frac{E e_o^2 (\log_e \frac{e_{o1}}{e_{o3}} - \frac{e_1^2}{e_{o1}} + \frac{e_3^2}{e_{o3}})}{X^3 E_sA_{st} + X^2 E e^2(\frac{e_1}{e_{o1}} + \frac{e_3}{e_{13}})}] \tag{7}$$

Case 2, Section Cracked and Steel Yielded

$$d_n = d - (E\, e_o^2/2\, \chi^2\, E_s A_{st})\, \log_e(e_{o1}/e_{ocr}) \tag{8}$$

$$R_T = \frac{E\, e_o^2}{\chi^3} \Big(-\, e_1 - 2\, e_{cr} - \frac{e_1 e_o^2}{e_{o1}} + 2\, e_o \tan^{-1} \frac{(e_1 + e_{cr})\, e_o}{e_{o1cr}}\Big) + (d - d_n)^2\, E_s A_{st}$$

$$+ \Big[\frac{E\, e_o^2\, e_1^2}{\chi\, e_{o1}} - 2\, e_2\, E_s A_{st}\Big] \times \Big[\frac{E\, e_o^2\, e_{o1}}{\chi^3 (E_s A_{st}\, e_{o1} + E\, e_o^2\, d_n)} \Big(\log_e \frac{e_{o1}}{e_{ocr}} - \frac{e_1^2}{e_{o1}}\Big)\Big] \tag{9}$$

Case 3, Section Uncrushed and Steel Yielded

$$d_n = \frac{1}{\chi} \Big[(e_{ocr}\, e^A - e_o^2)^{1/2}\Big] \tag{10}$$

$$R_T = \frac{E\, e_o^2}{\chi} \Big[-\, e_1 - 2\, e_{cr} - \frac{e_1 e_o^2}{e_{o1}} + 2\, e_o \Big(\tan^{-1} \frac{e_1}{e_o} + \tan^{-1} \frac{e_{cr}}{e_o}\Big)\Big]$$

$$+ \Big[\frac{E\, e_o^2}{\chi} \cdot \frac{e_1^2}{e_{o1}} - \sigma_{sy} A_{st}\Big] \Big[\frac{e_{ocr}\, \sigma_{sy}\, A_{st}\, e^A}{E\, e_o^2\, e_1\, \chi} - \frac{d_n}{\chi}\Big] \tag{11}$$

Case 4, Concrete Crushed but Steel Unyielded

$$d_n = d - \frac{E\, e_o^2}{2\, \chi^2\, E_s A_{st}} \log_e(e_{ocu}/e_{ocr}) \tag{12}$$

$$R_T = -\frac{2M}{\chi} - (d - d_n)^2\, E_s A_{st} \tag{13}$$

$$M = \frac{E\, e_o^2}{\chi^2} \Big[e_{cu} + e_{cr} - e_o \Big(\tan^{-1} \frac{e_{cr}}{e_o} + \tan^{-1} \frac{e_{cu}}{e_o}\Big)\Big] + (d - d_n)^2\, E_s A_{st} \chi \tag{14}$$

in which e_o is the concrete strain at peak stress σ_o; E and E_s are Young's modulii for concrete at zero strain and for steel, $e_{o1} = e_o^2 + e_1^2$, $e_{o3} = e_o^2 + e_3^2$, $e_{13} = e_1^2 + e_3^2$, $e'_{13} = e_1^2 - e_3^2$, $e_{ocr} = e_o^2 + e_{cr}^2$, $e'_{o13} = e_o^2 - e_1 e_3$, $e_{ocu} = e_o^2 + e_{cu}^2$, $e_{o1cr} = e_o^2 + e_1\, e_{cr}$, and $A = 2\sigma_{sy} A_{st}\, \chi/E\, e_o^2$.

The above expressions for d_n have to be solved iteratively for a given curvature determined in the current load stage. The correct flexural rigidities in the directions of orthotropy are thus determined for use in equation (5).

TEST EXAMPLES

To establish the efficiency and accuracy of the proposed method, the analytical results obtained by the authors were compared with three examples of the experimental/theoretical results obtained by other investigators.

The first example compared was the simply supported slab, S_1, tested by Taylor (7), which was 1525 mm square by 50.8 mm thick and had 0.55 per cent steel in both directions. The loads were applied through 16 points in the test. In Figure 4 the moment-curvature relationship obtained by the authors is compared with curves obtained using Branson's and Beeby's expressions. One quarter of this slab was also analysed using a 2×2 element grid and the experimental and analytical central deflection values are compared in Figure 5.

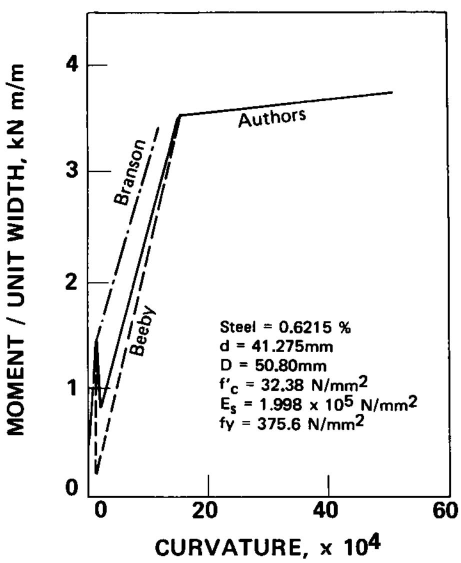

Figure 4 Moment Curvature Relationships

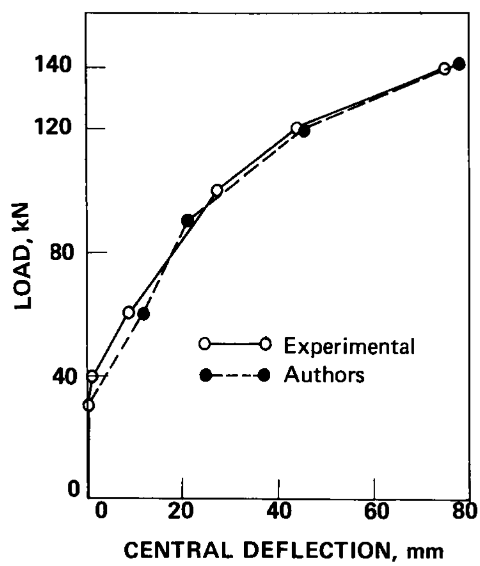

Figure 5 Comparison of Deflections for Taylor's Slab S1.

The crack pattern, crack penetration and yield lines as obtained by the analysis are shown in Figure 6; this information was not reported by Taylor.

As a second example, a comparison was carried out on a corner supported, 900 mm square by 43.75 mm thick, slab reinforced isotropically with 0.85 per cent steel mesh which was tested by McNeice (1) under a central load. The slab was analysed using a 6×6 element grid by McNeice and using a 6×6 element grid with 8 to 10 concrete layers and 2 steel layers by Lin and Scordelis (2) for the quarter slab. The central deflections obtained in the analysis carried out by the authors, using a single element over the entire quarter slab, are compared in Figure 7 with those obtained by the other analyses and those obtained experimentally. The crack patterns and yield lines as obtained analytically at various load levels are given in Figure 8.

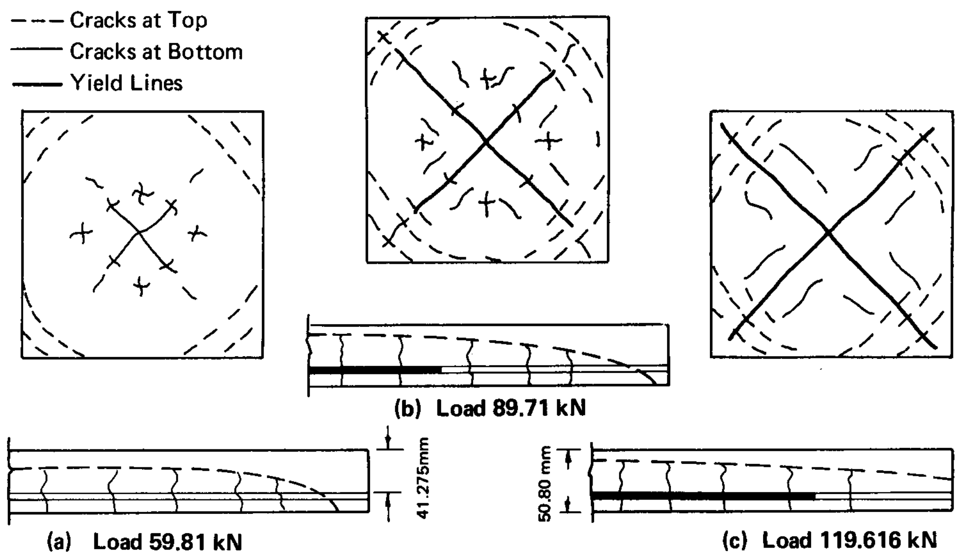

Figure 6 Crack Pattern, Crack Penetration and Yield Lines for Taylor's Slab S1

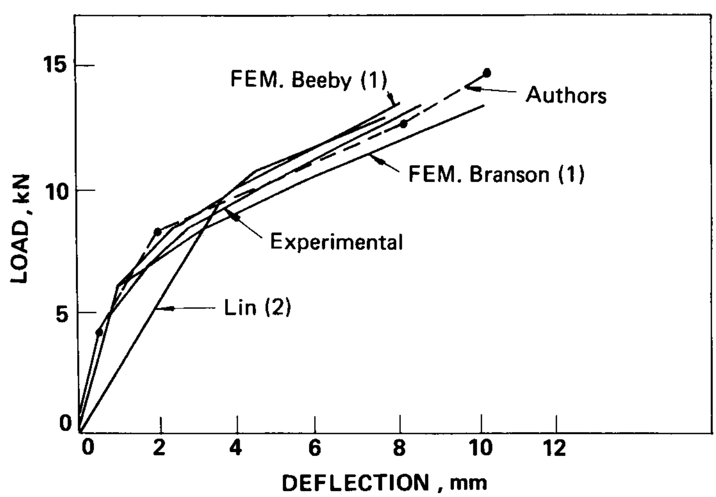

Figure 7 Comparison of Central Deflections for McNeice's Two-Way Slab

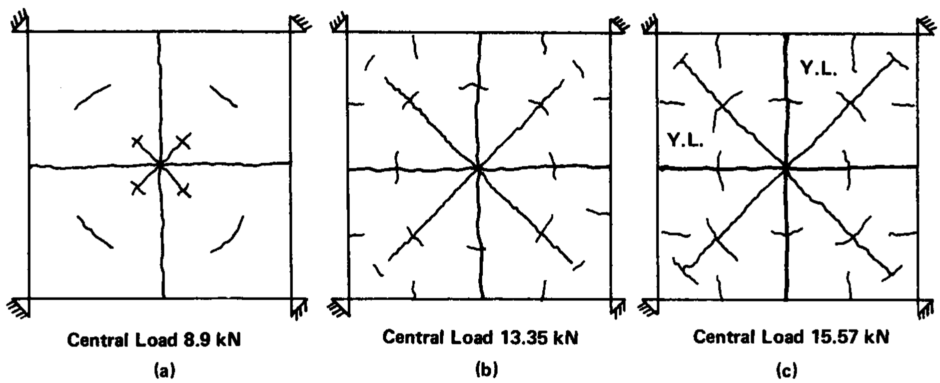

Figure 8 Prediction of Cracks and Yield Lines for McNeice's Two-Way Slab

The third example used was a 1525 mm square, 139.9 mm thick simply supported slab, B_1, reinforced with 0.5 per cent steel in both directions and tested under a single central load (8). The experimental load deflection curve is compared in Figure 9 with theoretical results obtained by the authors using a single element for the quarter slab.

The crack patterns, penetration of cracks and yield lines are shown in Figure 10; no experimental results were available for comparison of these parameters.

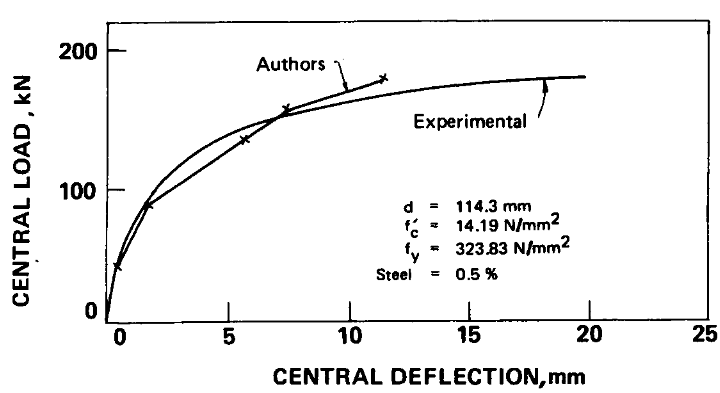

Figure 9 Comparison of Central Deflection for Slab B1

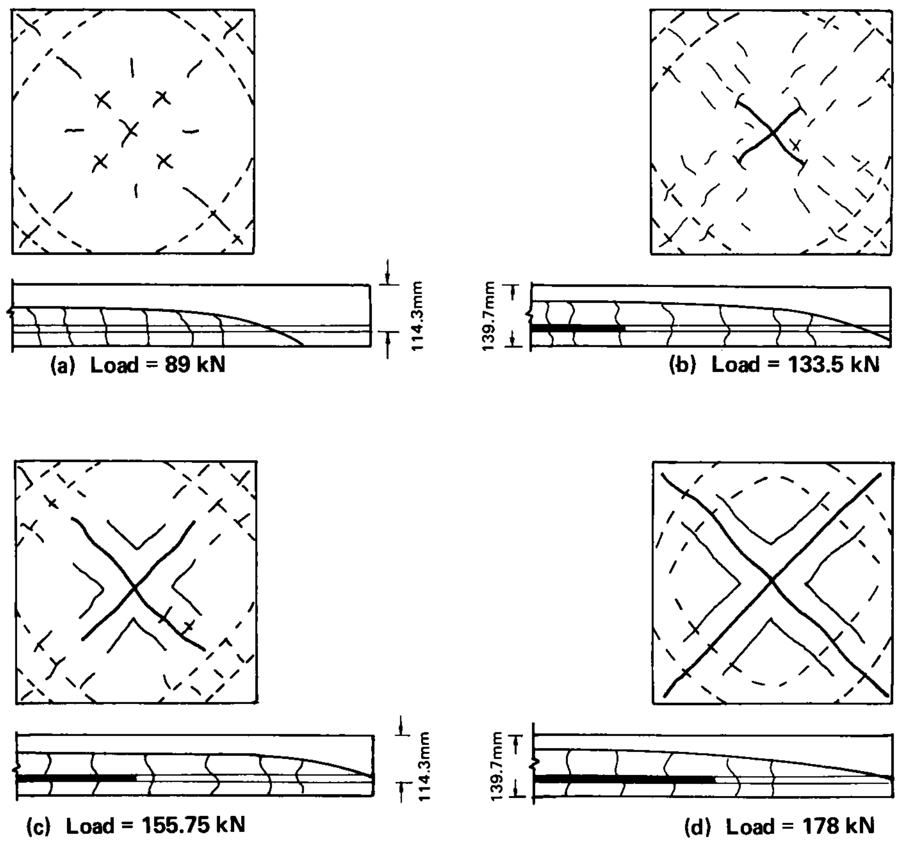

Figure 10 Crack Pattern and Yield Lines for Slab B1

CONCLUSIONS

The proposed procedure, which uses a more realistic moment curvature relationship as developed by the authors and a numerically integrated higher order element for discretization has been shown to correctly predict the post cracking behaviour of reinforced concrete slabs including crack patterns and yield lines. The formulation has been proved to be extremely efficient as regards computer storage and computation time, giving accurate results even with a very coarse mesh. The method has already been extended (6) for application to three dimensional plate structures such as folded plates, box girders and cylindrical shells.

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NONLINEAR FINITE ELEMENT ANALYSIS OF REINFORCED CONCRETE SLABS SUBJECTED TO TRANSIENT IMPULSIVE LOADING

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ABSTRACT The paper is concerned with the analysis of reinforced concrete slabs subjected to transient impulsive loading. Tensile cracking, the plastic behaviour of concrete in multiaxial state of stress and the plastic deformation of the reinforcement are taken into account. The effect of strain rate on the behaviour of concrete is also considered, as is geometrical nonlinearity. The equation of motion is derived using the principle of virtual displacements in total Lagrangian approach. The kinematic equations of plates are taken in accordance with the plate theories of Kirchhoff and Mindlin. The problem is discretized spatially and temporally by use of the finite element and finite difference methods, respectively. Some static and dynamic problems of slabs are analyzed and the results compared with experimental and numerical data obtained elsewhere.

INTRODUCTION

In problems of reactor safety and protective structures extreme dynamic loads can be encountered, and for safe and economic design the nonlinear behaviour of the material and the structure has to be considered. Major sources of nonlinearity in reinforced concrete structures are the progressive cracking in tension, the nonlinear response of reinforcing steel in tension and of concrete under compression, and other nonlinearities related to reinforcement and its interaction with concrete. In sudden loading the effect of a high strain rate on the concrete behaviour should also be accounted for.

The finite element method has enabled the solution of the complicated problems represented by the nonlinear behaviour of reinforced concrete structures and several studies on structures subjected to static loads have been published in recent years, see (1-11) and others. On the other hand, only few reports on dynamic analysis of reinforced concrete are available (12-14).

The purpose of this paper is to report on the study of reinforced concrete slabs subjected to transient impulsive loading. In the initial state, elastic behaviour is assumed for the composite material formed by concrete and reinforcement. Tensile cracking of the concrete and the plastic behaviour of the concrete in biaxial compression and of the reinforcing steel are taken into account. The reinforcement is described as smeared and orientated steel layers. The equation of motion is derived using the principle of virtual displacements in total Lagrangian approach, the kinematic equations of plates being taken in accordance with the theories of Kirchhoff and Mindlin. The problem is discretized spatially and temporally by use of the finite element and finite difference methods, respectively. Some static and dynamic problems of slabs are analyzed and the results compared with available experimental or numerical values.

KINETIC AND KINEMATIC EQUATIONS

In total Lagrangian approach the principle of virtual displacements can be written (15) in the form

$$\int_{V_o} S\delta E dV - \int_{V_o} f\delta u dV - \int_{A_t} t\delta u dA + \int_{V_o} \rho \ddot{u}\delta u dV = 0 \tag{1}$$

where S is the 2nd Piola-Kirchhoff stress, E the Green-Lagrange strain, f and t the prescribed body force and surface traction, respectively, ρ the density, u the displacement, and $\ddot{u}$ the acceleration. V_o denotes the initial volume of the body and A_t the part of the boundary on which the traction is given. The use of the finite element approximation u = Nq results in the matrix equation:

$$R(q) + M\ddot{q} = Q \tag{2}$$

where q is the vector of nodal displacements, and:

$$R = \int_{V_o} B^T S dV \tag{3}$$

$$Q = \int_{V_o} N^T f dV + \int_{A} N^T t dA \tag{4}$$

$$M = \int_{V_o} N^T \rho N dV \tag{5}$$

are the force of internal stresses, the load vector, and the mass matrix, respectively. The matrix B, dependent on the current state, is defined by the strain variation $\delta E = B\delta q$. The incremental form of the equation of motion, equation (2) is:

$$^1K_t \Delta q + M\,^2\ddot{q} = {}^2Q - {}^1R \tag{6}$$

where:

$$^1K_t = \int_{V_o} {}^1B^T\,{}^1D\,{}^1B dV \tag{7}$$

is the tangent stiffness matrix. Superscripts 1 and 2 refer to the configurations of the body at times t and t + Δt, respectively. The constitutive matrix D relates the stress and strain rates, $\dot{S} = D\dot{E}$.

In Mindlin's thick plate theory, the assumption is made that normals to the midplane remain straight but not necessarily normal to the midsurface after deformation. The nonlinear strain-displacement relation can be written (16) in the form:

$$\begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \end{bmatrix} = \begin{bmatrix} u_{,x}+w^2_{,x}/2 \\ v_{,y}+w^2_{,y}/2 \\ u_{,y}+v_{,x}+w_{,x}w_{,y} \\ w_{,x}+\phi \\ w_{,y}+\psi \end{bmatrix} + z \begin{bmatrix} \phi_{,x} \\ \psi_{,y} \\ \phi_{,y}+\psi_{,x} \\ 0 \\ 0 \end{bmatrix} \tag{8}$$

where u and v are the in-plane displacements, w the deflection of the midplane and ϕ and ψ represent the rotations of the normal with respect to the x- and y-axes. The corresponding relationships of the thin plate theory of Kármán-Kirchhoff are obtained by setting $\phi = -w_{,x}$, $\psi = -w_{,y}$ in equation (8).

SOLUTION TECHNIQUE

The central difference method (CD) is used to solve the system of ordinary differential equations represented by equation (2). The solution q_{n+1} at time t_{n+1} is computed from the formula:

$$q_{n+1} = h^2M^{-1}(Q_n - R_n) + 2q_n - q_{n-1} \tag{9}$$

where $h = (t_{n+1} - t_n)$ is the step length. The strain and stress increments are calculated using the equations:

$$\Delta E = B_n(q_{n+1} - q_n), \qquad \Delta S = D_n\Delta E$$

The internal force vector at time t_{n+1} is evaluated in accordance with equation (3) using B_{n+1} and $S_{n+1} = S_n + \Delta S$. The initial condition for $\dot{q}_o$ is used to eliminate q_{-1} in the first step $\dot{q}_o = (q_1 - q_{-1})/2\Delta t$. The CD-scheme is accurate and simple. As an explicit linear difference method its step length is limited by the largest natural frequency of the finite element mesh.

In the solution of equation (6) the trapezoidal rule or the Newmark scheme, with parameters $\gamma = 0.5$ and $\beta = 0.25$, was applied:

$$\dot{q}_{n+1} = \dot{q}_n + h\ddot{q}_n/2 + h\ddot{q}_{n+1}/2 \tag{10.1}$$

$$q_{n+1} = q_n + h\dot{q}_n + h^2\ddot{q}_n/4 + h^2\ddot{q}_{n+1}/4 \tag{10.2}$$

Use of these formulae results in an implicit scheme and therefore iteration has to be used at each time step. The displacement vectors q^i_{n+1} and q^{i+1}_{n+1} in the i^{th} and $(i+1)^{th}$ iteration cycles correspond to configurations 1 and 2 in equation (6). The use of equations (6) and (10.2) yields:

$$\tilde{K}\Delta q^{i+1} = \tilde{Q} \tag{11}$$

where $\Delta q^{i+1} = q^{i+1}_{n+1} - q^{i}_{n+1}$ and:

$$\tilde{K} = K^i_{t,n+1} + 4M/h^2 \tag{12.1}$$

$$\tilde{Q} = Q_{n+1} - R^i_{n+1} + M[-4(q^i_{n+1} - q_n)/h^2 + 4\dot{q}_n/h + \ddot{q}_n] \tag{12.2}$$

For the first iteration cycle, q^1_{n+1} is taken equal to q_n. The iteration is continued until:

$$\|\Delta q^{i+1}\| < \varepsilon \, \|q^{i+1}_{n+1} - q_n\|$$

where ε is a tolerance parameter. To account for the drastic changes due to cracking the tangent stiffness K_t was updated in three first iteration cycles but held constant thereafter in order to reduce computing time.

In Kirchhoff's plate theory a rectangular element with 24 degrees of freedom was used with 4 nodes and the displacement parameters u, v, w, $w_{,x}$, $w_{,y}$, and $w_{,xy}$ at each node. Integration was performed by a 2x2 Gauss quadrature formula. In Mindlin's plate theory rectangular elements with 20 and 40 degrees of freedom were employed with 4 and 8 nodes, respectively. The displacement parameters at each node were u, v, w, ϕ, and ψ. A 2x2 Gaussian integration rule was used with the 40 degree of freedom element; in the case of the 20 degrees of freedom element only one Gaussian point for the shear deformation was used. In the depth direction the integration was carried out by Simpson's rule with 7 integration points. Steel layers were considered separately at their proper places and their effects added to the internal force vector and the tangent stiffness.

MATERIAL PROPERTIES

Concrete

The behaviour of concrete is illustrated by the uniaxial stress-strain diagram shown in Figure 1.

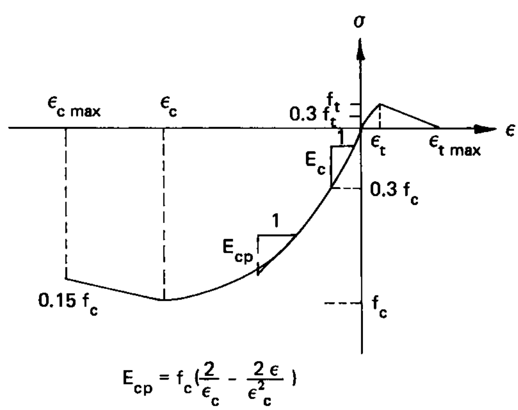

Figure 1 Uniaxial Stress-Strain Curve of Concrete

At the initial stage, concrete is linearly elastic and isotropic up to the level of 30 per cent of the compressive strength and of the tensile strength. Then plastic strain hardening yield takes place according to a parabolic function and cracking occurs when a cracking criterion is satisfied. Cracking is brittle and the stress at a discrete crack drops to zero at once. The slowly descending part of the curve after cracking in Figure 1, however, describes average cracking behaviour over a finite distance and gradual release of tensile stress. In compression, strain softening occurs after the compressive strength in two linear parts, of which the latter corresponds to the crushing region.

In multiaxial state of stress the yield criterion is defined as follows:

$$f(\sigma, \varepsilon^p) = \sqrt{3\alpha J_2 + \beta\bar{\sigma}J_1 + \gamma J_1^2} - \bar{\sigma} = 0 \qquad (13)$$

where J_2 is the second invariant of stress deviator, $J_2 = \sigma'_{ij}\sigma'_{ij}/2$, J_1 the first invariant of stress tensor, $J_1 = \sigma_{kk}$, and α, β, and γ are parameters to be determined on the basis of experimental data. The equivalent yield stress $\bar{\sigma}$ is a function of the equivalent plastic strain:

$$\bar{\sigma} = \bar{\sigma}(\varepsilon^p), \quad d\bar{\sigma}/d\varepsilon^p = E_p(\varepsilon^p) \qquad (14)$$

This can be obtained from the uniaxial diagram by subtracting the elastic strains. The initial yield stress in equation (13) is $\bar{\sigma} = 0.3\ f_c$. The yield locus expands when $\bar{\sigma}$ increases and its ultimate size is reached when $\bar{\sigma} = f_c$. Parameters α, β, and γ are determined at this stage to fit the experimental results for the biaxial stress state given in reference 17. Using values of $\sigma_2/\sigma_1 = 0$, $\sigma_1 = -f_c$; $\sigma_2/\sigma_1 = 0.5$, $\sigma_1 = -1.2\ f_c$; $\sigma_2/\sigma_1 = 1$, $\sigma_1 = -1.16\ f_c$ one finds for the compression-compression region that $\alpha = 1.93$, $\beta = 1.18$ and $\gamma = 0.25$, and with values $\sigma_2/\sigma_1 = 0$, $\sigma_1 = -f_c$, $\sigma_2/\sigma_1 = -10$, $\sigma_1 = -0.5\ f_c$, $\sigma_2/\sigma_1 = 1$, $\sigma_1 = f_t = 0.1\ f_c$ for the tension-compression and tension-tension regions, $\alpha = 40.6$, $\beta = 9.07$, $\gamma = -30.5$. In tension-compression and tension-tension regions the ultimate yield locus serves as the cracking criterion. Beyond the ultimate yield locus, the softening begins and the yield locus contracts, see Figure 2.

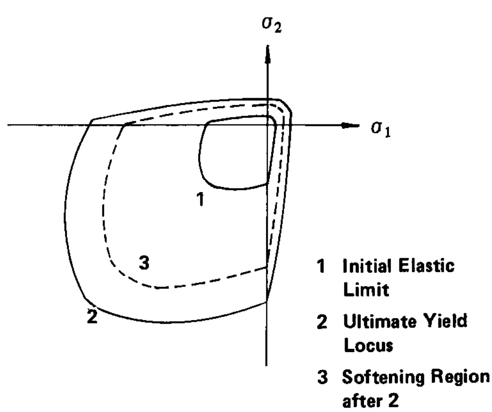

Figure 2 Biaxial Yield and Failure Criteria of Concrete

The usual associated flow rule is employed. The constitutive equation in plastic flow is accordingly:

$$\dot{\sigma} = D_{ep}\,\dot{\varepsilon} \tag{15}$$

where the elastic-plastic constitutive matrix is:

$$D = C - \frac{C\,n\,n^T\,C}{\left(1 - \frac{\beta J_1}{2\bar{\sigma}}\right)^2 E_p + n^T C n} \tag{16}$$

$$n \equiv \frac{\partial f}{\partial \sigma}$$

and C is the elasticity matrix of concrete.

The direction of the crack is taken perpendicular to the maximum tensile stress. After complete cracking concrete behaves uniaxially in the crack direction. The stress and strain have to be transformed from the crack direction to the global direction using the formulae:

$$\sigma_c = T\sigma, \quad \varepsilon_c = (T^{-1})^T\varepsilon \tag{17}$$

where T is the transformation matrix

$$T = \begin{bmatrix} \cos^2\alpha & \sin^2\alpha & 2\cos\alpha\sin\alpha \\ \sin^2\alpha & \cos^2\alpha & -2\cos\alpha\sin\alpha \\ -\cos\alpha\sin\alpha & \cos\alpha\sin\alpha & \cos^2\alpha-\sin^2\alpha \end{bmatrix} \tag{18}$$

After cracking some shear resistance still exists due to aggregate interlocking and dowel action. This is taken into account in a reduced shear modulus of concrete (9) as:

$$G_{red} = \left[\left(1 - \frac{\varepsilon}{\varepsilon_{tmax}}\right) . 0.6 + 0.4\right]G, \quad \varepsilon_t < \varepsilon < \varepsilon_{tmax} \tag{19}$$

$$G_{red} = 0.4G, \quad \varepsilon_{tmax} < \varepsilon$$

The crack is assumed to close when the strain perpendicular to the crack direction becomes compressive.

Reinforcement

For the steel bars the elastic plastic, linearly strain hardening idealization is used. Reinforcement takes axial and shear stresses.

Interaction Between Concrete and Reinforcement

Up to cracking, complete compatibility is maintained between concrete and reinforcement, the reinforcement being described as smeared uniaxial layers. After some cracking the deformations of concrete and reinforcement are determined by the same set of shape functions, while after complete cracking, concrete behaves uniaxially. The effect of bond slip is taken into account by reducing the modulus of elasticity of the reinforcement by 10 to 20 per cent.

Effect of Strain Rate

Experimental evidence indicates that the modulus of elasticity and compressive and tensile strengths of concrete can be considerably increased by a high strain rate. An increase of the yield limit of low carbon steel has also been observed and this effect can be taken into account using the viscoplastic constitutive equation (18):

$$\dot{\varepsilon}^{vp} = k \langle f \rangle \frac{\partial f}{\partial \sigma} \tag{20}$$

where $\langle f \rangle = f$ if $f > 0$ and 0 otherwise, and k is an experimental viscosity coefficient. For the elastic part of strain rate Hooke's law is valid.

NUMERICAL EXAMPLES

Static Case

The corner supported two-way slab shown in Figure 3 which is 36 in. square by 1.75 in. thick with an isotropic mesh of 0.85 per cent reinforcing steel and was tested under a central point load by McNeice (7), was analyzed to verify the proposed method of analysis and the reliability of the computer program; 2x2 and 3x3 finite element meshes for slab quadrant were used. The numerical load-deflection curve agrees favourably with the experimental data, see Figure 3.

Dynamic Case

A clamped rectangular slab subjected to a jet force at the centre is a structure analyzed by Stangenberg (12) using a difference method, see Figure 4. Finite element meshes 2x2 and 3x3 for plate quadrant were used. The calculated time history of the deflection of the central point agrees closely with the curve given in reference 12.

Discussion

The numerical results obtained for the cases considered indicate that the method describes satisfactorily the behaviour of reinforced concrete slabs, at least under static loading. Experimental evidence is needed for dynamic loading. The development of the proposed model is in progress. A systematic study of the factors, viz. cracking, elastic-plastic yield and strain rate effect of concrete, aggregate interlocking, dowel action, etc., affecting the behaviour of reinforced concrete structures and attempts to find realistic simplified models are continuing.

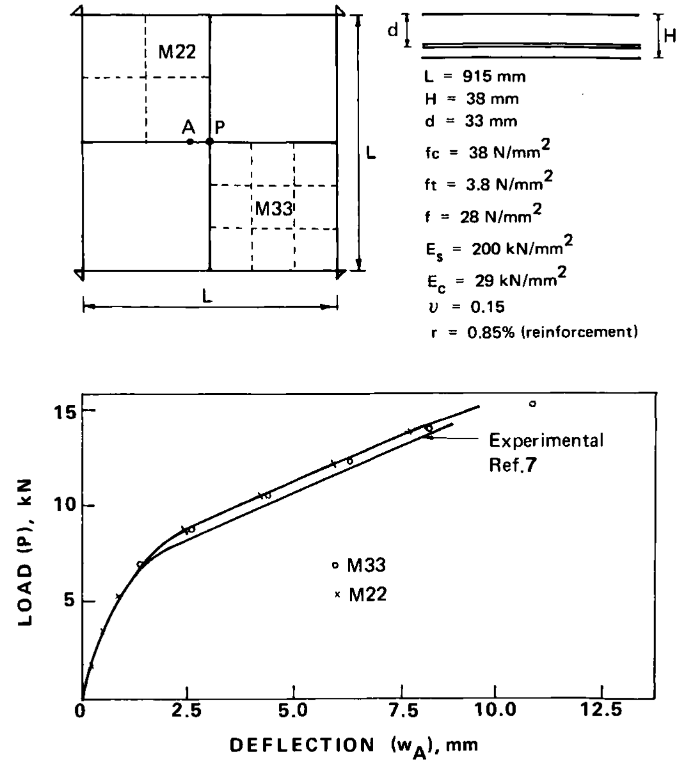

Figure 3 Load-Deflection Curve of Corner Supported Square Slab under Central Point Load

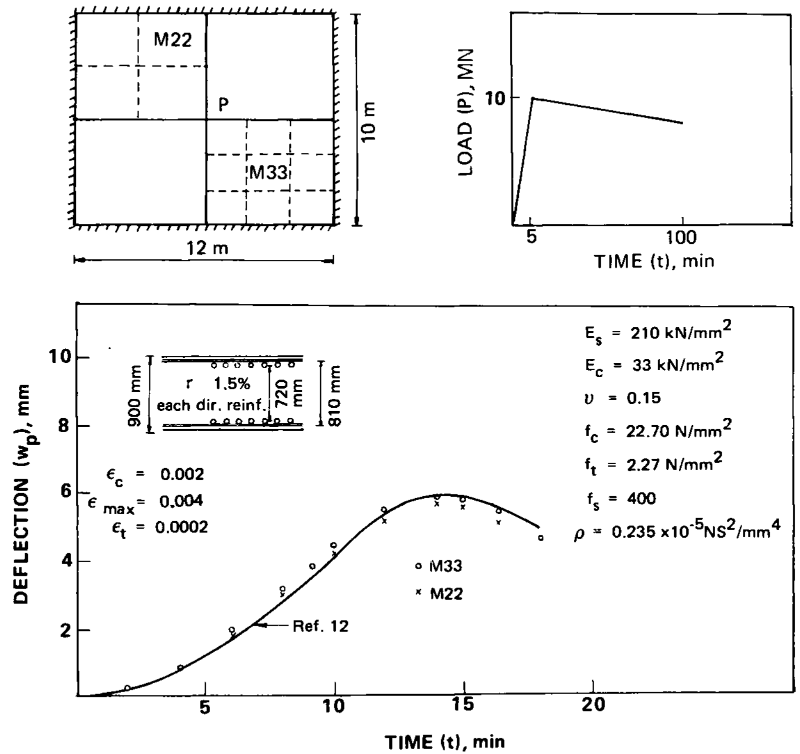

Figure 4 Deflection Time History of Rectangular Plate under Central Jet Force

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YIELD LINE ANALYSIS OF ORTHOTROPICALLY REINFORCED EXTERIOR PANELS OF FLAT SLAB FLOORS

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ABSTRACT The flexural strength of exterior panels of flat slab structures was investigated by use of Yield Line Theory. Three governing, geometrically admissible, collapse mechanisms were identified, two of which involved local flexural failures around the supporting columns. Orthotropic arrangement of the reinforcement and variations in the negative to positive yield moment ratio were considered. A set of graphs is presented which can be used for the analysis or design of exterior slab panels supported on circular or square columns, the graphs covering practical ranges of the important physical and geometrical parameters.

INTRODUCTION

The strength of exterior panels of flat slab floors depends to a considerable degree on the flexural strength of the slab-column joints. This strength must therefore be considered in the design of the slab. The shear strength of such joints may also play a large role, but it has been much investigated (3) and design for shear strength has long been codified (1, 2); design for shear therefore is not considered here.

To simplify the problem, only uniformly distributed vertical loads applied to the slab are considered and the structure is assumed to have neither spandrel beams nor drop panels. The effects of orthotropic arrangement of the reinforcement and varying positive to negative reinforcement ratios is considered, however, and, as shown in Figure 1, the slab may overhang the exterior column line.

Since reinforced concrete design is customarily (1) based on the ultimate strength of members, it is logically appropriate to employ an inelastic method of analysis, i.e. the Yield Line Theory (14). This method also readily lends itself to the development of relatively simple design procedures, leading to practical reinforcement arrangements.

A large body of literature exists on the shear strength of slabs (3). Some of the work (10, 12, 13, 15, 17, 18, 19) has dealt with the slab joint strength at boundary columns, but invariably from the point of view of shear. Flexural effects, when considered, were only assumed to influence the shear strength of the slab. Pictures of a number of these tests (10, 13, 17, 18, 19), however, show clear evidence of flexural distress in the specimens prior to collapse.

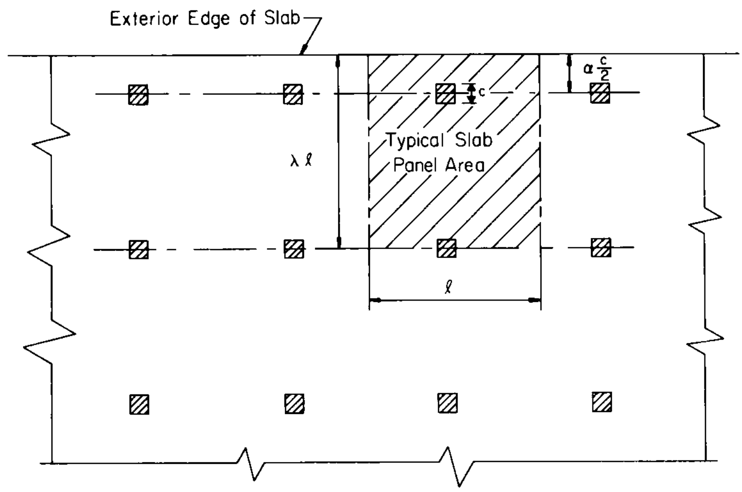

Figure 1 Typical Floor Plan

Yield line theory has previously been applied to the slab-column joint problem at interior columns (5, 6, 7, 16) and at exterior columns (5, 16). Unfortunately, only one of many possible flexural collapse mechanisms was examined in the latter two investigations, leading to predicted joint strengths greater than those which are obtained from other yield line patterns, including those to be discussed below.

FLEXURAL COLLAPSE MECHANISMS

Yield Line Theory Requirements

In the yield line theory it is assumed that the slab will exhibit rigid-plastic behaviour. Overloading thus leads to the formation of yield hinges in patterns that must form geometrically admissible collapse mechanisms. Force or virtual energy equilibrium equations are then written for each possible mechanism and, since yield line theory is an upper bound theory, the mechanism giving the lowest load carrying capacity is the correct one.

Three different flexural collapse mechanisms for exterior slab panels were identified in published reports of experimental work (10, 13, 17) and small scale supplementary tests (8). They are shown in Figure 2 for circular columns, but square and rectangular column mechanisms are very similar. The mechanisms are described below.

Overall Flexural Failure

This mechanism is the three dimensional version of the continuous beam mechanism. A number of variations are possible. The only geometrically admissible one under rigid-plastic assumptions, which is shown in Figure 2(a), requires that a straight negative yield line form parallel to the free edge of the slab and tangent to the inside faces of the columns. Small columns and small overhangs outside the columns may prevent formation of this yield line due to column rotation, or elastic twisting of the slab, or both. Since rigid-plastic material and slab behaviour were assumed in this study, only the overall flexural failure model shown in Figure 2(a) was analyzed. However, it is a simple matter to carry out a beam type analysis assuming a small or even zero resisting moment along the columns.

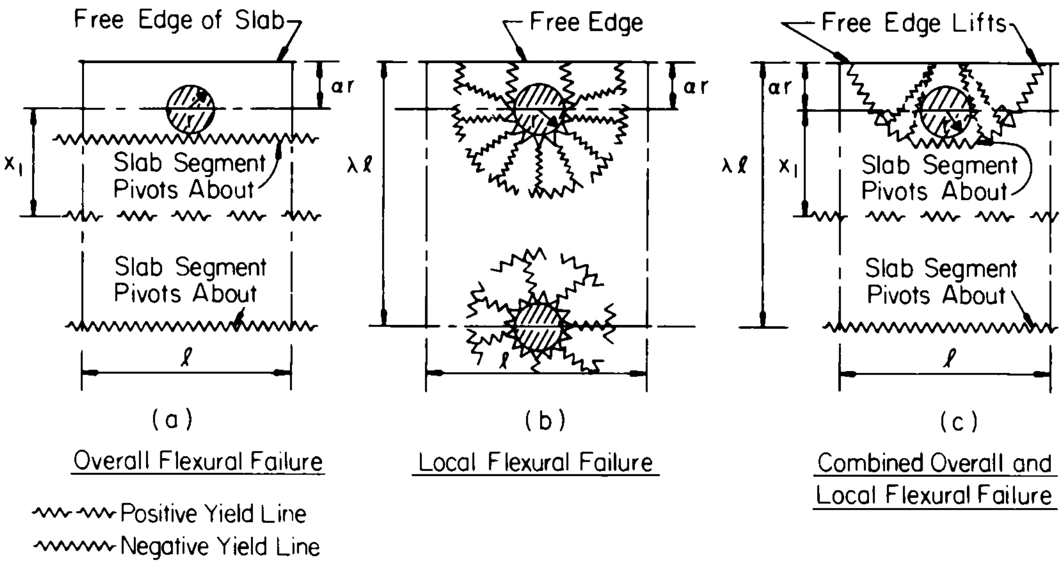

Figure 2 Flexural Collapse Mechanisms

It is also possible for the portion of slab outside the column line to break off in cantilever bending along a yield line tangent to the exterior faces of the columns. This possibility was considered in the calculations. It only governs when α in Figure 2 becomes rather large and the negative moment capacity of the slab in the cantilever bending direction is rather small.

It was assumed, in this mechanism, that the interior columns would not punch through the slab prior to failure of the exterior panel (9, 11). Therefore, to simplify analysis, a rigid support with a negative yield line running along it was substituted for the interior columns. This assumption would have to be checked in the course of a design, using expressions developed previously (5, 6, 16).

Local Flexural Failure

In this mechanism, shown in Figure 2(b), yield line patterns form around all columns simultaneously, permitting the slab outside the yield fans to drop as a rigid body. It can accurately be described as a "flexural punching" failure. In a real structure one or more columns will typically punch through before the others (10, 13, 18), since the slab is not really rigid. In the mathematical analyses of this mechanism only the exterior column was considered and the area of slab supported by the column was assumed to extend to midway between the interior and exterior columns. This was in accordance with the calculated locations of the positive moment yield lines in the other two collapse mechanisms shown in Figure 2.

Combined Overall and Local Flexural Failure

The third geometrically admissable collapse mechanism is shown in Figure 2(c). It appeared in experiments (8) and gave lower load carrying capacities than the other two for some combinations of the physical and geometrical parameters (see also references 9, 10, 13, and 17). In this mechanism the outer portion of the slab pivots about a yield line tangent to the inner face of the exterior column, thereby lifting the free edge of the slab and causing the yield line pattern shown. As in the overall flexural mechanism, a rigid support with a negative yield line was substituted for the interior columns to simplify the analysis.

Other Possible Mechanisms

A large number of other geometrically admissible mechanisms, as well as variations of the above, were investigated (8), but one of the above always gave the lowest load carrying capacity. One possible set of failure modes, however, was ignored. This is the set which would have involved the formation of straight yield lines perpendicular to the ones shown in Figures 2(a) and 2(c), i.e. failures perpendicular to the free edge. It was felt that these could not properly be considered to be exterior panel failures, but rather would be parts of interior panel failures, and could be investigated as such (5, 16).

METHOD OF ANALYSIS

Yield Line Theory Notation

Johansen's (14) stepped yield criterion was used in the analysis, with the yield moment notation shown in Figure 3. Top reinforcing bars perpendicular to the free edge of the slab were assumed to provide negative yield moment m per unit length of slab, bottom bars perpendicular to the free edge were assumed to provide positive yield moment K_1m per unit length of slab, top bars parallel to the free edge were assumed to provide negative yield moment $K_1'm$ per unit length, and bottom bars parallel to the free edge were assumed to provide positive yield moment $K'm$ per unit length. These yield moment capacities were assumed to be constant throughout the exterior slab panel.

A coefficient of orthotropy may be defined as $\mu = (K_1' + K')/(1 + K_1)$ and, letting $\Sigma K = K_1 + K' + K_1'$, the total moment resistance can be defined as $m(1 + \Sigma K)$. For simplicity in calculations, K_1' was set equal to μ, which leads to $K_1'/K' = 1/K_1$, i.e. the negative to positive moment ratio becomes the same in both principal directions.

Equation of Equilibrium

Using the notation of Figures 1, 2 and 3, letting w equal the ultimate load per unit area of slab, letting T_{SC} be the ratio of slab panel area to the cross sectional area of the exterior column (e.g. $T_{SC} = \lambda\ell^2/\pi r^2$ for circular columns), and letting a number of x_i be the unknown distances defining the geometry of the collapse mechanisms, it is possible to write equations of virtual energy equilibrium relating the displacements of the applied loads to the yield line rotations. These equations will have the form:

$$\frac{w\lambda\ell^2}{m(1 + \Sigma K)} = f(\lambda,\ \alpha,\ K_1,\ \mu,\ T_{sc},\ x_i/\ell) \tag{1}$$

The function, f, will be different for each collapse mechanism.

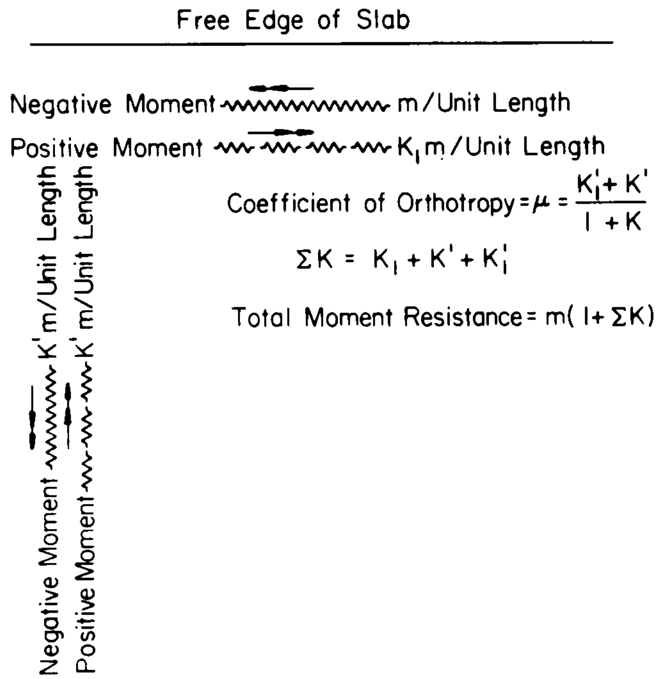

Figure 3 Orthotropic Yield Moment Notation

The dimensions x_i defining the geometry of each yield line pattern must be evaluated by minimizing the load parameter $w\lambda\ell^2/m(1 + \Sigma K)$ with respect to each x_i. Theoretically this could be done by partial differentiation and equating to zero. However, the equilibrium equations were too complex to make this practical. Instead, computer algorithms were written (4, 8) to iterate the various forms of equation (1) through successive values of the x_i until the minimum value of the load parameter was obtained.

For each set of physical and geometrical parameters examined, the above calculations were carried out for each of the collapse mechanisms. The lowest value of the load parameter was assumed to determine the governing mechanism and to be the correct load capacity.

RESULTS OF ANALYSES

Presentation of Selected Results

The results of the calculations were plotted in two forms. Letting $P = w(x_1 + \alpha r)\ell$ of Figure 2, i.e. the load transferred from the slab into the

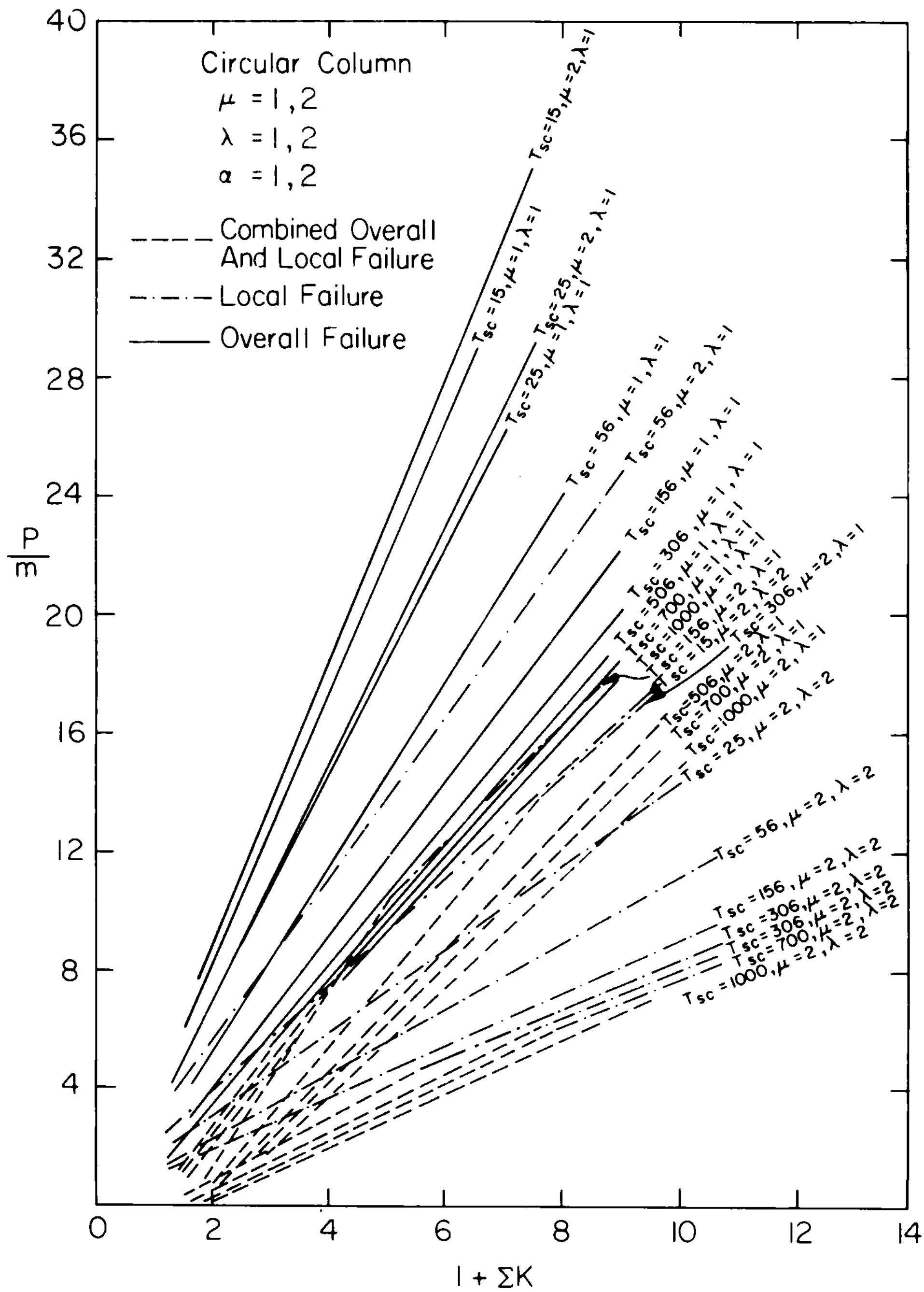

Figure 4 Flexural Strength of Exterior Panels

column, graphs were plotted of P/m versus 1 + ΣK, a typical set of which is shown in Figure 4. It should be noted that in the case of purely local flexural failure the value of x_1 cannot be obtained directly. Based on results obtained from the other two mechanisms, x_1 was therefore assumed equal to $\frac{1}{2}(\lambda\ell - \alpha r)$.

Two findings of particular interest in Figure 4 are that i) the collapse mechanism may change with change in 1 + ΣK, all other parameters remaining constant, and ii) the relationship between P/m and 1 + ΣK is linear except at points of change from one mechanism to another. This linearity turned out to be quite general, and reduces the number of calculations required for reasonable coverage of a given structural system to manageable proportions.

While Figure 4 and other graphs similar to it (8) provide much information of interest to the researcher, the practical designer will find the graphs of Figure 5 more useful.

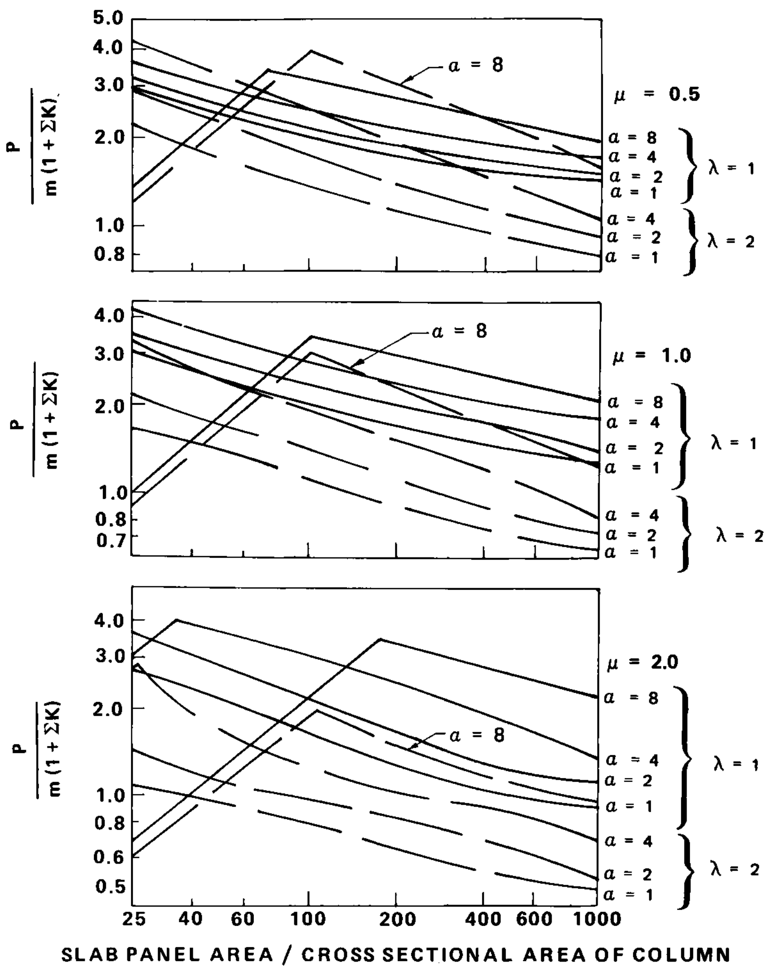

Figure 5 Flexural Strength of Exterior Flat Slab Panels (Circular Columns, K_1 = 0.5)

These graphs show the relationship between load carrying capacity and the ratio of slab panel area to column cross sectional area, or, more formally, the relationship between $P/m(1 + \Sigma K)$ and $\lambda \ell^2/\pi r^2$, for a variety of values of the other parameters. It should be noted that the solid lines are used for the cases where $\lambda = 1$ and the dashed lines for $\lambda = 2$ and that the straight lines sloping downward to the left represent cantilever failures. Although these graphs were drawn for slabs supported on circular columns and for only the case of $K_1' = \mu$ and $K_1 = 0.5$, they are actually quite widely useful, as further described below.

Comparison with Experimental Results

Results of experiments conducted by Harnden (10) and Hemakom (13) were analyzed by yield line theory, using the collapse mechanisms described previously. It was found (8) that the collapse loads predicted by flexural theory agreed very closely with the experimentally obtained loads, and that previously published failure patterns (9, 10, 11, 13, and 18) were quite similar to those shown in Figure 2.

Additional Observations

Studies of Figure 5 and of the results of calculations for slabs supported on square columns and with other values of K_1 (8) lead to the following observations.

1. For T_{sc} (i.e. slab panel area/column cross sectional area) > 100 the difference in collapse load between circular and square columns with the same physical and geometrical parameters was < 10 per cent in all cases examined.

2. With $K_1' = \mu$, varying only K_1, i.e. the ratio between positive and negative yield moments (equal in both principal directions), within the reasonable limits $0.33 \leqslant K_1 \leqslant 1.0$ caused $P/m(1 + \Sigma K)$ to vary in magnitude from 10 per cent less to 20 per cent greater than the values shown in Figure 5.

3. The position of the positive moment yield line, i.e. the magnitude of x_1, was calculated in the overall flexural, the combined flexural and the local collapse modes. The results showed that, for circular columns with $T_{sc} \geqslant 100$, $0.45(\lambda\ell - \alpha r) \leqslant x_1 \leqslant 0.50(\lambda\ell - \alpha r)$. Similar results were obtained for square columns (8).

4. As can be seen from Figure 5, changes in α have a reasonably small effect on $P/m(1 + \Sigma K)$. Thus linear interpolation for α between plotted values will be sufficiently accurate for design.

5. As α becomes very large, the cantilever failure mode becomes dominant. However, it will also then be necessary to check that an interior column collapse mechanism (6, 16) does not form and govern the failure around the exterior column.

6. Figure 5 also shows that $P/m(1 + \Sigma K)$ is approximately inversely proportional to λ, i.e. the values for $\lambda = 2$ are approximately ½ the values for $\lambda < 1$. Thus linear

interpolation should be valid for $1 < \lambda < 2$. Extrapolation, particularly for $\lambda < 1$, should be approached cautiously because a transverse flexural collapse mechanism may become dominant.

7. Figure 4 shows that μ has a great effect in determining the collapse mechanism and Figure 5 shows that consequently its effect on $P/m(1 + \Sigma K)$ is too large to permit accurate interpolation between plotted values.

8. The reasons for apparent inconsistencies in the graphs of Figure 5 for $T_{SC} < 100$ are changes in collapse mechanism with changes in T_{SC}, see Figure 4.

Interpretation and Application to Design

The observations outlined above lead to the following conslusions which are of use to the practising designer.

1. Figure 5 can safely be used for the design of exterior panels of slabs supported on circular or square columns, provided T_{SC} and K_1 are within the ranges which would be expected in normal construction.

2. For design purposes, the positive moment yield line, or line of zero shear, can safely be assumed to occur midway between the column center lines for all three collapse mechanisms. Thus, knowing the distributed load, w, per unit area, the designer can assume that $P = \frac{1}{2} w\ell(\lambda\ell + \alpha r)$ for circular columns or $P = \frac{1}{2} w\ell(\lambda\ell + \frac{1}{2} \alpha c)$ for square columns. This location can also be used to calculate, by static equilibrium, the bending moment which will be transferred into the column at ultimate load.

3. Linear interpolation is permissible between the plotted values of λ and α. Extrapolation should be approached very cautiously. Interpolation between the values of μ is not permissible.

CONCLUSIONS

Within the limitations imposed by the simplifying assumptions made herein, the ultimate flexural strength of exterior panels of orthotropically reinforced flat slabs, supported on circular or square solumns, can be predicted with good accuracy by use of the graphs given in Figure 5. These graphs can also readily be used for design. The analytical method has been found to correlate well with test results reported in published literature.

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EXAMPLE

An exterior panel of a flat slab structure 7.5 m x 7.5 m in size is supported on square exterior columns 0.6 m x 0.6 m in cross section. The column faces are assumed to be set back 0.3 m from the slab edge. The service live load is 400 kg/m^2 and the slab thickness is specified to be 230 mm in order to control deflections. Using ACI Code (1) load factors, the total ultimate load will then be 400 x 1.7 + 0.23 x 2400 x 1.4 = 1453 kg/m^2.

If it is assumed that the positive yield line forms midway between column center lines, then $P = \frac{1}{2} \times 1453 \times 7.5 \times (7.5 + 0.6) \times 9.8 = 433$ kN. Since the slab panel is square, the same amount of reinforcement will be used in both directions. The negative moment strength of the slab will arbitrarily be made twice the positive moment strength for crack control purposes.

The above dimensions lead to $T_{sc} = 156$, $\alpha = 2$, $\lambda = 1$, $K_1 = 0.5$, $\mu = 1$, $K_1' = 1$, and $K' = 0.5$. Then $1 + \Sigma K = 3$. From Figure 5, $P/[m(1 + \Sigma K)] = 2.1$ or the required $m(1 + \Sigma K) = 206$ kNm/m. It should be noted that Figure 5 was drawn for circular columns; for comparison, the correct value for a square column (8), as calculated by minimizing equation (1) with $T_{sc} = 156$, is $P/[m(1 + \Sigma K)] = 2.3$, which leads to a total required moment capacity of 190 kNm/m. Evidently, the use of the circular column graphs leads to a small error on the safe side.

If K_1 had been chosen to be equal to 1 with all other parameters the same as above, the analytical procedure gives $P/[m(1 + \Sigma K)] = 2.2$ for circular columns and 2.4 for square columns. A choice of $K_1 = 0.33$ leads to $P/[m(1 + \Sigma K)] = 2.0$ for circular and 2.1 for square columns. Again, the use of the graphs of Figure 5 leads to very small errors and, considering all other uncertainties in the design of reinforced concrete structures, is well justified.

Choice of a much smaller column, say 1/3 m x 1/3 m in cross section, gives Tsc = 506, and for $K_1 = 0.5$ and $\alpha = 2$, $P/[m(1 + \Sigma K)] \simeq 1.6$ from Figure 5. The correct value for a square column is also 1.6. For $K_1 = 1$ the correct values are 1.8 for both the circular and the square columns, and for $K_1 = 0.33$ they are both 1.5.

If the physical size of the overhang had been kept constant while the column size was reduced, α would have become equal to 3.6, requiring interpolation between the $\alpha = 2$ and the $\alpha = 4$ curves. For $\alpha = 2$, $P/[m(1 + \Sigma K)] = 1.6$ and for $\alpha = 4$, $P/[m(1 + \Sigma K)] = 2.0$. The interpolated value would be approximately 1.9, and computer analysis shows this to be correct.

A DETAILED YIELD-LINE STUDY OF UNIFORMLY LOADED, COLUMN-SUPPORTED INTERIOR SLAB PANELS

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ABSTRACT A fairly thorough yield-line study of interior slab panels, which attempts to fill some of the gaps left by previous investigations, is reported. Two basic shapes of column, square (or rectangular) and circular, are considered in the work. Four basic yield-line mechanisms are investigated. The main variables in the investigation are the size of the columns and the strength of the beams, although the influence of the width of the beams is also commented upon. The effects of these variables in changing the critical yield-line pattern, and in altering the required slab and beam moments of resistance are reported.

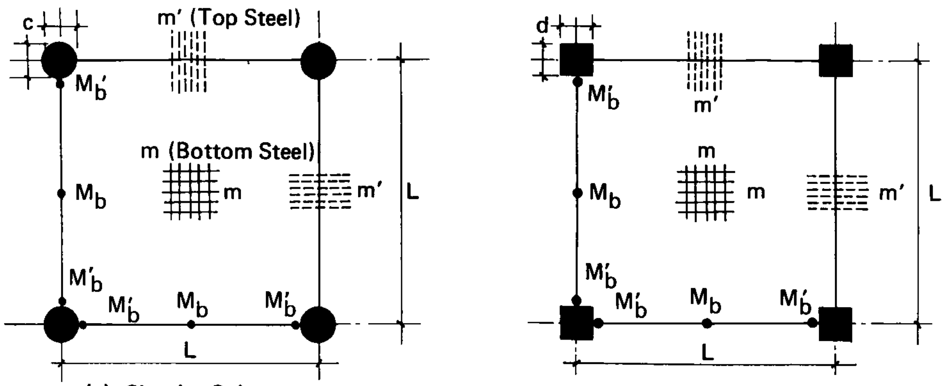
INTRODUCTION

Yield-line investigations of interior slab panels have been reported by many authors (e.g. ref. 1 to 5). However these have all, even taken together, been of quite restricted scope. Those studies of beam and slab panels which have been presented have involved only point columns and line beams and have not included column fan yield-line patterns amongst their failure mechanisms. Those studies of flat plates (i.e. beamless slabs, without drops, on column supports) which are known to the authors deal only with point and circular columns, and failure mechanisms of the type illustrated in Figure 4(a) have not been considered.

The purpose of this paper, therefore, is to fill some of the gaps left by previous investigations.

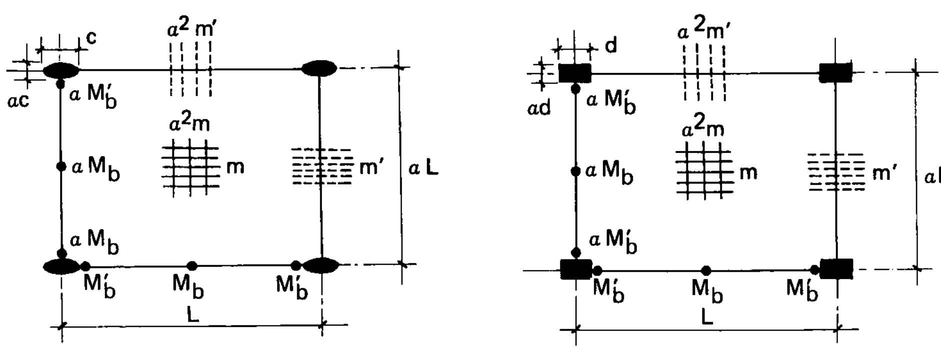
The two panels shown in Figure 1 are solved for the yield-line mechanisms illustrated in Figures 3, 4 and 5; the solutions obtained are also applicable to the affine panels shown in Figure 2(b). The influence of the strength of the beams, and the size and shape of the columns, upon the moments of resistance required to resist the various modes, and their effects in changing the critical failure mechanism (that which gives the lowest failure load) are revealed.

The influence of the finite width of the beams in modifying the line-beam solutions, as illustrated in Figure 6, and in adjusting the trends disclosed by them, is discussed; as is the suitability of using the line-beam solutions to design practical structures.



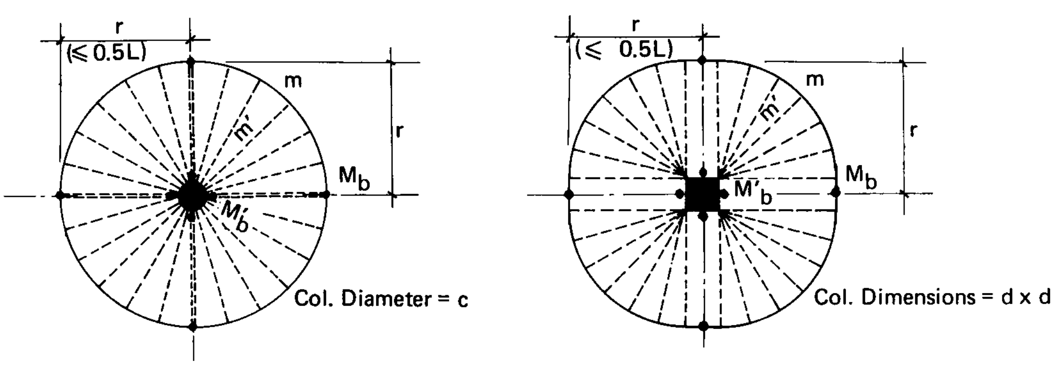
(a) Circular Columns (b) Square Columns

Figure 1 Basic Interior Panel Cases



(a) Rectangular Panel with Eliptical Columns (b) Rectangular Panel with Rectangular Columns

Figure 2 Affine Cases



(a) Fan Around Circular Column (b) Fan Around Square Column

Figure 3 Fans around Interior Columns

NOTATION

α	the ratio of the lengths of the sides of a panel
b	the width of the beams in the direction under consideration
c	the diameter of a circular, or elliptical, column head
d	the side of a square, or rectangular, column head
K	an important parameter which denotes the ratio between the strength of a beam and the strength of the slab in the same direction, generally = $(M_b + M_b')/(m + m').\alpha L$, or, for a square panel, = $(M_b + M_b')/(m + m').L$
L	the distance between column centre-lines in one, or both, directions
m, m'	slab positive and negative moments of resistance, respectively
M_b, M_b'	beam positive and negative moments of resistance, respectively
q	load per unit area
r	radius of a column fan yield-line pattern

CIRCULAR COLUMNS

Column Fan Mechanism

If it is assumed that fans of the type shown in Figure 3(a) form around all of the columns which appear in Figure 1(a), and the centre of the panel undergoes unit deflection, the following expressions for the work done by the load and the work absorbed in the yield-lines are obtained:

$$\text{W.D.} = q(L^2 - \pi r^2) + q\tfrac{1}{2}(\pi c + 2\pi r)(r - \tfrac{c}{2}).\tfrac{1}{3}\left(\frac{c + 4r}{c + 2r}\right) \tag{1}$$

$$\text{W.A.} = ((m + m').2\pi r + 4(M_b + M_b'))/(r - \tfrac{c}{2}) \tag{2}$$

Equating expressions (1) and (2) and introducing the parameter K, = $(M_b + M_b')/(m + m')L$, gives:

$$(m + m') = q\left(\frac{24L^2r - 12L^2c - 8\pi r^3 + \pi c^3}{48\pi r + 96KL}\right) \tag{3}$$

When $(m + m')$ is a maximum, $\partial(m + m')/\partial r = 0$. Thus from equation (3) the following expression for the critical value of r (that which gives the maximum value of $(m + m')$) is obtained:

$$16\pi^2r^3 + 48\pi KLr^2 + \pi^2c^3 - 12\pi L^2c - 48KL^3 = 0 \quad (\text{but } r \leqslant 0.5L) \tag{4}$$

Rewriting expressions (3) and (4) in terms of dimensionless parameters it is possible to obtain:

$$\frac{(m + m')}{qL^2} = \frac{24(r/L) - 12(c/L) - 8\pi(r/L)^3 + \pi(c/L)^3}{48\pi(r/L) + 96K} \tag{5}$$

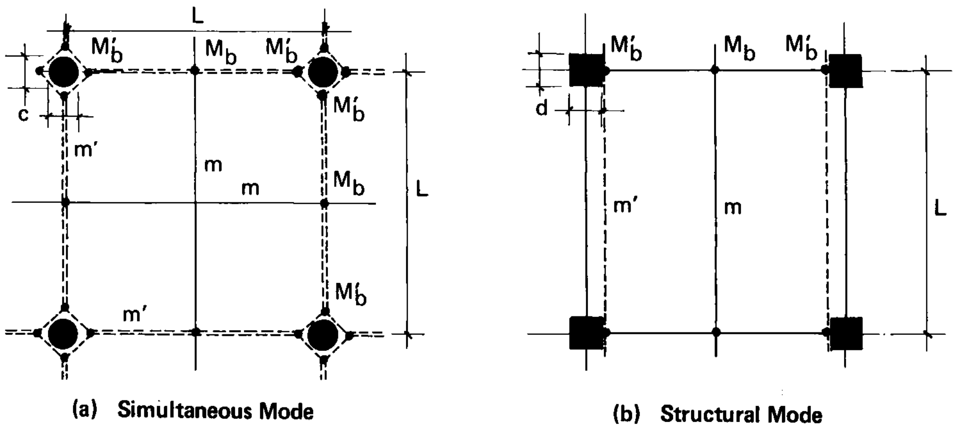


Figure 4 Structural Modes

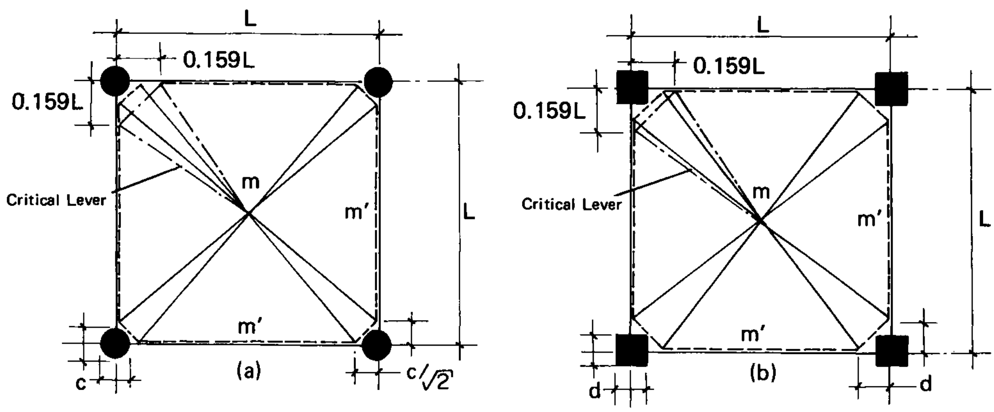


Figure 5 Panel Modes

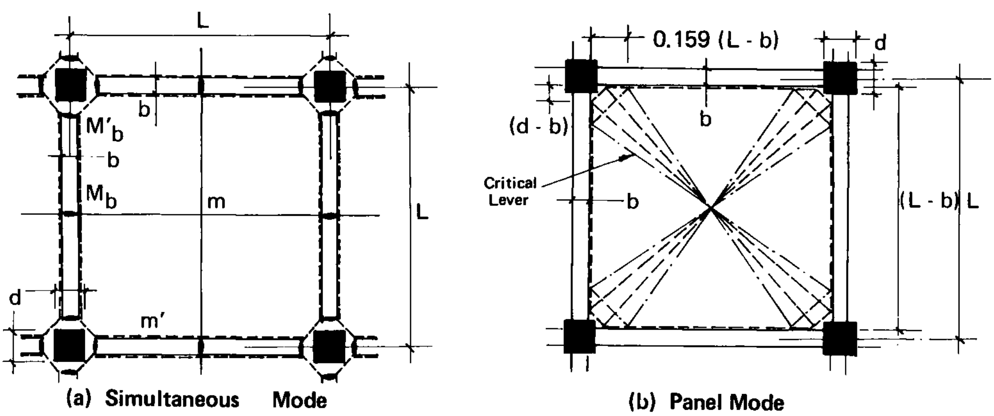


Figure 6 Examples of the Influence of Finite Beam Width

and $$16\pi^2(r/L)^3 + 48\pi K(r/L)^2 + \pi^2(c/L)^3 - 12\pi(c/L) - 48K = 0 \tag{6}$$

(with $(r/L) \leqslant 0.5$).

Equations (5) and (6) can be used, together, to get solutions to the fan problem. Curves given by these two expressions can be seen in Figures 7 to 11. The variation of the critical fan radius with K and c/L, as given by equation (6), is illustrated in Figure 12.

'Simultaneous' Failure Mode

This yield-line pattern, which is shown in Figure 4(a), has been called the 'simultaneous' mode since it is only kinematically possible if failure occurs in both directions at the same time (whereas, in contrast, this is not necessary for the yield-line mechanism shown in Figure 4(b)). The mechanisms shown in Figure 4(a) and (b) have been given the generic name of structural modes since they involve both slab and beams and extend over the full width of the structure.

Assuming unit central deflection of the panel shown in Figure 4(a), the following expressions are obtained:

$$\text{W.D.} = 4q\left(\frac{L^2}{8}.\frac{(2L - 3c/\sqrt{2})}{3(L - c/\sqrt{2})} + \frac{(L + 2c/\sqrt{2})(L - 2c/\sqrt{2})}{24}.\frac{(L + c/\sqrt{2})(L - 2c/\sqrt{2})}{(L - c/\sqrt{2})(L + 2c/\sqrt{2})}\right) \tag{7}$$

$$\text{W.A.} = (m + m')\,\frac{L}{2}.\frac{1}{(L - c/\sqrt{2})}.8 + \frac{(M_b + M_b')}{(L - c/\sqrt{2})}.4 \tag{8}$$

Equating expressions (7) and (8), substituting $K = (M_b + M_b')/(m + m').L$ and writing the result in terms of dimensionless parameters gives:

$$\frac{(m + m')}{qL^2} = \frac{1}{8(1 + K)}\,(1 - \sqrt{2}.(c/L) + (\sqrt{2}/3)(c/L)^3) \tag{9}$$

Plots of the relationship of $(m + m')/qL^2$ with K given by equation (9) are shown in Figures 7 to 11.

'Structural' Mode

By considering the mechanism shown in Figure 4(b), and replacing the square columns with circular ones, it is easy to obtain:

$$(m + m').L + (M_b + M_b') = \frac{qL}{8}.(L - c)^2 \tag{10}$$

and from this $$\frac{(m + m')}{qL^2} = \frac{1}{8(1 + K)}.(1 - c/L)^2 \tag{11}$$

The relationships given by equation (11) are also shown in Figures 7 to 11.

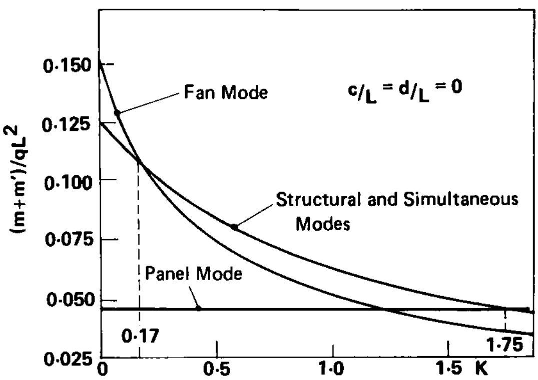


Figure 7

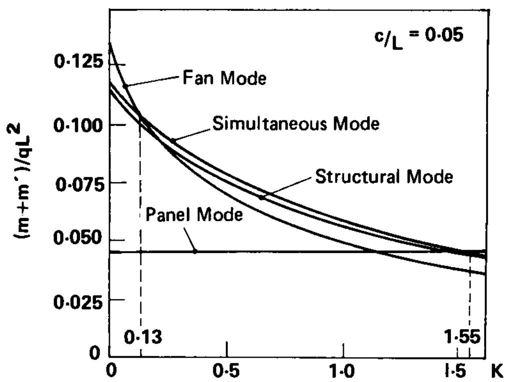


Figure 8

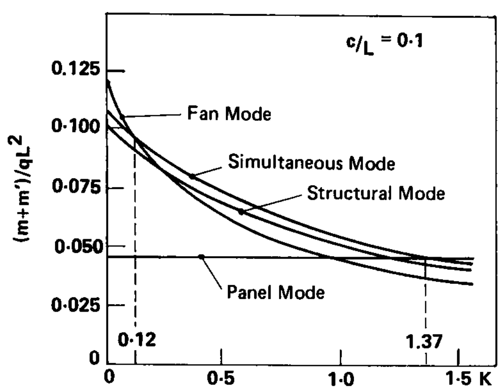


Figure 9

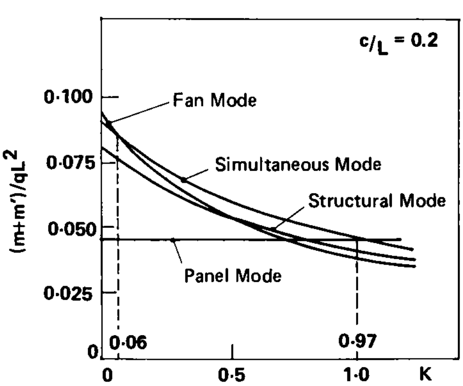


Figure 10

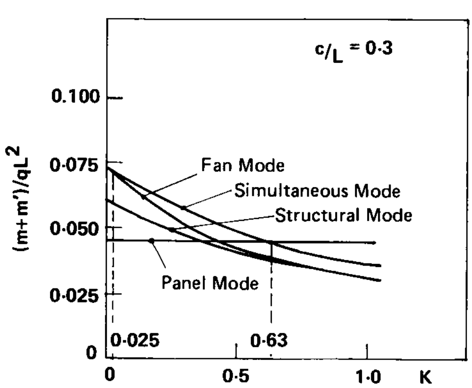


Figure 11

Panel Mode

The yield-line patterns shown in Figure 5 were chosen since, in the first place, the finite size of the columns made the adoption of some form of corner cut-off unavoidable. At the same time, it was felt that the simple corner-lever systems shown would give solutions which were acceptably close to those which would be given by more complicated corner fans (2).

It can be seen from Figure 5(a) that the negative yield-lines which are tangential to the columns intersect the beam lines at a distance $c/\sqrt{2}$ from the column centres. Now it can be shown (1) that this simple corner-lever mechanism gives a maximum value of $(m + m')/qL^2$ (= 0.0455) when the levers are a distance 0.159L from the corners of the panel. Hence it was decided that when $c/\sqrt{2} < 0.159L$, i.e. $c/L < 0.225$, the appropriate solution to use was $(m + m')/qL^2 = 0.0455$, i.e. it was assumed that the columns could force the levers further out than 0.159L but could not bring them closer to the column centres than this.

From Figure 5(a) it is possible to obtain the following full yield-line solution:

$$\text{W.D.} = 4q\left(\frac{(L - 2c/\sqrt{2})}{2} \cdot \frac{L}{2} \cdot \frac{1}{3} + \frac{c}{2}\,\frac{(L - c/\sqrt{2})}{\sqrt{2}} \cdot \frac{1}{3}\right) \tag{12}$$

$$\text{W.A.} = 4(m + m')\left((L - 2c/\sqrt{2}) \cdot \frac{2}{L} + c \cdot \frac{\sqrt{2}}{(L - c/\sqrt{2})}\right) \tag{13}$$

These two expressions give:

$$\frac{m + m'}{qL^2} = \frac{1}{24}\left(\frac{(1 - (c/L)^2)(1 - (1/\sqrt{2})(c/L))}{1 - \sqrt{2}(c/L) + (c/L)^2}\right) \tag{14}$$

Values of $(m + m')/qL^2$ given by equation (14), constrained such that $(m + m')/qL^2 = 0.0455$ when $c/L < 0.225$, can be found in Figures 7 to 11.

SQUARE COLUMNS

Column Fan Mechanism

Combining Figures 1(b) and 3(b) produces:

$$\text{W.D.} = q\left(L^2 - \frac{\pi r^2}{3} + \frac{\pi r d}{3} - \frac{\pi d^2}{12} - 2rd\right) \tag{15}$$

$$\text{W.A.} = (m + m').\left(\frac{8d + 4\pi r - 2\pi d + 8KL}{2r - d}\right) \tag{16}$$

Equating the above two expressions, putting $\partial(m + m')/\partial r = 0$ and rewriting in terms of dimensionless parameters eventually gives:

$$\frac{(m + m')}{qL^2} = [(2 - (\pi/2)(d/L)^2 + 2(d/L)^2)(r/L) + (\pi d/L - 4\,d/L)(r/L)^2 - (2\pi/3)(r/L)^3 - d/L + \pi/12(d/L)^3]/[(8 - 2\pi)(d/L) + 4\pi(r/L) + 8K] \tag{17}$$

and
$$8\pi^2(r/L)^3 - (12\pi^2.d/L - 48\pi.d/L - 24\pi K).(r/L)^2 -$$
$$((48\pi - 6\pi^2 - 96).(d/L)^2 + (24\pi - 96).K.d/L).r/L -$$
$$(24 - 6\pi(d/L)^2 + 24(d/L)^2).K + (12\pi - \pi^2 - 24).(d/L)^3 -$$
$$24d/L = 0 \quad (\text{with } r/L \leqslant 0.5) \tag{18}$$

Curves given by equations (17) and (18) are shown in Figures 14 and 16 to 19.

'Simultaneous' Mode

Replacing the circular columns in Figure 4(a) with square ones again eventually produces:

$$\frac{m + m'}{qL^2} = \frac{1}{8(1 + K)} \cdot (1 - 2(d/L) + \frac{4}{3}(d/L)^3) \tag{19}$$

Solutions to this equation are given in Figures 16 to 19.

'Structural' Mode

Using Figure 4(b) it is possible to obtain:

$$\frac{(m + m')}{qL^2} = \frac{1}{8(1 + K)} \cdot (1 - d/L)^2 \tag{20}$$

Values of $(m + m')/qL^2$ from this expression can also be found in Figures 16 to 19.

Panel Mode

Using Figure 5(b) gives:

$$\frac{(m + m')}{qL^2} = \frac{(1 - 2(d/L)^2)(1 - d/L)}{24(1 - 2d/L + 2(d/L)^2)} \tag{21}$$

however when $d/L \leqslant 0.159$, $(m + m')/qL^2 = 0.0455$.

The above expressions gave the panel solutions shown in Figures 16 to 19.

COMMENTS ON THE LINE-BEAM SOLUTIONS

It can be seen from the Figures that for both column shapes there are three possible critical yield-line mechanisms. For a given column size, the relative strength of the beams, as represented by the parameter K, determines which one of the three is the actual critical mechanism. When beams are absent, or are very weak, the fan mode is critical; with stronger beams the 'simultaneous' mode becomes critical with

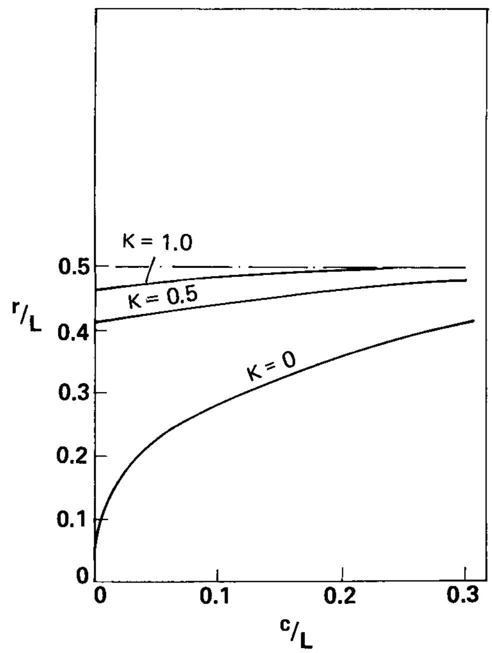


Figure 12 Critical Fan Radii with Circular Columns

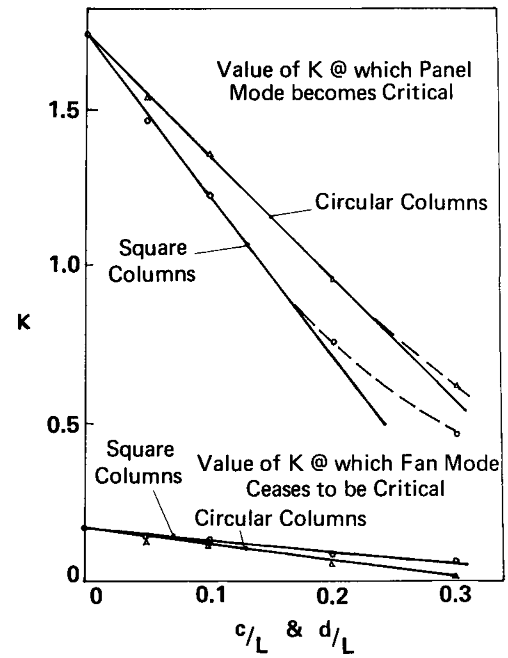


Figure 13 Critical Values of K

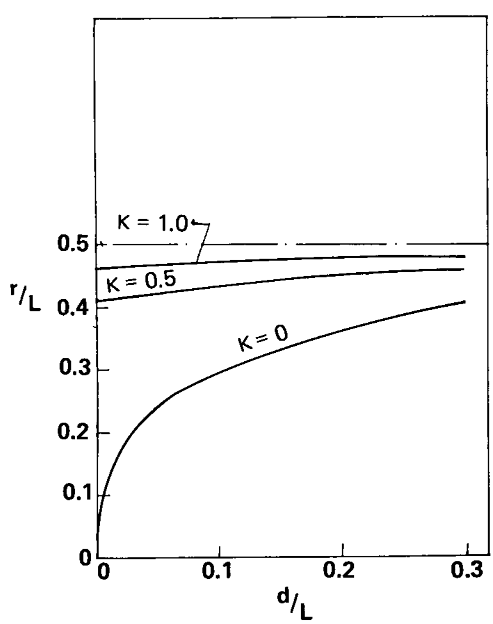


Figure 14 Critical Fan Radii with Square Columns

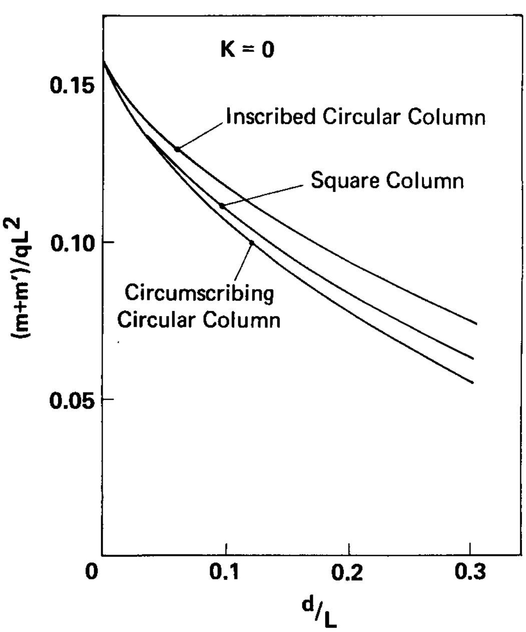


Figure 15 Comparison of Fan Solutions for Square and Circular Columns (no beams)

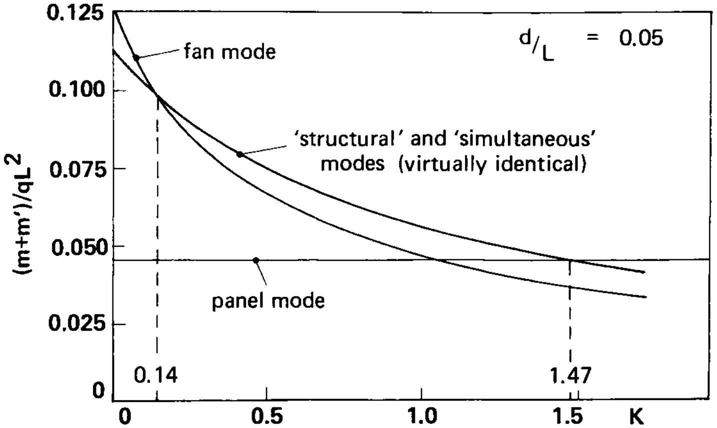


Figure 16

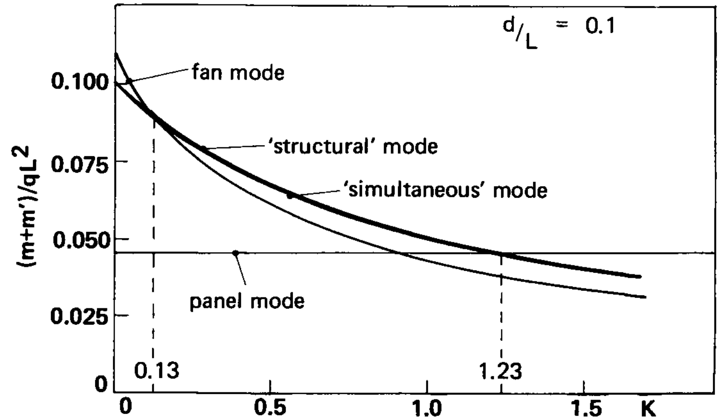


Figure 17

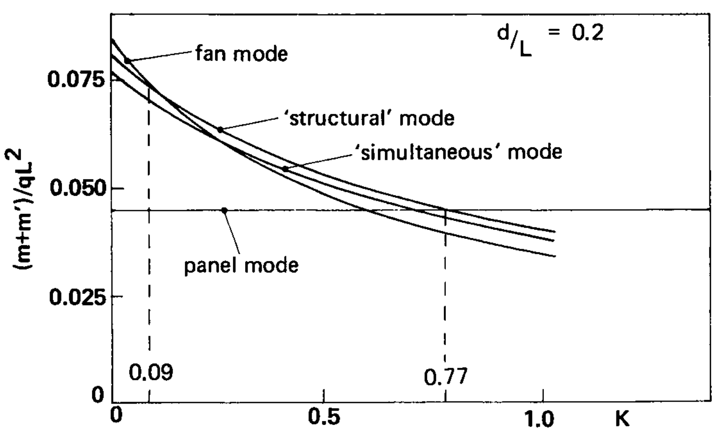


Figure 18

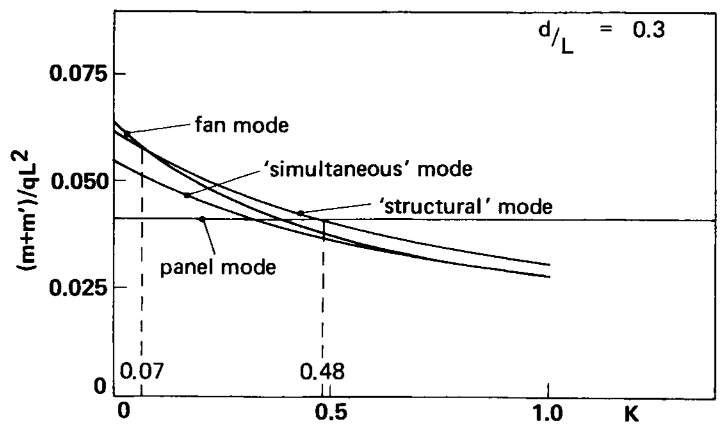


Figure 19

circular columns and the 'structural' mode becomes critical with square, or rectangular, columns. As the strength of the beams increases further the panel mode eventually becomes crucial. The values of K at which the changes in critical mechanism occur are plotted in Figure 13. It can be seen that these values of K decrease almost linearly as the column size increases.

The slab moments of resistance needed to resist all the modes decrease considerably as the strength of the beams increases, as might have been expected. Both slab and beam required moments of resistance decrease, quite significantly, as the size of the columns increases. This is particularly true for the column fan mode, as demonstrated in Figure 15. In this figure the fan solution for a flat plate with square columns is compared with the solutions for the inscribed and circumscribing circular columns. As expected the results for the square columns fall between those for the circular columns.

Figures 12 and 14 show that the radii of the critical fans increase with both column size and beam strength. In particular, in flat plates, the critical radii increase rapidly from zero as the columns increase from points (with zero cross-sectional area).

THE INFLUENCE OF BEAM WIDTH

The effects on the previous solutions, and on the conclusions drawn therefrom, of allowing for the finite widths of the beams have been examined in detail elsewhere (6) and only the principal findings will be reported here.

Perhaps the most important conclusion was that for normal beam and slab construction the line-beam solutions can be safely used for design as long as the values of K used in the calculations are based upon the line-beam definition of $(M_b + M_b')/(m + m').\alpha L$, rather than upon the more strictly correct definition of $K = (M_b + M_b')(m + m').(\alpha L - b)$. This is not true when the 'beams' are wide and shallow, i.e. wide but relatively weak; in this case the line-beam solutions should not be used.

Some of the other points disclosed by the examination carried out in reference (6) were:

i) there is no need to consider the effects of beam width upon the fan mode since this mechanism will only be critical if the beams are extremely small.

ii) the solutions found previously for the 'simultaneous' and 'structural' modes (equations (9), (11), (19) and (20)) can be adapted to allow for the finite width of the beams by multiplying the right-hand side of the equations by $1/(1 - b/L)$; and by adopting the definition $K = (M_b + M_b')/(m + m').(\alpha L - b)$ (the solutions then apply to square panels with beams of width b; and to the rectangular panels shown in Figure 2, with beams of width b and αb in the short and long directions, respectively, assuming $\alpha < 1$); if $b > d$, or $b > c$, then d, or c, should be replaced by b in the equations. In the case of the 'structural' mode the resulting equations are exact solutions. In the case of the 'simultaneous' mode they are acceptably approximate, e.g. if $b/L = 0.2$ and $d/L = 0.25$ (an extreme case) then, for a given value of K, the calculated value of $(m + m')$ will be 95 per cent of the correct solution.

iii) the line-beam panel mode solutions given in equations (14) and (21) can be adapted, approximately, by multiplying the right-hand sides by $(1 - b/L)^2$. More simply, more conservatively, and usually more accurately, the solution for this mode can be taken as:

$$(m + m') = 0.0455(1 - b/L)^2.qL^2 \tag{22}$$

iv) It can be seen from the above that the finite width of the beams, in effect, prolongs the importance of the 'structural' and 'simultaneous' modes, i.e. it has the effect of increasing the value of K at which the panel mode becomes critical, since the slab moments of resistance required to resist the latter are decreased whilst, for a given value of K, those required to resist the 'structural' and 'simultaneous' modes are increased. This is illustrated in Table 1.

Table 1 Values of K at which the Panel Mode becomes Critical (Square Panels)

d/L	b/L	K	
		Allowing for beam width*	Line-beam solution
0.05	0.05	1.89	1.47
	0.1	2.06	
0.1	0.05	1.60	1.23
	0.1	2.06	
0.2	0.05	1.05	0.77
	0.1	1.41	

* $K = (M_b + M_b')/(m + m').(L - b)$

CONCLUSIONS

Three yield-line failure mechanisms are possible in the interior panels of beam and slab structures. With square, or rectangular, columns these are the column fan, the 'structural' and the panel modes. With circular columns and square, or very nearly square, panels (or elliptical columns and rectangular panels) the 'simultaneous' mode replaces the 'structural' mode as a possible failure pattern. Column fans are only likely when beams are absent or are very weak ($K < 0.2$).

Unless the columns are very large (e.g. $d/L \geqslant 0.2$) and the widths of the beams are small ($b/L \leqslant 0.05$) the beams must be quite strong ($K > 1.25$) before panel mechanisms can be critical. In most practical cases, with beams of moderate strength ($K \simeq 1.5$), either the 'structural' or the 'simultaneous' mode will be the critical yield-line mechanism.

The moments of resistance required for the various mechanisms decrease quite considerably with increasing column size (especially relative to the point column datum). This is particularly true of the column fan mode.

The solutions obtained for panels with line-beams (in fact really referring to the solutions for the 'structural' and 'simultaneous' modes) can be safely used to design panels with normal beams, i.e. beams which are not wide and shallow, as long as the values of K used in the calculations are based upon the line-beam definition of $K = (M_b + M_b')/(m + m').\alpha L$.

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DESIGN FOR PUNCHING STRENGTH OF SLABS AT INTERIOR COLUMNS

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ABSTRACT Local or punching failures of flat slabs around columns may occur in one of three possible ways: 1) a localized, primarily flexural failure, 2) a punching shear failure, and 3) a combination of flexure and shear. These three types of local or punching failure are analyzed and compared. Limit design procedures considering them are proposed and integrated with the overall flexural limit design of slabs directly supported on columns. Practical flexural design considerations are explored, design methods are recommended, and reasonably simple flexural design formulas and graphs are presented which are exact for some cases and good approximations for all others. Design to prevent shear and combination failures is discussed and an expression is presented which should enable the engineer to predict the type of failure most likely to occur first. An example is provided to illustrate some of the points.

INTRODUCTION

Limit design of reinforced concrete slabs requires simultaneous consideration of a number of different possible failure modes. These generally include i) overall slab bending failures, ii) overall slab shear failures which are quite rare, iii) local shear punching failures around the columns, and iv) local flexural failures around the columns. Combinations of these failure types can also occur, possibly at loads smaller than those which would otherwise cause one of the individual failures. The primary concern of this paper is with the local flexural or shear failures of slabs around columns, though overall slab bending failures will also be analyzed since they may precede or have an effect on the localized failures.

Local failures at columns can be divided into three types: i) primarily flexural failures, ii) shear failures, actually principal tension stress or diagonal tension failures, and iii) combinations of shear and flexural failures. These classifications are somewhat arbitrary as the following definitions show:

i) A local failure may be regarded as primarily flexural when it occurs at a load greater than or equal to the load required to cause generalized yielding of the slab tension reinforcement in a fan pattern surrounding the column. The load carrying capacity can then be predicted by the yield line theory (12). This type of failure will hereafter be called a flexural punching failure.

ii) A punching shear, or more properly, a principal tension stress(diagonal tension or shear tension)failure can occur at a load less than that causing flexural distress if there is sufficient flexural reinforcement to prevent high flexural stresses at the load at which the principal tension stresses in the slab concrete around a column become larger than the tensile strength.

iii) The combination failure may be regarded as one in which punching takes place after high reinforcement stresses and probably some local yielding have developed in the vicinity of the column, but prior to the formation of a full yield mechanism. It can occur when the effects of local bending moments around the column combine with the local shear stresses to cause failure at a load less than that which would have normally caused failure from either of the two modes acting alone.

The punching strength of a slab will evidently be determined by the smallest of these three strengths and all must therefore be considered. The costs of preventing the three types of failures are not equal, and neither is the suddenness with which each can occur. Principal tension stress failures are the most sudden, most brittle, most difficult to predict and costliest to prevent. In most structures they are also quite unlikely since the bending moments are usually very large around columns, leading to flexural or combination failures. Combination failures tend to have the same drawbacks as principal tension stress failures, except that conspicuous flexural cracks normally form prior to collapse as some of the flexural reinforcement is highly stressed and has probably yielded. The cracking may then provide warning of structural distress if not hidden by floor coverings.

Principal tension stress and combination failures have traditionally been prevented through use of shear head reinforcement, drop panels, and/or capitals. The latter two, in particular, are expensive and awkward. Unfortunately the A.C.I. Code punching analysis method (1, 2) does not permit accurate evaluation of the punching strength. It concentrates on shear and ignores flexure-shear interaction, and also the possible primarily flexural punching problem.

In order to obtain more accurate analyses of punching strength it is necessary to be able to predict which failure mode will be critical and what parameters influence the various strengths. With this information the factor of ignorance is reduced, thereby reducing the need for various expensive devices for enhancing shear strength. It also becomes possible to design required shear head reinforcement more accurately and with, or without, its use, to force punching failure to occur in the more ductile and desirable flexural mode while preserving adequate load carrying capacity. With some ductility assured, the designer can make use of inelastic moment redistribution at ultimate loads to arrange the reinforcement in the simplest and most convenient patterns. Plastic design and analysis of slabs are also basically simpler than elastic methods.

FLEXURAL DESIGN AND ANALYSIS

The most frequently used flexural limit design method for slabs is the yield line method (12). It is based on a rigid-plastic model of slab bending, a requirement for geometric admissibility of possible flexural collapse modes, and the assumption that the slab everywhere possesses sufficient rotation capacity to permit the entire yield mechanism to form before collapse. It is an upper bound theory.

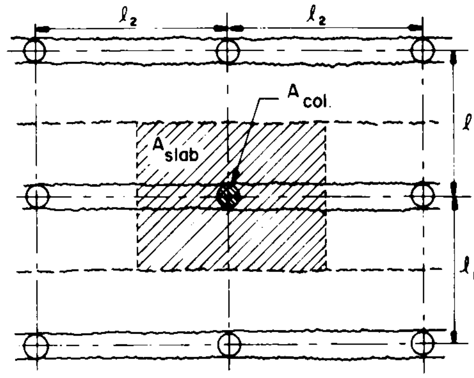

Figure 1 Overall Flexural Failure Mode

Three different possible flexural collapse modes can be identified for any interior panel or group of panels of a braced frame, flat slab structure: i) overall slab flexural failure with formation of a series of yield lines similar to those of Figure 1, ii) local flexural punching failure at one or more columns with formation

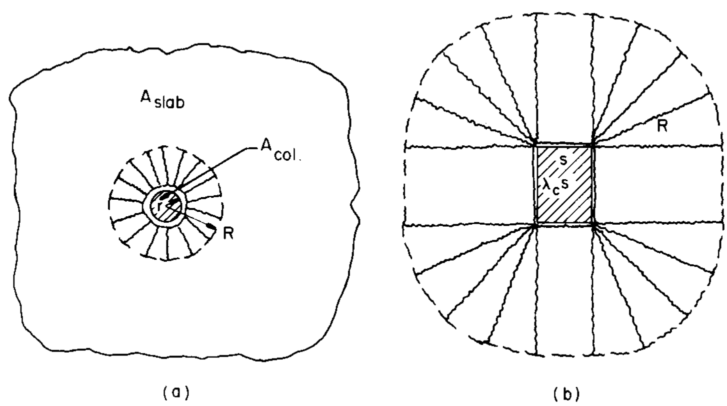

Figure 2 Circular and Rectangular Column Flexural Failure Mode

of the types of yield fan shown in Figure 2, and iii) a possible combination of punching and overall failure. In these Figures positive moment yield lines are shown dashed and negative moment yield lines are continuous.

Overall Flexural Collapse

Looking first, briefly, at the overall slab flexural failures, it is possible to derive the following expression for the load carrying capacity, w, per unit area of the observed (8) and geometrically admissible yield line pattern of Figure 1. For simplicity the columns are assumed to be circular and the reinforcement is assumed to provide isotropic yield moments of magnitude m per unit length for negative moment resistance and $k_m m$ per unit length for positive moment resistance.

$$\frac{wA_{slab}}{m(1 + k_m)} = \frac{8\,\dfrac{A_{slab}}{A_{col.}}}{\lambda\,\dfrac{A_{slab}}{A_{col.}} - \sqrt{\dfrac{16}{\pi}\,\lambda\,\dfrac{A_{slab}}{A_{col.}}} + \dfrac{4}{\pi}} \tag{1}$$

where, $\lambda = \ell_1/\ell_2$ of Figure 1, $A_{col.}$ is the cross sectional area of the column, and A_{slab} is the area of slab contributing load to the column. A_{slab} can be determined by yield line analysis of the overall slab action as recommended by A.C.I. Committee 426 (13). Equation (1) will also serve for rectangular columns of side lengths s and $\lambda_c s$, oriented with s parallel to ℓ_1, if $4\lambda_c$ is substituted for π.

When the columns are relatively small compared to the slab dimensions, only a single negative yield line may appear to form over each row of columns. In that case the second two terms of the denominator of the expression change considerably, but they remain relatively small and the calculated load carrying capability changes very little for the normally large values of $A_{slab}/A_{col.}$ involved. It is particularly interesting to note here, as will also later be observed in the flexural punching analysis, that theoretically the distribution of reinforcement between positive and negative moment has no influence on the load the slab can carry.

If the reinforcement provides isotropic moment resistances, λ must be taken as greater than or equal to 1. When $\lambda = 1$ the yield pattern can form in either direction. It cannot form in both directions simultaneously since that would produce a simplified version of the symmetric flexural punching yield line pattern to be discussed below, with resulting higher predicted load resistance than that obtained by use of the correct flexural punching pattern.

If the reinforcement were arranged so as to produce orthotropic moment resistance, λ could be less than 1 in linear relation to the degree of orthotropy. For optimum design, in fact, the orthotropy should be such that flexural yielding would theoretically occur in both directions at the same load. Variable distributions of the reinforcement, for instance in column and middle strips, can also readily be accommodated in the yield line theory (12, 18).

Flexural Punching

Analyses of parts of the flexural punching problem have been published previously (5, 6). The theoretically exact equation derived there for the flexural punching of an interior circular column of radius r through a symmetrical, uniformly loaded slab with isotropic moment resistances, see Figure 2(a), can be simplified with negligible loss of accuracy for practical ranges of slab and column sizes (18) to:

$$\frac{wA_{slab}}{m(1 + k_m)} \simeq \frac{2\pi\,\sqrt[3]{\dfrac{A_{slab}}{A_{col.}}}}{\sqrt[3]{\dfrac{A_{slab}}{A_{col.}}} - 1.30} \tag{2}$$

This equation is illustrated in Figure 3, as are the load carrying capacities of rectangular columns supporting uniformly loaded isotropic slabs. The latter are

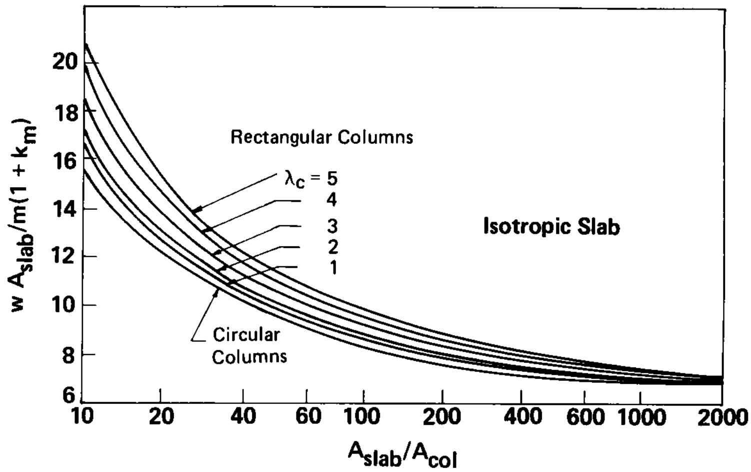

Figure 3 Flexural Punching Load of Interior Columns

based on an analysis (18) of the rectangular column yield line pattern of Figure 2(b) which gives the following expression:

$$\frac{wA_{slab}}{m(1 + k_m)} = \frac{2\,\dfrac{A_{slab}}{A_{col.}}\,(\beta\pi + \lambda_c + 1)}{\beta\left[\dfrac{A_{slab}}{A_{col.}} - \dfrac{\pi}{3\lambda_c}\,\beta^2 - \left(\dfrac{1 + \lambda_c}{\lambda_c}\right)\beta - 1\right]} \tag{3}$$

where $\beta = R/s$ may be obtained by solving:

$$\beta^3 + \frac{3}{\pi}(1 + \lambda_c)\beta^2 + \frac{3}{\pi^2}(1 + \lambda_c)^2\,\beta - \frac{3}{2}\,\frac{\lambda_c}{\pi^2}(1 + \lambda_c)\left(\frac{A_{slab}}{A_{col.}} - 1\right) = 0 \tag{4}$$

Comparing the punching strengths shown in Figure 3 for circular and rectangular columns, it is evident that circular and square columns of equal cross sectional area have almost the same flexural punching load. As the side length ratios of the rectangular columns increase, the flexural punching loads also increase, but for $A_{slab}/A_{col.} = 100$ the difference in punching load between a rectangular column with side length ratio = 5 and a circular column of equal cross sectional area is only 12 per cent. Since the circular shape consistently gives slightly lower flexural punching strength, a designer would be well justified in using only the simpler circular column expression, equation (2), to determine flexural punching strengths of slabs.

It should be noted from equations (2) and (3) that, while a certain total yield moment resistance is required to prevent flexural punching, its distribution between the positive and negative moment regions does not affect the strength. This is similar to the finding in overall flexural action and, in fact, equations (1), (2) and (3) contain many of the same parameters. Therefore, as far as flexural strength is concerned, the designer should be free to distribute the total reinforcement between the top and the bottom of the slab as may be convenient or necessary from other considerations such as serviceability or the exigencies of concrete placement.

Size of Yield Fan

An important factor to consider in the design of a slab against flexural punching is the radius of the yield fan. This will very much influence the distribution and detailing of the reinforcement and, if drop panels are to be used, their horizontal dimensions, in order to ensure that the required yield moments are actually available at the locations of the various yield lines. At the same time it is necessary to ensure that the slab is not weakened so much between planned yield lines that unforeseen ones develop, producing a pattern less strong than that envisioned in the design. Thus, for instance, the bottom, i.e. positive moment, reinforcement should be carried on into and around all columns to avoid arbitrarily producing the positive moment circular yield line at a location with much reduced $k_m m$. (The bottom reinforcement should also be continued into the columns to increase the shear strength as well as post-punching strength. This will be discussed later. See also reference 13).

For interior circular columns supporting an isotropic slab, the ratio, ρ_r, of the fan radius, R, to the column radius, r, can be written to a close approximation (5) as:

$$\rho_r \approx \sqrt[3]{\frac{3}{2}\frac{A_{slab}}{A_{col.}}} \tag{5}$$

The fan size for rectangular columns can also be found approximately by using the R of the equivalent circular column as the corner fan radius of the rectangular column.

Flexural Punching of Orthotropic Slabs

In many slab structures elementary statics indicates that the reinforcement in two mutually perpendicular directions should not be equal. This leads to an orthotropic distribution of yield moment strengths in the slab. The theoretical flexural punching behaviour of such orthotropic slabs has been studied in some detail by Nabar (18), and the following tentative (until confirmed by experimental evidence) conclusions can be drawn from his work:

i) The most important parameter in determining the flexural punching strength of slabs is the total yield moment available. The distribution of the yield moments between the two orthogonal directions has relatively little influence, even for yield moment ratios in the two directions as high as 5 to 1.

ii) As a consequence, when dealing with flexural punching of an orthotropic slab, it is possible to define an equivalent isotropic slab with m equal to the average of the two negative yield moments per unit length in the two orthogonal directions, and $k_m m$ equal to the average of the two orthogonal positive yield moments per unit length.

iii) Use of the equivalent isotropic slab for interior circular or square columns will lead to flexural punching loads no more than 5 per cent greater than the correct ones.

iv) Interior circular columns give slightly lower (less than 5 per cent for practical values of A_{slab}/A_{col}.) flexural punching strengths than interior square columns of equal cross sectional area. Therefore, the flexural punching load calculated for an equivalent isotropic slab supported on a circular column of equal area will be very close to the correct flexural punching load for an orthotropic slab supported on a square column.

v) In general, interior rectangular columns supporting orthotropic slabs give somewhat higher (up to 15 per cent) flexural punching loads than do square columns of equal cross sectional area. In the few cases where the rectangular column gave lower punching loads, the difference was of the order of one or two per cent.

vi) This means that the approximate expression, equation (2), applied to interior circular columns of equal area, supporting equivalent isotropic slabs, can be used to safely and reasonably accurately predict flexural punching strengths for interior rectangular columns supporting orthotropic slabs. Columns with side length ratios up to 5 to 1 were studied.

Since orthotropy, and also the distribution between positive and negative yield moment resistance, apparently do not materially affect the flexural punching strength of slabs, the designer should have great freedom to arrange the reinforcement in such a way as to maximise the overall flexural strength and serviceability of the slab.

SHEAR ANALYSES

Shear Punching

As was indicated above, it is unlikely that a true shear or diagonal tension failure will occur in a slab with normal building construction proportions. There are always large bending moments in the slab near a column which means that the relationship between applied slab moment and slab yield moment capacity will have an influence on the shear strengths. Hognestad (9) was apparently the first to recognize this and to include a term, V_{flex}, which supposedly represented the flexural punching strength of the slab, in a recommended punching shear expression. Unfortunately, he, and several others (4, 14, 16, 17) who followed him, used over-simplified yield line patterns (6) to calculate V_{flex}, thus appreciably over-estimating the flexural strengths of their specimens and making their empirically obtained expressions at least suspect.

In writing the Code (1, 2) A.C.I. Committee 318 did not include the influence of the bending moment on the punching shear strength, requiring the designer to only compare a nominal shear stress with a measure of the tensile strength of the concrete. The joint A.S.C.E.-A.C.I. Task Committee on Shear and Diagonal Tension (A.C.I. 426) has, however, made a number of recommendations and comments on the problem of shear punching in slabs (13).

Shear - Flexure Interaction in Punching Failures

As mentioned above, it has long been recognized that the shear and the bending moment interact, and Committee 426 (13) cited a large number of studies in which the interaction was considered in arriving at punching shear strength expressions. More recently, Long (15) has provided some additional interaction formulas. None of these expressions have, so far, found sufficient favour with either Committee 426 or Committee 318 to be recommended for design. It is possible, however, to examine various effects of such interaction qualitatively and to draw some conclusions regarding good design practice.

Shear-flexure interaction theoretically cannot influence the predicted flexural punching failure strength of the slab, which is based on yield line theory and the knowledge that slabs are almost invariably considerably under reinforced. Therefore, punching of the column through the slab at a load less than the theoretical flexural punching load will here be blamed on shear, or on shear-flexure interaction. The interaction can result in either a shear-tension or a shear-compression type of failure, or in a combination of the two. In the shear-tension type, the initial flexural cracking progresses to form the inclined principal tensile stress cracks characteristic of such a failure, causing the familiar frustrum of a cone or pyramid to be pushed out of the slab. In a shear-compression failure, extension of the flexural cracks reduces the depth of the compression zone of the slab around the column to such an extent that the principal compressive stresses exceed the concrete compressive strength. This leads to local crushing which reduces the shear strength of the region, again producing the typical shear punching failure.

Since slabs are usually relatively lightly reinforced, shear-compression at first would appear to be an unlikely failure mode. However, the development of a full yield fan, which is necessary for complete moment redistribution, requires considerable rotation of the yield lines around a column, which leads to wide cracking and consequently large reductions in the depth of the compressive stress block. There is evidence (13, 15, 20) that many shear punching failures of realistic specimens are preceded by yielding of the top reinforcement around the column periphery, and Zaghlool and de Paiva (20) have stated that the punching of the column through the slab at ultimate can be considered a secondary phenomemon following the destruction of the compression zone, due to the combined action of flexure and shear.

Evidently, it is advantageous to improve the ductility of the region of the slab surrounding the column as well as to strengthen it against shear failure. It has been recognized for some time (3, 5, 6, 13) that extension of the bottom reinforcement through the negative moment region of the slab above and around a column has a beneficial effect on the punching shear strength. Two mechanisms seem to be involved. One is the dowel action of the reinforcement passing through the column concrete, and the other is the confining action of the bar grid on the concrete in compression around the column. Both may act simultaneously, with the dowel action being maximized by use of fewer but larger bars, while the confinement effect would be improved by use of many, closely spaced, small bars. This reinforcement will also enhance the membrane action of the slab to prevent total collapse after a punching failure.

A question which is still unresolved is how much ductility, i.e. rotation capacity, is required in the yield lines around the column periphery. Long (15) cites references which indicate that in many specimen failures the full yield fan did not form prior to punching. It is, however, quite possible that even partial redistribution of moments will result in a flexural punching resistance almost equal to that obtained by formation of the full yield fan. If then partial formation of the fan, with considerable rotations around the column, triggered a shear failure, it would indicate that the flexural punching analysis can sometimes give a reasonable prediction of the punching strength even though the final failure is apparently of the shear type.

Determination of the Critical Failure Mode

It is important that the designer be able to choose the failure mode of his structure under gross overloading. Yield line analysis of both the overall and flexural punching failure provides a means for making this choice between the two flexural failure modes, since the flexural strength of slabs can be determined fairly accurately. As pointed out above, the shear punching strength is not so

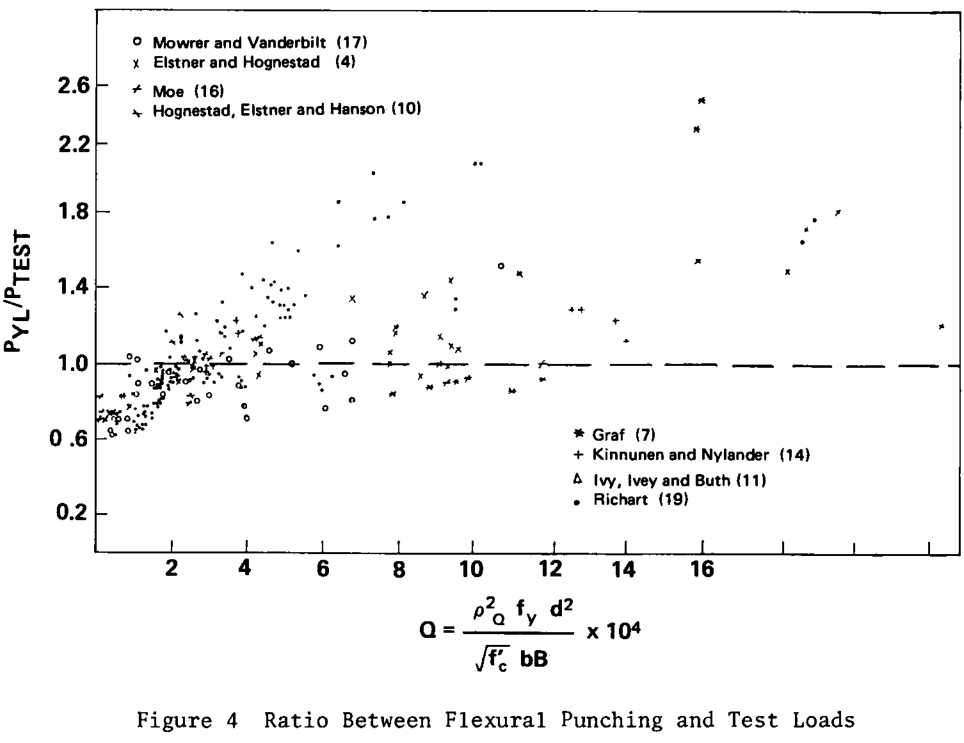

Figure 4 Ratio Between Flexural Punching and Test Loads

easily determined. However, a large number of punching tests have been conducted over the years (4, 7, 10, 11, 14, 16, 17, 19 and many others). The interior column type specimens of the cited references were analysed by yield line theory (6) and the ratio $P_{Y.L.}/P_{test}$ is plotted in Figure 4 against an empirical parameter

$$Q = \frac{\rho_Q^2 \ f_y d^2}{\sqrt{f_c'} \ b \ B}(10^4) \tag{6}$$

whose origins are described in reference 6. In this expression B is the perimeter of A_{slab}, b is the column perimeter, d is the effective depth of the slab, f_c' is the concrete compressive strength and f_y is the yield strength of the reinforcement (both expressed in pounds per square inch, requiring $\sqrt{f_c'}$ to be multiplied by .08 when stress is in N/mm^2). The term ρ_Q is the negative moment reinforcement ratio in the slab around a column when the positive moment (bottom) reinforcement of the slab is extended through the column and the slab region surrounding it. If the positive moment reinforcement is not so extended, ρ_Q must be taken as the sum of the positive and the negative moment reinforcement ratios in the column strips. This rule is based on the beneficial effect of bottom reinforcement on the shear punching strength and on the ductility of the slab-column joint. It biases expression (6) to predict a greater likelihood of shear rather than flexural punching failure of joints in which the bottom slab reinforcement is omitted in the column region and gives good agreement with experimental results (5, 6).

Examining the cloud of data points in Figure 4, it is apparent that for values of Q less than 2, $P_{Y.L.}/P_{test}$ is always less than or equal to 1. That means the theoretical yield line flexural punching load is smaller than the load at which the specimen collapsed, indicating that the failure was of the flexural punching type. For values of Q between 2 and 4 the failures seem to be divided rather evenly between $P_{Y.L.}/P_{test}$ less than or greater than 1, indicating that about half the specimens failed in the flexural punching mode and half in the shear punching mode. For Q greater than 4 the failures were predominantly in shear. Since the value of Q varies as the square of ρ_Q, this emphasizes the importance of extending the bottom reinforcement to prevent premature punching shear failures.

EXAMPLE

Interior slab panels of the type shown in Figure 1 are to be designed for a service live load of 400 Kg/m^2. The column spacings are to be 6 m centre to centre in the North-South direction and 8 m centre to centre in the East-West direction. The columns will be circular, 0.5 m in diameter. To control deflections, a slab thickness of 0.2 m will be used, leading to an effective depth, $d \simeq 160$ mm. The ultimate load will then be 1350 Kg/m^2, using A.C.I. Code (1) load factors.

Using equation (1), the total moments required to prevent overall flexural collapse will be $m(1 + k_m) = 50$ kNm/m in the North-South direction and 93 kNm/m in the East-West direction. To resist flexural punching, use of equation (2) leads to an isotropic moment requirement of $m(1 + k_m) = 80$ kNm/m. Since Nabar (18) has shown that this can be satisfied by making the average of the two orthotropic moment capacities equal to the isotropic $m(1 + k_m)$, the flexural strength requirements can be met by providing sufficient total positive plus negative moment reinforcement to furnish 60 kNm/m North-South and 100 kNm/m East-West. Using a concrete strength $f_c' = 30$ N/mm^2, standard U.S. grade 60 reinforcement, and setting $k_m = 0.5$, the required reinforcement ratios will be: $\rho_{NS}^+ \simeq 0.0024$, $\rho_{NS}^- \simeq 0.0048$, $\rho_{EW}^+ \simeq 0.0040$, and $\rho_{EW}^- \simeq 0.0080$. In accordance with the previously cited findings (3, 5, 6, 13), the bottom reinforcement should be extended into the columns and will therefore most conveniently be made the same throughout the slab, while use of equation (5) indicates that the top reinforcement should extend 1.8 m plus development length in each direction from the centres of the columns.

Checking punching shear strength with equations 11-37 of reference 1, V_c = 600 kN and the applied force due to the ultimate load is 635 kN. Checking further with equation (6), using ρ_Q = 0.008 since the bottom reinforcement will be detailed to run through the columns, Q = 0.35 indicating that the punching failure will occur in the flexural mode. Therefore the apparent understrength in punching shear can be disregarded.

SUMMARY AND CONCLUSIONS

The problem of the punching of columns through flat slabs has been discussed. Flexural and shear punching failures have been identified, classified and analysed. Graphs and equations have been presented to enable the designer to quickly check slab designs for flexural punching problems at interior columns and design guidance has been given for increasing shear and flexural punching strength when necessary. The flexural punching problem has also been related to the overall flexural limit design of slabs.

The most important conclusions that can be drawn from this study are:

Punching of columns through the slabs they support may occur in one of three modes: flexural punching, shear punching and combined flexural and shear punching.

Reasonably simple design equations and graphs, based on yield line theory, are available for the analysis or design of flexural punching, see equations (2) and (3), and Figure 3.

The flexural punching equation, equation (2), for circular columns supporting isotropic slabs is also safely and quite accurately applicable to rectangular columns and orthotropic slabs.

The overall flexural strength of the slab can be related to the same parameters and is influenced by the same design variables as the flexural punching strength, as can be seen by comparing, for example, equations (1) and (2).

An expression is given which appears to successfully differentiate between slabs likely to fail first in flexural punching and those likely to fail first in shear punching.

Theoretical and experimental findings indicate that one of the most effective means for increasing flexural ductility and shear punching strength, without resorting to separate shear reinforcement, is extension of the bottom reinforcement of the slab through the column and the region surrounding it. Further tests should, however, be conducted to confirm and quantify these findings.

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BEHAVIOUR OF FLAT SLAB/EDGE COLUMN JOINTS

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ABSTRACT Scaled models have been used in an experimental investigation of the punching capacity and moment transfer characteristics of typical flat slab edge column connections. One of the main aims was to examine the influence of, and attempt to relate, the location of the slab free edge to the column punching capacity. The punching loads of the various joints were found to be in excess of those predicted by both C.P. 110 and A.C.I. 318-77. In addition, the tests showed that the moments transferred from the slab to the columns could be closely predicted by the A.C.I. method of analysis whilst C.P. 110 overestimated these moments by between 25 and 50 per cent.

INTRODUCTION

One of the major problems associated with flat slab construction is the susceptibility of the structure to punching shear failure. For edge columns in particular the presence of moment transfer adds further to the concentrated stress condition at the junction.

A brief review of the differences in the design provisions for shear in the major Codes of Practice reflects the considerable uncertainty which exists. A comparison of the differences in design capacity for interior columns has been presented by Long (1) where the permissible stresses in C.P. 110 (2) were found to be very conservative relative to its predecessor C.P. 114 (3) or A.C.I. 318-77 (4). Some of the difference can be accounted for by the location of the critical perimeter from the column face, being at 3/2 x slab depth in C.P. 110, while for C.P. 114 and A.C.I. the perimeter is at a distance equal to 1/2 x effective depth.

For edge columns however, the A.C.I. code provides the more conservative estimates of the punching capacity. For edge columns, the influence of moment transfer which did not appear in C.P. 114 at all, is now included in C.P. 110 as a constant magnification factor of 1.25 on the design shear force. In A.C.I. 318-77, however, the moment transferred to the column is obtained from an equivalent frame analysis of the loaded structure and this is used to compute additional shear stresses at the critical section.

The series of tests reported in this paper was drawn up in order to provide an accurate assessment of the accuracy of the two codes regarding ultimate shear capacity and the influence of moment transfer.

TEST PROGRAMME

Model Types

Most of the research carried out to date into the strength of the slab edge column junction has employed models of the type shown in Figure 1(a) where the line of contraflexure forms the boundary of the slab. On the basis of the modelling work carried out by Neth (5), a two column model, shown in Figure 1(b), was used as the main model type of the test series. Such a model more accurately reproduces the boundary conditions which exist in an actual structure. Some one column models were also used; these models, when loaded around their boundary, tend to exhibit excessive deflections and they do not always produce a mode of failure similar to that which develops in practice. The main reason for their use is their simplicity and ease of testing relative to the much more complex and expensive two column models.

Boundary Conditions

Figure 2(a) shows a typical floor plan of a flat slab structure. The basis of the two-column model is the combination of the two shaded areas each representing an edge column with its corresponding half panel section, see Figure 2(b).

Under the system of gravity loading, points C_1 and C_2 on the column centre lines will be characterized by zero, or near zero, slopes of their respective centre line deflection profiles. Figure 2(c) shows how the model will, when loaded, automatically satisfy the zero slope condition at position C_1 on account of the model symmetry. However, position C_2 at the model edge will, when loaded, have the maximum slope along that edge unless some form of external restraint is provided. The function of the edge restraint system is shown in Figure 2(d) where an anticlockwise moment is introduced at C_2 to bring about the required null slope condition at this edge position. It consists basically of a pair of vertical members clamped to the slab at their top ends and braced apart at their bottom ends by a strut.

Details of the Models

Neth has shown that for models of a scale smaller than 1/4 a correction factor to take account of scale effects is required; for larger models this factor has negligible influence. In order to comply with Neth's findings, and also to suit available supplies of aggregate and small diameter reinforcement of the required yield and bond characteristics, 1/3 scale was adopted for the two column models of the main test programme. Table 1 gives an outline of the variables which were investigated with the models.

Details of the loading and support systems are presented in Figure 3. Great care was taken to ensure accurate monitoring of the horizontal column reactions and column and slab rotations. In addition, column and slab deflections and the strain in reinforcing bars in critical areas were also measured.

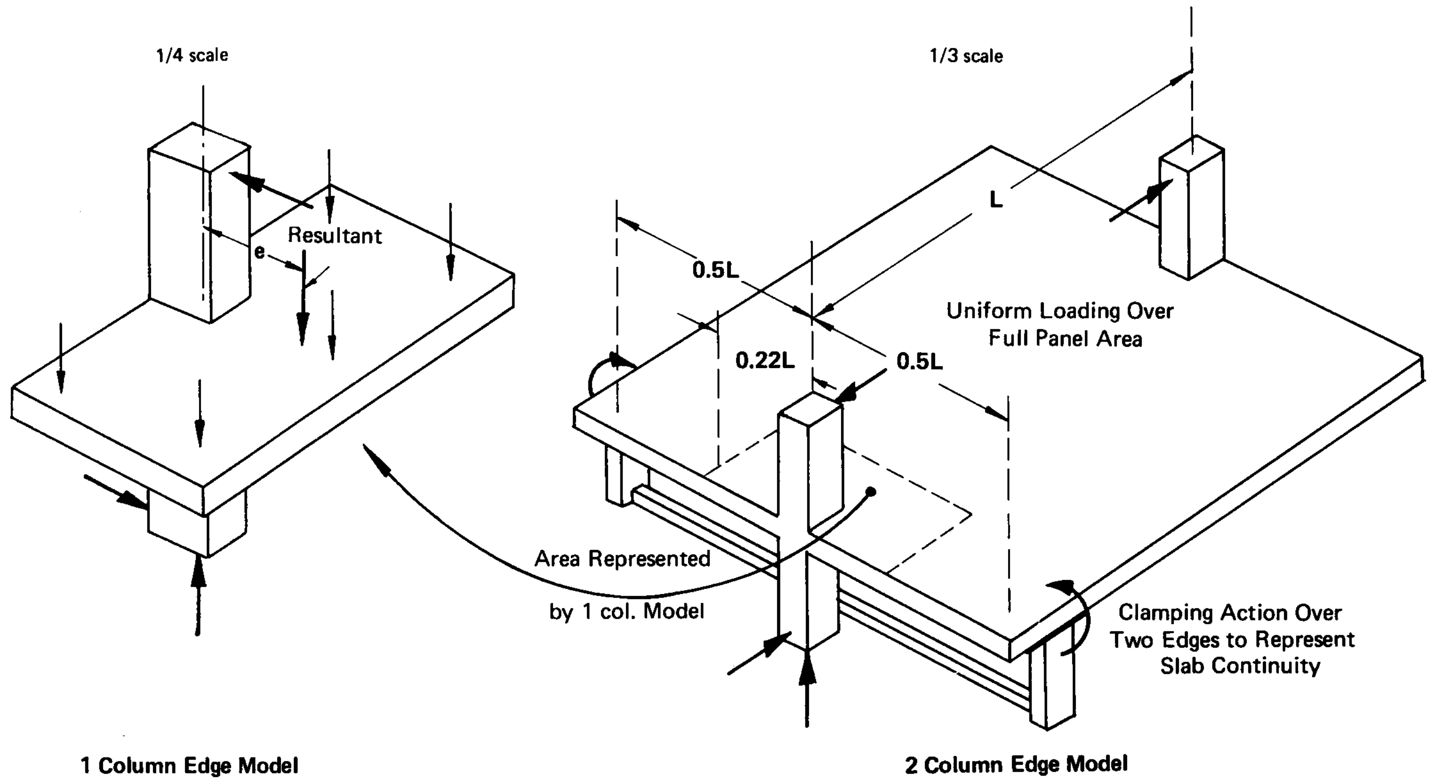


Figure 1 Edge Column Models

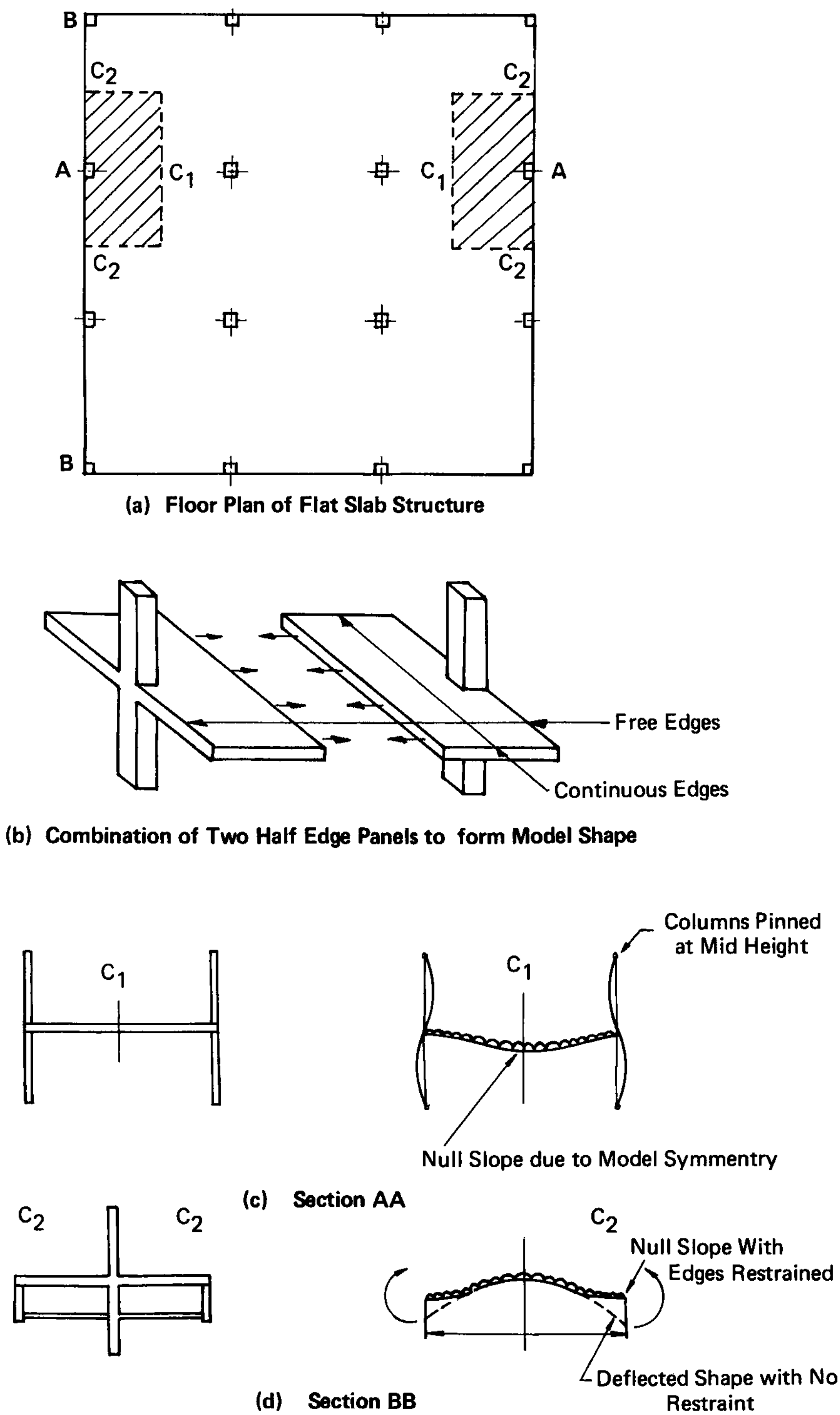


Figure 2 Model Shape and Boundary Conditions

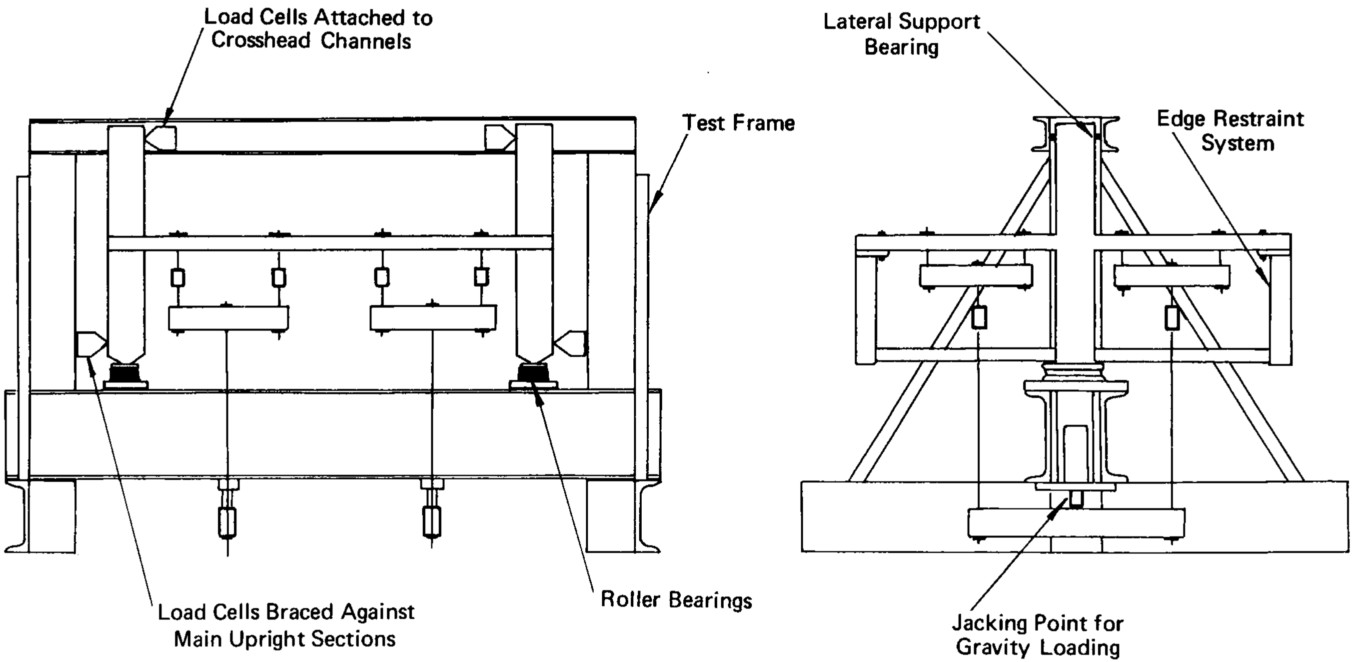


Figure 3 Loading and Support Systems

Table 1 Model Details

MODEL NUMBER	FREE EDGE POSITION	COLUMN SIZE mm x mm	AVERAGE STEEL PERCENTAGE Negative	Positive
1		100 x 100	1.0	0.5
2		100 x 100	1.0	0.63
3		100 x 100	0.74	0.37
4		150 x 100	0.69	0.38
5		150 x 100	0.69	0.38
6		150 x 150	0.69	0.38

f_y = 515 N/mm^2

Single Column Models

As previously stated these much simpler models have been extensively used in previous testing programmes by Stamenkovic (6), Regan (7), Hawkins (8), Beukel (9) and others. The main reasons for which they are favoured are their simplicity in manufacture and both test set-up and instrumentation. They also allow larger scale models to be tested than would be possible with the full panel models. However, they are not capable of permitting the redistribution of moments which can occur in a real structure as this results in a migration of the line of contraflexure. The location of the line of contraflexure or the ratio of moment to shear can vary in the two column models but for the simple model it will be a constant value throughout the test.

Neth and Taylor (10) have suggested a way of resolving the moment-shear (M/V) ratio dilemma by testing similar specimens loaded at different radii from the column. In order to establish whether it would be possible to obtain more satisfactory results from the one column model it was decided to conduct a series of tests in which the essential variable was the eccentricity (i.e. M/V) of the applied loading.

TEST RESULTS

Crack Pattern

The development of the crack patterns can be divided into four main areas.

Flexural cracking around the columns. Circumferential cracks were always the first to appear around the column junction and occurred at approximately the design load. As the loading was increased radial cracking extended outwards from the inner column face, usually initiating at the column corners.

Torsion cracking. Apart from models 1 and 5 where this type of cracking was either prevented or eliminated, torsion cracking could be easily identified by a crack formation extending backwards and outwards from the inner column corners towards the slab edge and then progressing down the free edge at an angle of approximately 45° to the horizontal.

Midspan cracking. Apart from the actual failure surface, cracks in this region were the most pronounced during the test. Occurring soon after the torsional release these flexural cracks developed on the undersurface of the slab at midspan between the columns and at the face of the two lines of tee pieces, extending across the full slab width in these three areas.

Punching shear failure. The type of cracking associated with this type of failure is well known and characterized by the slab separating from the column around an inclined failure surface. The inclined failure surface could be clearly observed on the slab edge starting at the column/slab soffit junction, crossing the torsion cracks and emerging at the surface, often splitting off a considerable amount of the concrete cover to the top reinforcement.

Ultimate Capacity

The test failure loads are presented in Table 2 and compared with predictions based on C.P. 110 (2) and A.C.I. 318-77 (4).

It must be pointed out that the punching failure of model number 3 was strongly influenced by excessive midspan deflections which occurred as a result of the low percentages of both column positive and midspan negative steel. The actual failure surface was more indicative of a torsion-shear failure in an overall yield mechanism and this accounts for the low capacity recorded. Indeed, using yield line analysis, the test load of model 3 can be predicted to within a few per cent.

Table 2 Ultimate Capacity

MODEL NUMBER	PUNCHING LOAD (P_T), kN	PREDICTED CAPACITY (P_P), kN		P_T/P_P	
		C.P. 110	A.C.I. 1977	C.P. 110	A.C.I. 1977
1	66.21	37.8	43.8	1.75	1.51
2	50.08	26.4	22.9	1.89	2.18
3	32.66**	22.8	17.7	1.43	1.84
4	37.12	22.7	15.5	1.64	2.40
5	31.81	13.0	13.6	2.45	2.34
6	37.63	22.9	16.8	1.64	2.24

It is interesting to note that the repositioning of the slab free edge from the outer column face (model number 4) to the inner column face (model number 5) caused only a 14 per cent reduction in the punching capacity, thus showing the small contribution of the 'edge beams' to the ultimate joint capacity. The C.P. 110 prediction in this case was extremely conservative. The median value of the ratio of test capacity to ultimate capacity for C.P. 110 was 1.8 with predictions from the A.C.I. 318-77 code approximately 17 per cent lower.

The presence of the slab overhang in model number 1 resulted in a failure surface which was very similar to that obtained from interior column tests when a column is subject to both shear and moment transfer. When the joint is treated as such, according to C.P. 110 the ratio P_T/P_P is reduced from 1.75 to 1.54.

Moments Absorbed by the Columns

A typical moment transfer versus live load curve is presented in Figure 4. For loads up to approximately 0.24 P_T, A.C.I. 318-77 gave excellent agreement with the total moment line. Thereafter the effect of the flexural and torsional cracking caused significant changes in the behaviour of the moment transfer curve. Also at this stage the moments absorbed by the upper and lower columns were radically changed as the slab started to undergo considerable horizontal expansion due to flexural cracking. It can be seen from Figure 4 how the upper column moment had

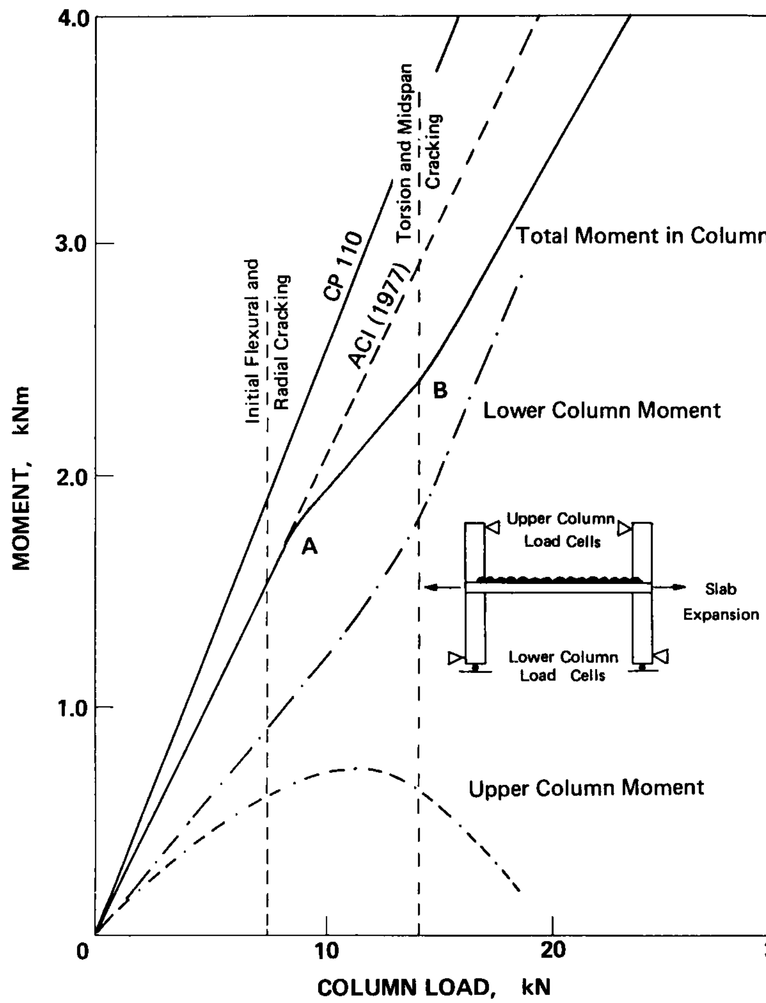


Figure 4 Moment Transfer versus Live Load Curve for Model Number 4

been reduced almost to zero at only a little over midway through the test. Due to the nature of the model the effect of this problematic expansion was eliminated when the upper and lower column moments were combined to give the total moment curve as shown. Serious misunderstanding would arise if the model was constructed with lower column stubs only unless allowance was made for the horizontal expansion, either in the analysis or else by adjustment of the column load cells during the test corresponding to the measured slab expansion.

Apart from the irregularities arising close to failure there are two main changes on the total moment curve. The reduction in rate of moment increase occurring at stage A of the curve can be identified with flexural and radial cracking around the columns. Stage B, at which the moment transfer increased to a value close to the initial rate prior to cracking, corresponded to the occurrence of the torsional and, more especially, the midspan cracking which caused a redistribution of moments back to the column.

As a means of comparing measured moments with predicted values, Table 3 presents values of the various eccentricities obtained from the model tests (the eccentricity is taken as the slope of the M/V line). As well as giving the values obtained from the C.P. 110 and A.C.I. 318-77 methods, results from a plate bending finite element program are also provided.

Table 3 Moment Transfer-Eccentricity (all values in mm)

MODEL NUMBER	EXPERIMENTAL M/V FINAL SLOPE*	C.P. 110	A.C.I.318-77	FINITE ELEMENT
1	100	118	91	110
2	145	163	126	142
3	135	163	126	142
4	176	256	211	236
5	161	256	205	176
6	222	268	217	248

* Slope of line after stage B.

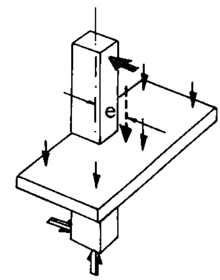
In all cases, and more especially for the models with the larger column sections, C.P. 110 overestimated the moment transfer. The A.C.I. approach, in which the structure is analysed as an equivalent frame, gave much better estimates, although for models 1 and 2 the method produced underestimates of the moment transfer. These models on the other hand, which were for cases of relatively low column stiffness, showed a higher safety margin for shear than the other models tested. The finite element analysis gave results lying within the range bounded by C.P. 110 and A.C.I. values. A slightly more amenable design office tool in the form of a beam grillage method of analysis has been investigated by Rankin (11). Results have been found to be similar to the finite element values but one of the main interests in this method has been that it is possible to accurately predict the changes of slope on the moment transfer curve associated with the various stages of cracking.

Single Column Model Test Results

The single column models were representative of the appropriate portion of model number 4 of the main test programme and the single variable used was the eccentricity of the applied loading. The effect of reducing this eccentricity was to produce a much more characteristic punching failure surface along with deflections which were more compatible with the results of the two column model test. Of more importance, however, the punching loads themselves were much closer to the ultimate strength of model number 4 as Table 4 shows.

Table 4 Comparison of One and Two Column Model Results

MODEL NUMBER	ECCENTRICITY (e), mm		PUNCHING CAPACITY, kN	
	1/4 scale	1/3 scale*	1/4 scale	1/3 scale*
1'	158	210	13.2	23.5
2'	121	161	16.5	29.3
3'	99	132	22.8	40.5
4		135 at ultimate		36.7



* converted from 1/4 scale of single column models to 1/3 scale of two column models.

DESIGN PROPOSALS

It would not be possible to put forward a highly sophisticated design method on account of the rather limited extent of present test results, so pursuit of a more straightforward approach was considered appropriate. The following points were used in a tentative method for the prediction of the ultimate capacities of the present model tests. Further details of the calculations for ultimate capacity, involving the various factors mentioned below, are given in Appendix A.

Eccentricity factor. At present C.P. 110 has no regard for the variation in the amount of moment transfer from the slab to the edge column. The shear force magnification factor quoted in clause 3.6.2 of C.P. 110 has the form [1 + (12.5 M/VL)] which can be expressed as [1 + (12.5 e/L)]. A similar expression is proposed for use with edge columns of the form [1 + (3.0 e/L)] where the e/L coefficient has been selected on an empirical basis.

Reduction of the critical area. To take account of the torsion cracking and consequent redistribution of shear stress which occurs, the critical area has been reduced by 75 per cent of the side length of the critical perimeter. The location of critical perimeter itself has been shifted from a distance of 3/2 x slab depth from the column face to 1/2 x effective depth. Following Long's recommendation for the use of higher ultimate shear stresses with regard to his work on interior columns, the values given in C.P. 110 have been increased for application to the edge column tests of this series.

The shear stresses given in table 5 of C.P. 110 are based on the relationship

$$\nu_c = K(\rho f_{cu})^{1/3} \text{ with } K = 1$$

When K = 3 the resulting values of ν_c correspond closely with 4 $\sqrt{f'_c}$ as in A.C.I. 318-77. The proposed values of ultimate shear stress have been obtained from:

$$\nu_c = 3(\rho f_{cu})^{1/3}$$

Table 5 summarizes the results obtained from the proposed method and while the method is very simplistic, it does deal with the variation of conditions (excluding the low reinforcement percentage of model number 3) set up within the test series in a fairly consistent manner.

Table 5 Comparison of Test Results with Predicted Capacities

MODEL NUMBER	SHEAR STRESS N/mm^2	A_1 mm^2	A_2 mm^2	REDUCTION FACTOR	V' kN	V_T kN	V_T/V'
1	2.14	37,760	20,060	0.79	33.9	66.2	1.95
2	2.47	24,780	13,275	0.74	24.26	50.1	2.06
3	1.91	24,780	13,275	0.74	18.76	32.7	1.74
4	2.01	26,208	12,480	0.72	18.06	37.1	2.05
5	2.04	10,608	10,608	0.77	16.66	31.8	1.91
6	1.78	28,808	15,080	0.705	18.92	37.6	1.99

If as suggested by Long (12), 5 $\sqrt{f_c'}$ was employed for values of v, then the ratio of V_T/V' would be reduced by approximately 20 per cent.

Needless to say other results with a much greater variation of boundary conditions would need to be considered before such a method could be established.

CONCLUSIONS

Whilst some slightly unsafe results of moment transfer were produced by the A.C.I. 318-77 method of analysis, the results from the tests have shown that the equivalent frame analysis used in the A.C.I. method produces more realistic estimates of moment transfer than the simple frame analysis of C.P. 110.

The method proposed in the text for the prediction of the ultimate shear capacity, which recognizes the importance of torsional cracking and moment transfer effects, was found to be more consistent than either C.P. 110 or A.C.I. 318-77.

Although limited in extent, the results obtained from the simple one column model tests have indicated that this type of model does provide an accurate estimate of the ultimate punching capacity of the slab/column junction when the moment transfer is equal to that found from a two column model.

ACKNOWLEDGEMENTS

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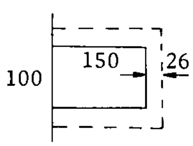
APPENDIX A



Model No. 4

Typical Calculations for Ultimate Capacity

effective depth, d = 52 mm
critical perimeter at d/2, 26 mm, from column face
critical area A_1 = 504 x 52 = 26208 mm^2
area reduction based on 75% of side length of critical perimeter reduction = 0.75 x 2 x 52 x 176 = 13,728 mm^2
$\therefore$ A^2 = 12,480 mm^2

Effective stress, $v_c = 3(\rho f_{cu})^{1/3}$.

For model number 4, ρ = 0.69% and f_{cu} = 44.1 N/mm^2 giving
$v_c = 3(0.0069 \times 44.1)^{1/3} = 2.01$ N/mm^2.

Magnification factor to allow for moment transfer is $(1 + (3.0e)/L)$. For an eccentricity, e = 236 mm (finite element analysis, Table 3) the resulting factor = 1.39.

Expressing this as a reduction factor the reciprocal is used, therefore the factor = 0.72.

Thus predicted ultimate capacity:

$$V' = \frac{0.72 \times 2.01 \times 12480}{1000}$$

$$= \underline{18.06 \text{ kN}}$$

THE PUNCHING STRENGTH OF UNBONDED POST-TENSIONED SLABS AT COLUMNS

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ABSTRACT The results of eight tests on post-tensioned unbonded flat slabs involving both shear and moment transfer at columns are presented. The amount of moment transferred to the columns is compared with predictions based on frame analysis and finite element analysis. A comparison is also made of the shear capacity with that predicted by the British and American codes and an improved method is proposed. The proposed method was found to give more consistent results than those found using either of the current recommendations.

INTRODUCTION

In recent years the use of post-tensioning has increased for medium to long span flat slab floors. These are now widely used in parking structures, various low rise and some high rise buildings in many countries. However many designers are reluctant to use this method of construction for at least two reasons:

i) Uncertainties exist about the ultimate load behaviour of unbonded flat slabs and only a relatively small number of tests have been carried out to examine this aspect.

ii) Punching failure at the slab column connection of these relatively thin slabs can often be a critical factor in design.

Most of the equations which have been proposed for predicting the punching capacity are purely empirical and are based on a limited range of test results (1, 2). Design recommendations published by The Concrete Society (3) are an extension of existing practice for reinforced concrete slabs, while the A.C.I. Committee 423 (4) proposed, for slabs, the use of the A.C.I. design equation for shear stress which is based on the results of tests on beams.

This paper deals with two series of tests on post-tensioned slabs. The first series (E) are edge panel tests while the second series (I) examine the behaviour of interior panels subject to shear and moment transfer. The results are compared with existing design methods and some amendments to these are proposed.

NOTATION

A_{ps}	area of prestressing tendons
A_s	area of tension reinforcement
b	width of section
c	column dimension
d	effective depth
f_c	specified cylinder compressive strength (lb/in^2)
f_{cu}	characteristic cube compressive strength
f_{sp}	split cylinder tensile strength
f_{cp}	initial level of average prestress
f_{up}	level of average prestress at slab failure
f_{pb}	tensile stress in tendons at slab failure
f_{pu}	characteristic strength of prestressing tendons
L	slab span (longer span in the case of interior columns)
M	total column moment
V	column axial load
v	critical design shear stress
v_u	nominal critical shear stress when failure occurs in the slab
v_c	ultimate allowable shear stress in concrete
x	distance from column face to shear perimeter
ρ_s	ordinary bonded reinforcement ratio in column region (A_s/bd)
ρ_{ps}	prestressing steel ratio in column region (A_{ps}/bd)
ρ_e	equivalent reinforcement ratio

ANALYSIS OF SLAB COLUMN CONNECTION WITH SHEAR AND MOMENT TRANSFER

The effect of moment being transferred from the slab to the column is generally considered to cause a higher shear stress along one side of the critical perimeter. The magnitude of this stress in relation to the average shear stress or column axial force is calculated in different ways in the American and British Codes. For example, C.P. 110 (5) considers the relationship between the critical and average shear stress to be constant for edge columns, irrespective of column or slab dimensions.

In an attempt to arrive at a more satisfactory idealisation of connections subject to combined loading two parametric studies have been carried out using finite element analyses to produce Figure 1(a) and (b). Figure 1(a) gives the relationship between the critical shear stress and the column axial load for edge columns subject to different levels of moment transfer. Various positions of the critical perimeter were considered and the appropriate graph can be selected on the basis of the distance from the column face to the shear perimeter.

For interior columns a similar family of curves has been derived, see Figure 1(b), although the term describing the position of the critical perimeter is in this case based on its distance from the column centre. This difference in the parameter designating the position of critical section in the two cases has similarities with the method of Di Stasio and Van Buren (6). Figure 1(a) and (b) does have limitations in that it applies to square columns and in that there is some error for span to depth ratios (long span in the interior column case) outside the usual range of 30-45.

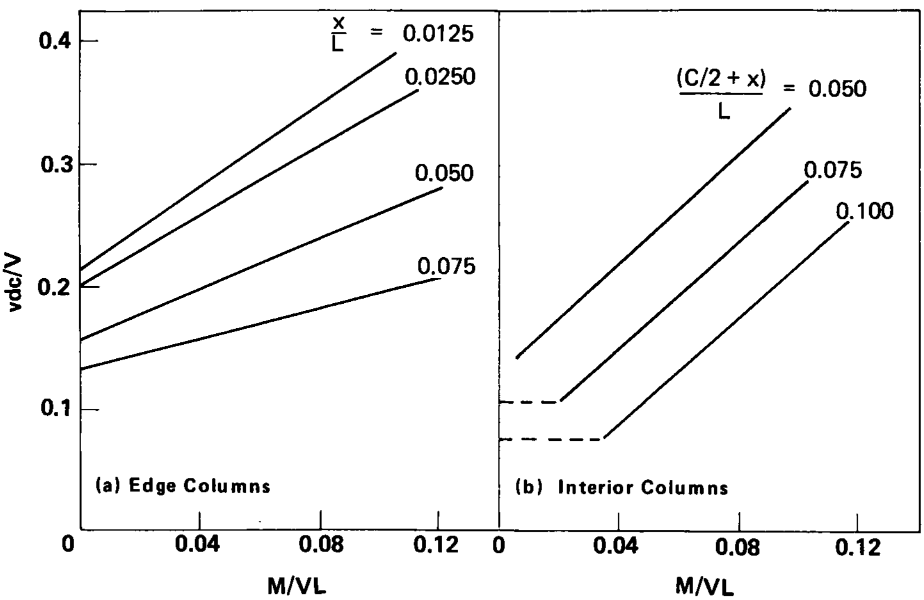

Figure 1 Relationship between Critical Shear Stress and Column Load for (a) Edge Columns and (b) Interior Columns

DESCRIPTION OF TEST PROGRAM

Most of the tests which have been reported in the literature (1, 2, 7, 8) were carried out on the common type of model which extends only to the assumed line of contraflexure. These are loaded along the edge while supported on a column stub. However for the present test series it was decided that all models should have dimensions equal to the span length in each direction, for the following reasons:

i) The normal method of analysis is to consider a strip of width equal to one bay which makes the model of direct significance.

ii) The stress increase in the prestressing tendons would be similar to those in a real structure as the length over which stress equalisation takes place is approximately the same.

iii) The average prestress is governed by the total number of tendons per bay and not the prestressing force near the column.

iv) With this type of model the boundary conditions could be made to represent fully the prototype structure.

It is interesting to note that recently Clarke (9) has also arrived at some of these conclusions after considering the previous tests on post-tensioned slabs. Hence models representing the shaded areas in Figure 2 were considered to be appropriate. The boundary and support conditions of both types of test are also shown. These boundary conditions enable one panel of a real structure to be accurately modelled with proper deflection profiles and the ability to redistribute moments (10).

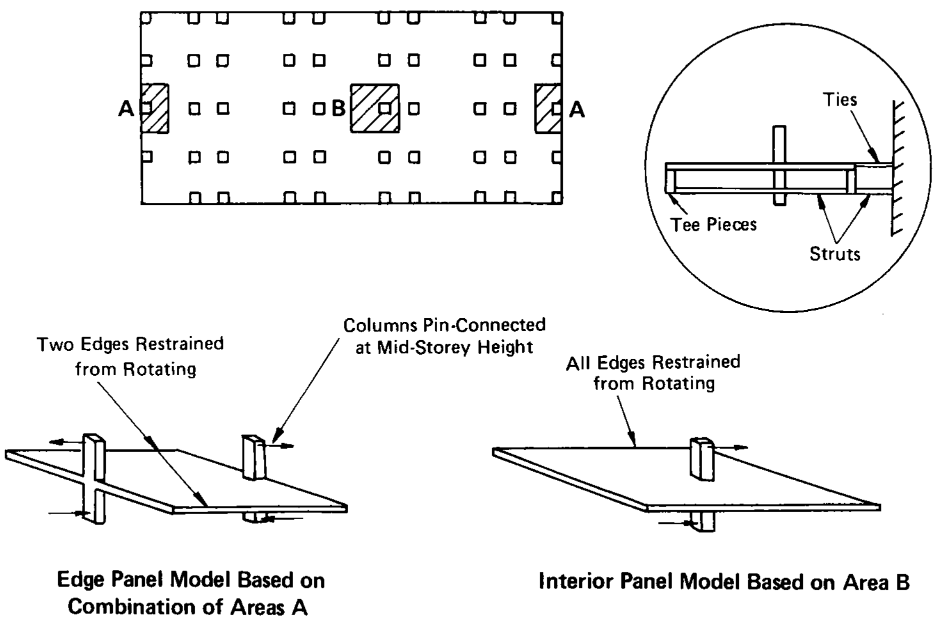

Figure 2 Details of Models

Details of Test Slabs

Various factors affecting the behaviour and ultimate load capacity have been investigated, including the level of average prestress, the distribution of prestress, and the column load eccentricity; Table 1 lists the important variables for each test.

Table 1 Variables Considered for Each Test

MODEL NUMBER	M/VL initial	BONDED REINFORCEMENT RATIO $100\rho_s$	LEVEL OF AVERAGE PRESTRESS N/mm^2	DISTRIBUTION OF PRESTRESS
E1	0.137	1.53	2.35	65% in column strip in both directions
E2	0.137	1.03	2.35	Banded in the direction of moment transfer
E3	0.137	1.03	3.52	"
E4	0.137	1.03	1.17	"
E5	0.137	1.03	2.35	"
I1	0.042	0.71	2.63	"
I2	0.044	0.71	2.68	"
I3	0.123	0.78	2.48	"

The models were approximately one-quarter scale giving an average span of about 2.5 m and slab depth of 57 mm. The columns were 166 mm square and extended to mid-storey height (0.56 m). The concrete used in all models was made with 6 mm crushed basalt and had an average cube strength of about 48 N/mm^2. To minimise any scale effects care was taken to ensure that the ratio of tensile to compressive strength was similar to that for the prototype mix.

Since small diameter 7-wire strand was not available,greased prestressing wires of 5 mm diameter were used as an alternative. The number of wires per span was approximately one half of that in the prototype slab which was considered satisfactory since the latter are commonly placed in pairs. With the exception of Model E1 the prestressing wires in one direction were banded in a zone extending about 0.15 L on either side of the column. In the other direction the distribution was uniform across the panel. This arrangement is quite common in practice since it simplifies the placing sequence of the strands.

Bonded reinforcement was supplied in each slab in the negative moment region directly above the column. This consisted of at least four bars in each direction and conformed with the current A.C.I. Committee 423 recommendations (4). The reinforcement was 6 mm diameter ribbed high yield steel with a distinct yield point at 516 N/mm^2.

Loading and Support System

In order to simplify measurement of the column moments the columns were pin-connected at mid-storey height. For the continuous edges, which were prevented from rotating as indicated in Figure 2, a system of tee-pieces and struts, similar to that used by Long and Masterson (11) was adopted. For the edges at right angles to the direction of moment transfer in the I series tests, additional ties and struts connected to a rigid vertical member were also employed (insert Figure 2).

Loading was applied through 16 points per panel, representing a uniformly distributed load. For the interior column tests each span could be loaded independently so that dead load could be applied to both spans with increasing live load also on one span. The exception to this loading arrangement was model I1 which extended to mid-span, but the edges of which were not restrained. In this case the loading was applied along a ring at a position consistent with the line of contraflexure.

Stressing Procedure

Because the models were scaled versions of a real slab special provision was required during post-tensioning. A proportion of the prestress was applied at an early stage to facilitate handling and to prevent cracking due to the self weight of the slab. The remaining prestress, with the compensating dead load, was then applied in three stages a short time before testing. In this balanced load position the slab was in a state of stress across its depth which was almost uniform and equal to the average prestress. The wires which were tensioned in the first operation were found not to have lost any significant tention on completion of the total stressing.

Instrumentation

Load cells were used at both the top and bottom columns to measure the horizontal reactions at the pin-connections, since the bottom column rested on virtually frictionless roller bearings. These supports were regularly adjusted, in most tests, to maintain zero lateral movement of the column ends. Throughout testing the force in eight of the prestressing wires was monitored using suitable load cells. The rotation of the slab at various points, the level of strain in the bonded reinforcement and the deflection of the slab were also measured.

RESULTS

The behaviour of all slabs in each test series was similar up to initial formation of cracks. In this elastic range the slabs recovered practically all deflection upon unloading. After cracking the deflections began to increase at a greater rate although the presence of bonded reinforcement in all the slabs ensured reasonably ductile behaviour.

In the edge column models initial cracks occurred along the inside transverse face of the columns while at about the same stage torsion cracks extended from these to the free edge. Further loading caused cracks to extend radially from the columns and just prior to failure some flexural cracking occurred at mid-span. In the interior column models initial cracking took place along the face of the column adjacent to the heavily loaded span. Further loading caused radial cracking and in some cases a transverse crack formed across the slab through the column line. In each test punching failure took place quite suddenly. The failure zone was significantly more broken than for reinforced concrete slabs and was unsymmetrical for the interior column tests. As expected most of the failure cone was in the heavily loaded panel. Details of the ultimate capacity of the tests and relevant concrete strengths are summarised in Table 2.

Table 2 Summary of Test Results

MODEL NUMBER	AVERAGE CUBE STRENGTH N/mm^2	AVERAGE TENSILE STRENGTH N/mm^2	ULTIMATE CAPACITY	
			Shear, kN	Moment, kNm
E1	48	4.27	47	13.1
E2	50	-	49	13.8
E3	48	4.28	56	15.7
E4	48	4.02	40	9.4
E5	45	4.10	52	16.3
I1	48	4.78	100	11.9
I2	48	4.10	103	8.3
I3	46	4.28	75	6.6

Column Moments

Up to initial cracking the total column moments (sum of both top and bottom column) in the E series tests increased linearly with increasing live load, see Figure 3(a). After cracking the rate of increase of this moment decreased. For models E2, E3, and E5, which were heavily reinforced in the column region, the stiffness of the slab column connection was only marginally reduced. However, in the case of models E1, and E4 the stiffness of the connection was much reduced resulting in lower bending moments being transferred to the column after cracking. The

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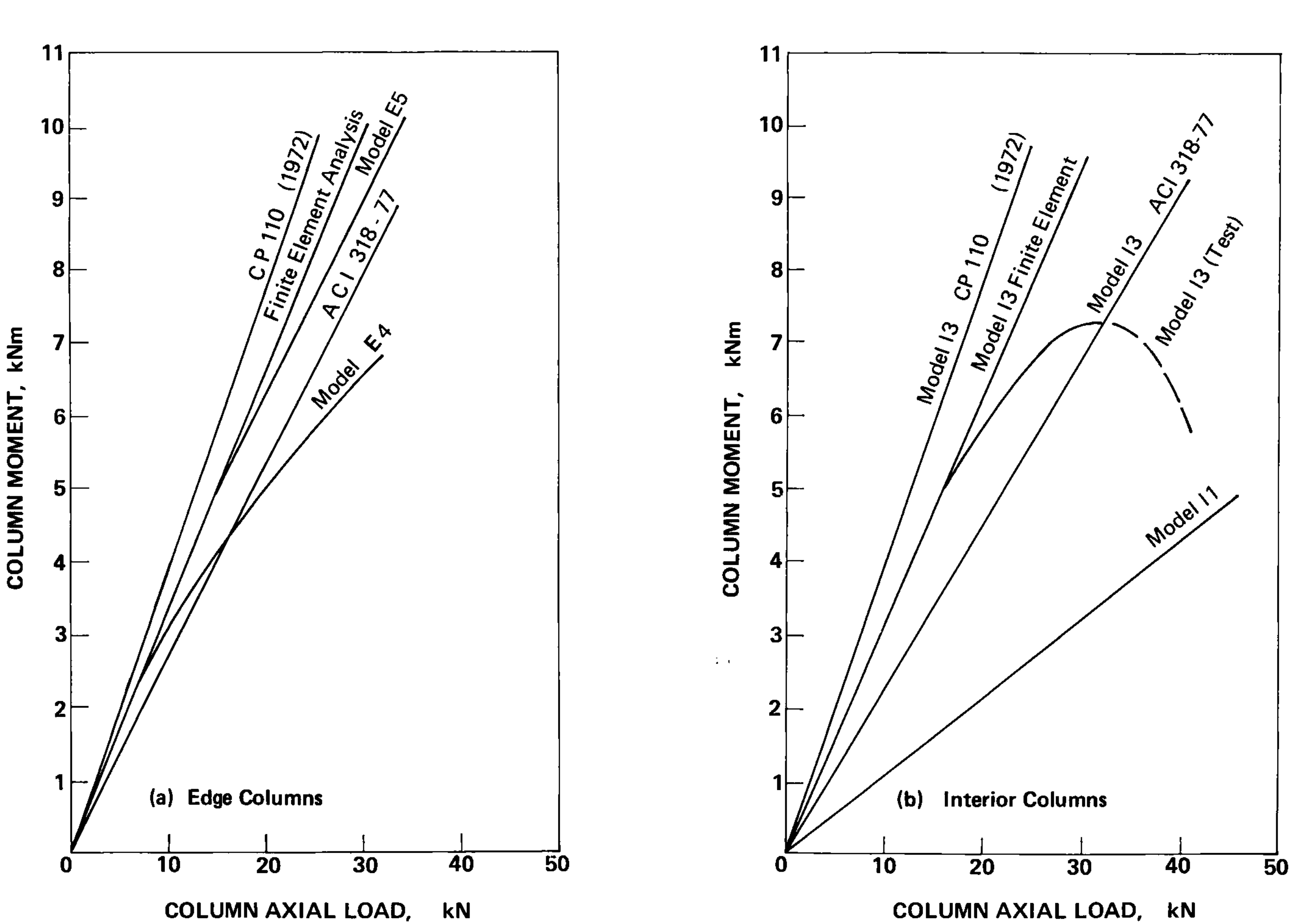

Figure 3 Variation of Column Moment with Applied Load for (a) Edge Columns and (b) Interior Columns

significant difference between these results and those of equivalent reinforced concrete slabs (10) is that cracking at mid-span did not occur in the post-tensioned slabs until just prior to failure. Hence there is significant redistribution from the negative moment region at the column to the positive moments at mid-span.

For model I1 the column moment increased linearly because the model was statically determinate. For the other interior column models the column moments increased linearly until near failure when the rate of increase of moment decreased considerably, see Figure 3(b). This again demonstrates the ability of these more sophisticated models to redistribute moments in a similar way to a real structure.

In all cases the simple frame analysis of C.P.110 (5) predicts moments which are in excess of those measured, even before cracking has occurred, see Figuure 3(a) and (b). The values predicted by finite element analyses are in good agreement with the test results up to cracking but take no account of the loss of stiffness of the connection. A less conservative estimate of the column moment can be obtained from the equivalent frame analysis of A.C.I. 318-77 (12). Although the test results are underestimated at low load levels the correlation at ultimate load has been found to be quite good.

Ultimate Capacity

The failure loads for the eight tests are compared in Table 3 with those predicted by the British and American code methods. The effect of limiting the allowable shear stress to that value consistent with 3 per cent reinforcement can be seen in the constant values predicted by C.P.110 (5) for edge columns. The single magnification factor of 1.25 for edge columns according to C.P.110 can be quite unconservative as, for example, in the case of model E4 for which the factor of safety is only 1.25; this is considerably lower than is evident in the interior column tests (I series).

Within each series the A.C.I. 318-77 predictions are more consistent with the test results giving factors of safety of about 1.8 for the interior columns and about 2.5 for edge columns. This discrepency between the two series is due to the method of calculating the critical shear stress at columns subject to moment transfer. This is shown more clearly in Figure 4, in which these results and those of other authors (2,7 & 8) are compared with the design equation for the allowable stress.

Table 3 Comparison of Code Predictions

MODEL NUMBER	FAILURE LOADS, kN		
	Test Value	Predicted by C.P.110,	Predicted by A.C.I.
E1	47	32	20
E2	49	32	20
E3	56	32	23
E4	40	32	17
E5	52	32	19
I1	100	42	56
I2	103	42	60
I3	75	26	47

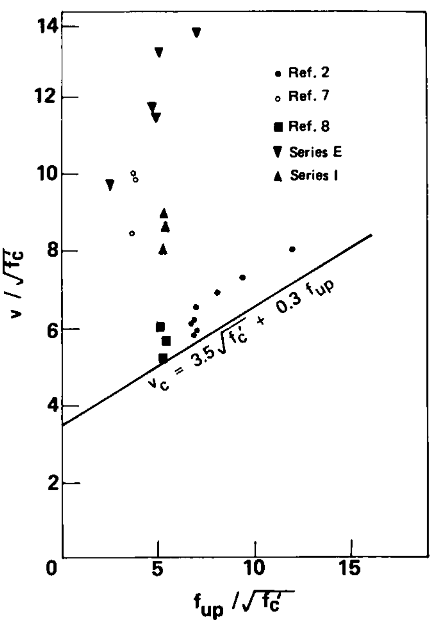

Figure 4 Comparison of Test Results with ACI Design Method (Imperial Units)

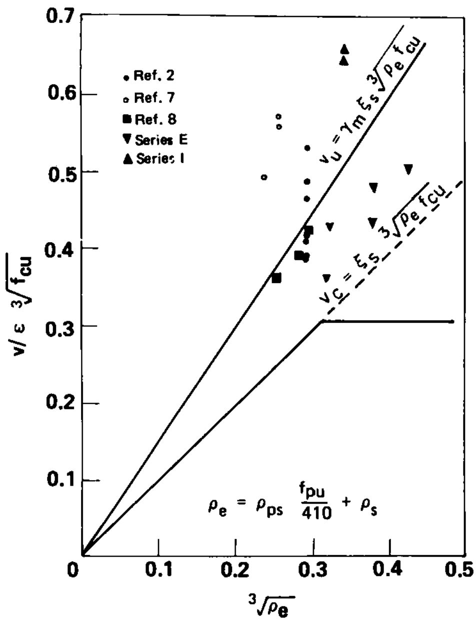

Figure 5 Comparison of Test Results with CP110/Concrete Society Design Method

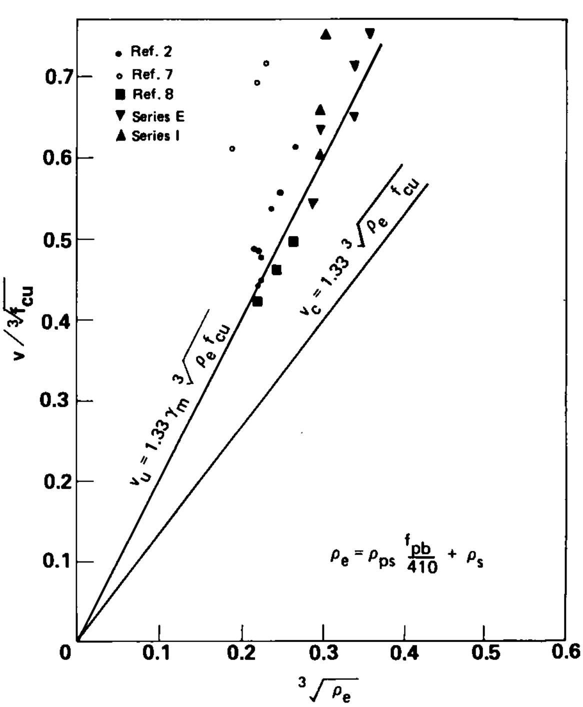

Figure 6 Comparison of Test Results with Proposed Design Method

The safety factor for slab column connections subject to moment transfer is significantly greater than for simple shear models. Although the equation is conservative for all the tests which were considered, it can prove unreliable when insufficient reinforcement is placed directly over edge columns (13). Hence it is reasonable to conclude that the equation should include as one of its parameters the reinforcement and prestress level in the column region, and not the average over the full panel. From this aspect the C.P. 110 equation for the allowable stress has some advantage. However the simplified treatment of moment transfer leads to estimates of ultimate load for edge columns which are significantly less safe than for interior columns subject to transfer of moment, see Figure 5.

The proposed method of determining the critical shear stress in moment transfer situations, Figure 1(a) and (b), in conjunction with the C.P. 110 perimeter at 1½h from the column is tested against the same results in Figure 6, the M/V value being calculated from finite element analyses or a suitable equivalent frame analysis (14). The equivalent reinforcement percentage was taken as:

$$\rho_e = \rho_{ps}\frac{f_{pb}}{410} + \rho_s$$

This change from the Concrete Society's recommendations (3) is a logical step since unbonded tendons never reach stresses near their ultimate value in a real slab structure. The value of f_{pb} which was used was that measured in the tests but the F.I.P./C.E.B. estimate (15) would be satisfactory for realistic slab configurations.

The correlation with the test results shown in Figure 6 is good in spite of the wide range of variables present in the tests. The nominal shear stress at failure could be given by:

$$v_u = 2.0\sqrt[3]{\rho_e f_{cu}}$$

For design purposes the partial safety factor for materials could be included in the equation giving:

$$v_c = 1.33\sqrt[3]{\rho_e f_{cu}}$$

In the above analyses it has been assumed that for test results with good quality control the characteristic strength does not differ greatly from the mean strength. For real slabs higher factors of safety may be obtained since the mean strength usually exceeds the characteristic strength by a significant margin.

CONCLUSIONS

On the basis of the eight test results reported and those of other researchers the following conclusions have been reached.

Up to cracking the level of moment absorbed by the columns is accurately predicted by finite element analysis. The A.C.I. 318-77 equivalent frame analysis gives good correlation with the test results near ultimate, while C.P. 110 (1972) overestimates the moment at all stages.

Using both the British and American codes the critical shear stresses at connections subject to moment transfer are inconsistent with the stresses in simple shear models. With the exception of the C.P.110 edge column approach, the codes are more conservative when moment as well as shear are transferred to the column.

The proposed relationship between axial shear force and maximum shear stress take account of variations in column size and stiffness and provide ultimate load predictions which are consistent with test results.

The equivalent reinforcement percentage proposed in this paper gives better correlation than that in the Concrete Society Report (3). In addition the results do not show any trend towards a limit on the ultimate shear stress corresponding to an equivalent reinforcement percentage of 3.

ACKNOWLEDGEMENTS

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TESTS ON PRESTRESSED, PARTIALLY PRESTRESSED AND REINFORCED CONCRETE SLABS

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ABSTRACT Deflections, crack widths and ultimate loads have been measured for slabs containing various proportions of prestressed and non-prestressed reinforcement. The experimental results have been compared with values computed from the theoretical formulae which have been developed mainly from tests on beams. The study indicates that some modifications to the formulae might be necessary when used for slabs, especially those with a high content of prestressed reinforcement. A concept of the degree of warning of failure has been introduced and it is suggested that modification factors be applied to the partial safety factors given in the existing codes of practice, the modification factors depending on the degree of warning.

INTRODUCTION

The majority of tests carried out by various researchers on prestressed concrete structures with and without non-prestressed reinforcement have been on beams of various cross sections. The effective moment of intertia (I_e) method used in A.C.I. 318:1971 (1) was developed by Branson (2) in 1965 for determining deflections in reinforced concrete beams. In 1970 it was shown by Shaikh and Branson (3) to apply equally well to rectangular section prestressed beams with and without non-prestressed reinforcement. Bennett and Veerasubramanian (4) carried out tests in 1972 on beams of I and T sections and showed that the shape of the cross section influenced the deflections in beams and that the I_e method was conservative for monolithic and composite T sections. The same authors found that in prestressed beams with non-prestressed reinforcement, the width of flexural cracks was related to the increase of stress in the non-prestressed reinforcement but was otherwise independent of the shape of the cross section.

Thin prestressed and partially prestressed slabs are now being used in factory, commercial and other buildings and the formulae used in their design are the same as those developed for beams. Tests were, therefore, carried out at the department of Building Science, University of Singapore, to examine the behaviour of thin prestressed slabs, with and without non-prestressed reinforcement, in comparison with tests on reinforced concrete slabs.

NOTATION

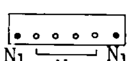
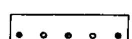
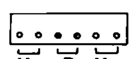
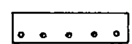
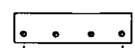
A_{ps} Area of prestressing steel
A_{sn} Area of non-prestressed steel
b Breadth of slab
d Effective depth of tensile steel
f_{cu} Characteristic concrete cube strength
f_{pu} Characteristic strength of prestressing steel
f_y Characteristic strength of non-prestressed reinforcement

EXPERIMENTAL WORK

Test Slabs

Ten slabs were tested, all having the same cross sectional dimensions but having varying amounts of prestressed and non-prestressed reinforcement. Five different arrangements of reinforcement were used, two slabs being tested for each arrangement. The proportions of prestressed and non-prestressed reinforcement for each arrangement were adjusted so that the slabs had a similar calculated ultimate moment of resistance for all the arrangements. The two slabs designated S1 were reinforced concrete slabs with no prestress, while the two slabs designated S5 were fully prestressed, with 4 prestressing wires of 5 mm diameter and no non-prestressed reinforcement. The intermediate slabs designated S2, S3 and S4 contained respectively 1, 2 and 3 prestressing wires of 5 mm diameter and varying amounts of non-prestressed reinforcement. Details of the slabs are given in Table 1.

Table 1 Details of Test Slabs

PROPERTY	SLAB TYPE				
	S1	S2	S3	S4	S5
Steel Arrangement	 N_1 N_2 N_1	 N_1 N_2 P N_2 N_1	 N_2 P N_2	 N_2 P N_2	 P
A_{ps}, mm^2	0.00	19.63	39.27	58.90	78.54
A_{sn}, mm^2	270.18	213.63	113.10	56.55	0.00
$\rho_1 = \frac{A_{ps}}{bd}$, %	0.00	0.13	0.26	0.39	0.52
$\rho_2 = \frac{A_{sn}}{bd}$, %	1.80	1.42	0.754	0.377	0.00
$\rho_1 \frac{f_{pu}}{f_{cu}} + \rho_2 \frac{f_y}{f_{cu}}$	0.17	0.18	0.17	0.18	0.19
$\frac{A_{ps}f_{pu}}{A_{ps}f_{pu} + A_{sn}f_y}$	0.000	0.260	0.324	0.799	1.000

Note: All slabs 3000 mm x 300 mm x 75 mm thick, with d = 50 mm, tested on a simply supported span of 2850 mm.
P = 5 mm dia. prestressing wire; N_1 = 10 mm dia. non-prestressed steel;
N_2 = 6 mm dia. non-prestressed steel

Materials

The prestressing reinforcement used in the slabs was 5 mm diameter hard-drawn wires with a characteristic strength of 1570 N/mm^2, while the non-prestressed reinforcement was hot rolled high yield steel with a characteristic strength of 410 N/mm^2. The concrete used for all slabs had a characteristic strength of 30.2 N/mm^2 at 7 days and 43.3 N/mm^2 at 28 days, thirty 100 mm cubes being tested at each age.

Test Procedure

The slabs,which were 300 mm wide by 75 mm deep and 3000 mm long, were simply supported over a clear span of 2850 mm with loads applied at the third points of the span. Electrical resistance strain gauges were used to measure strains on the upper surface of the slab and on the surface of the prestressing wires. The strain gauges used on the prestressing wires were suitable for long term measurements and were made moisture-proof by coating with wax and wrapping well with a tape. The deflections at the centre of each slab were measured with dial gauges reading to 0.01 mm.

Small scales attached to the slab were included in close-up pictures which were taken of cracks in the slab. Crack widths were measured later using a Topcon measuring projector with a maximum magnification of 100. Using this procedure it was possible to measure crack widths down to 0.07 mm.

Test Results

A comparison between computed and observed ultimate loads is presented in Table 2.

Table 2 Observed and Computed Ultimate Load Capacities of Slabs

SLAB TYPE	ULTIMATE LOAD, kN				
	Computed 1*	Computed 2†	Observed		
			Individual		Mean
S1	9.1	7.8	10.6	11.2	10.9
S2	9.6	8.3	8.4	9.2	8.8
S3	8.8	7.6	9.1	9.1	9.1
S4	9.4	8.1	9.1	9.9	9.5
S5	9.9	8.6	8.6	9.0	8.8

* Ultimate load computed without considering partial safety factors for materials
† Computed from C.P. 110 (6)

The computed and observed relationships between loads and deflections at midspan for the five slab types tested are shown in Figure 1, in conjunction with computed and observed crack widths for the slab types S2 to S5. The observed deflections plotted in Figure 1 are the averages of the deflections of two individual slabs of each type. The computed deflections were calculated using I_e, the equivalent moment of inertia given in A.C.I. 318:1971 (1). The crack widths presented in Figure 1 were taken from the second slab of each type. The computed crack widths were calculated using the C.E.B.-F.I.P. recommendations (5).

DISCUSSION OF TEST RESULTS

Deflections

There is an acceptable correlation between observed and computed deflections for the four slab types S1 to S4, the difference between observed and computed deflections being within 20 per cent of the observed deflections at 50 per cent of the ultimate load and within 16 per cent of the observed deflections at 75 per cent of the ultimate load. For slab type S5, which contained only prestressed reinforcement, the correlation is satisfactory up to approximately 60 per cent of the ultimate load, which is just prior to the formation of cracks in the slab. After the formation of cracks in this slab, the difference between observed and computed deflections increased rapidly as the load was increased, the difference being equal to 42 per cent of the observed deflection at 75 per cent of the ultimate load.

Number of Cracks

The higher observed deflections in slabs of type S5 after the onset of cracking appears to be due to the low reinforcement ratio used together with a considerable destruction of the bond between the steel and concrete, indicated by the occurance of only four cracks in the first slab and five cracks in the second slab of type S5. The improvement in bond with increasing reinforcement is indicated by the increase in the number of cracks in slabs with higher reinforcement ratios; the average number of cracks was thirteen in slabs of type S4, eighteen in slabs of type S3, twenty in slabs of type S2 and twenty one in slabs of type S1.

Crack Widths

The crack widths computed using the C.E.B.-F.I.P. recommendations showed acceptable agreement with the observed crack widths for slab types S2 and S3. The computed crack widths for slab type S4 were very much on the conservative side, being approximately twice the observed crack widths. The large deflections observed for slab type S5 after the formation of cracks were accompanied by cracks whose widths were many times larger than the computed crack widths.

Ultimate Loads

The slabs used in these tests were not expressly designed to carry any working load satisfying the serviceability limit state, but were designed to carry approximately the same ultime loads. The observed average ultimate load for the reinforced concrete slabs S1 was 20 per cent higher than the computed load without partial safety factors, see Table 1. At the top end of the range of prestressed slabs, the observed average ultimate load for the slabs S5 containing only prestressed steel was 11 per cent lower than the computed load, the low ultimate loads being attributable to the destruction of bond due to low reinforcement ratio.

Comparison of Performance of Slabs

The load-deflection relationships for the five slab types are compared in Figure 2. If it is assumed that the working loads are 50 per cent of the ultimate loads, slab types S1, S2 and S3 are clearly unacceptable with reference to the serviceability

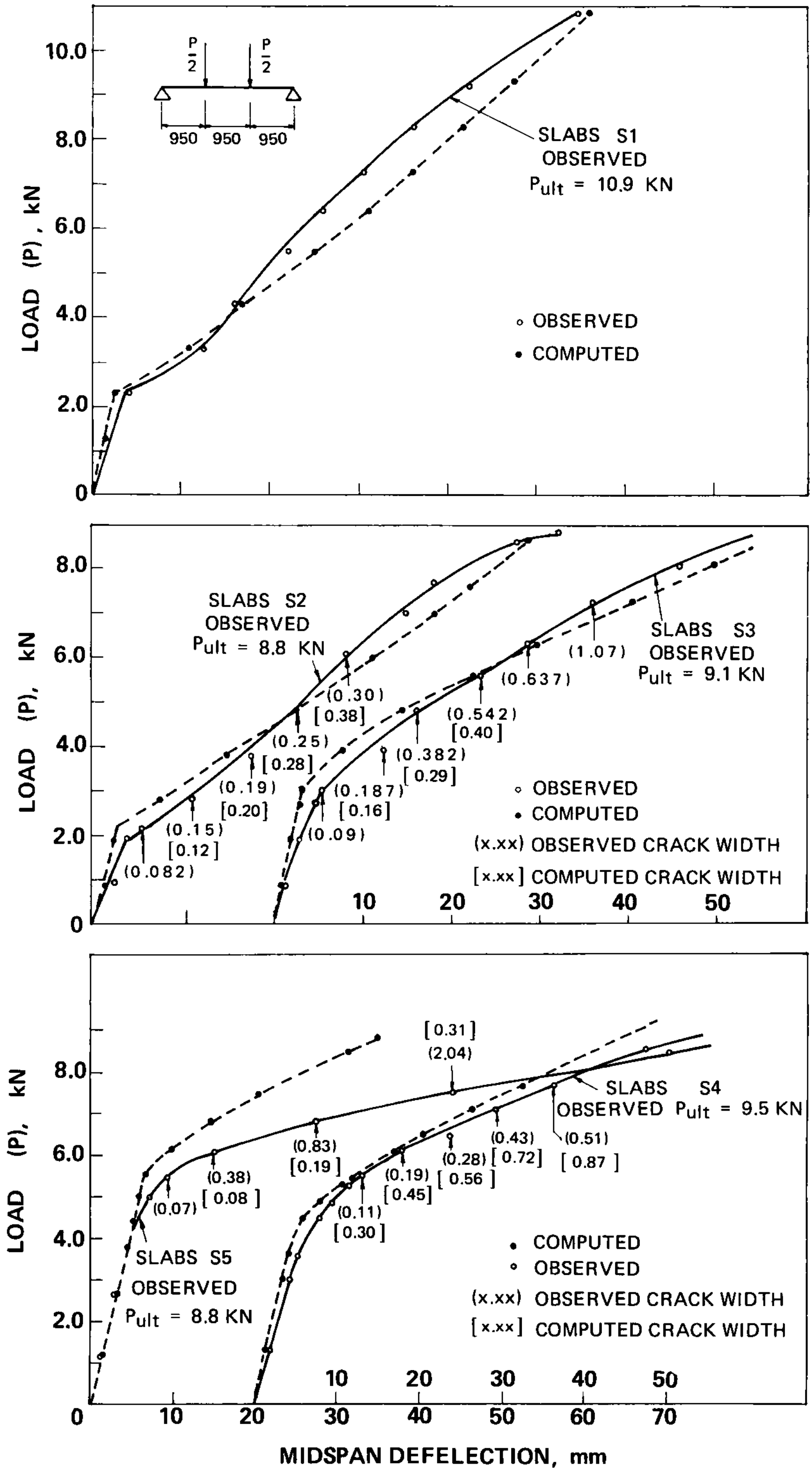


Figure 1 Relationship Between Load and Midspan Deflection

limit state of deflection, the crack widths at the corresponding loads also being higher than the maximum permissible value of 0.2 mm for Class 111 structures. The slab types S4 and S5 would clearly be suitable in practice, the deflections at 50 per cent of their ultimate loads being within the acceptable limits; in addition slab S4 had only micro-cracks while slabs S5 were uncracked at 50 per cent of their ultimate loads. However, the performance of slabs S4 was far superior to that of slabs S5 at loads above the cracking load, the deflections and crack widths increasing at a much slower rate in slabs S4 than in slabs S5. Also, the ratio of observed to computed ultimate loads was higher for slabs S4 than for slabs S5. It can therefore be concluded that there is an optimum ratio of prestressed to non-prestressed reinforcement which gives the best overall performance of the slabs at service loads and above. In slabs S4, the areas of prestressed and non-prestressed reinforcement were approximately equal and 80 per cent of the tensile force in the steel at ultimate load was provided by the prestressed reinforcement.

DEGREE OF WARNING

Warning of failure, by way of large deflections, is desirable at about 75 per cent of the ultimate loads so that necessary action can be taken to relieve the floors of large loads, thus reducing the risk of complete structural collapse. This is particularly important in commercial and factory buildings where accidental overloading of floors is common. Also, since poor quality control on site might result in weaker concrete in the structure than indicated by the cube tests, and a lower prestress in the steel than designed for, a warning of failure is again important even if design loads are not exceeded. Although wide cracks in slabs may also give warning of failure where they are exposed, in most buildings the soffits of slabs are often concealed and the cracks are not visible.

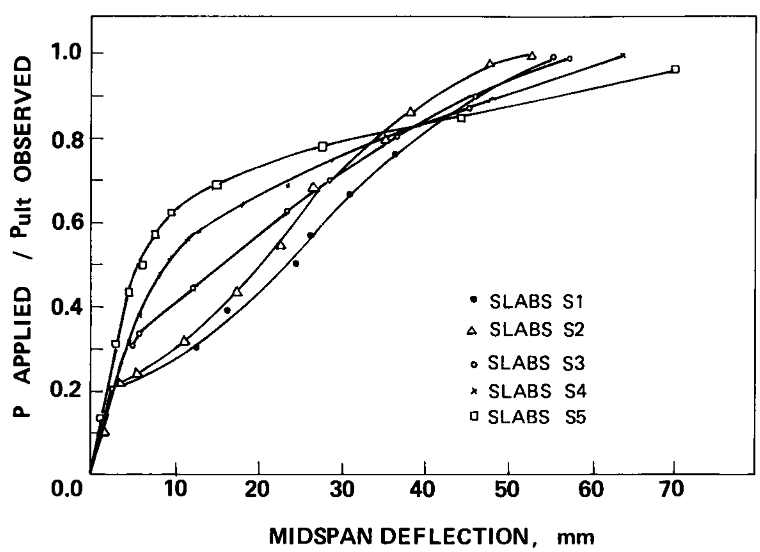


Figure 2 Comparison of Deflection Curves

As can be seen from Figure 2, the degree of warning reduces with an increase in the degree of prestress. It also follows that the degree of warning reduces as the difference between cracking load and failure load reduces, the degree of warning approaching zero when the cracking load and failure load are equal. The degree of warning can be defined as the ratio of the actual deflection to the deflection required to give warning of failure, the deflections being considered at a proportion of the ultimate load at which warning is required. Careful investigations are necessary to establish the value of desirable deflection required to give the necessary warning of failure and the proportion of ultimate load at which such warning is required. Although partial safety factors for materials and loads are considered in the C.E.P.-F.I.P. recommendations (5) and the British Code of Practice, C.P. 110 (6), it appears necessary to use modification factors to the suggested values of partial safety factors depending on the degree of warning required.

CONCLUSIONS

The formulae for deflections, crack widths and ultimate loads for prestressed concrete members which have been developed from tests on beams require some modification when applied to thin prestressed concrete slabs, especially when the slabs contain prestressed steel only.

The overall performance of the slabs at service loads and above is best when the ratio of the areas of prestressed to non-prestressed reinforcement is an optimum. From the test results on slabs reported in this paper, the overall performance was best when the areas of prestressed and non-prestressed reinforcement were approximately equal, 80 per cent of the tensile force in the reinforcement at ultimate load being provided by the prestressed reinforcements.

Warning of failure by means of large deflections at a load equal to some proportion of the ultimate load is necessary to reduce the risk of complete structural collapse. The degree of warning reduces with an increase in the degree of prestress and it appears necessary to use modification factors for the partial safety factors for materials and loads depending on the degree of warning required.

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DISCUSSION

Mikael W. Braestrup. I should like to make a comment on the keynote address by Professor Cusens. I think it was the very first Table to which you referred, one square of which was conspicuously blank. You suggested that this was due to the fact that anything that could be said about elastic design of slabs had already been said. I should rather think that this is not the case. You mentioned the Wood-Armer equations. I am not familiar with those but I should think that they are actually plastic design, since I imagine that they assume that the steel is yielding, no matter what the curvature of the plate is at the particular point under consideration. I imagine that they also assign some plastic properties to the concrete, possibly by some stress block factors. What we actually do is to use the lower bound theory of plasticity, employing as our moment distribution the one that we get from analysis of the slab as if it was an elastic plate, which does not mean that we are using elastic design. We are still relying on the applicability of plasticity to the slab. That is just a point I should like to make, I do not know if you, Professor Cusens, or any of the other authors, would like to comment on it.

Anthony R. Cusens. I agree with what the speaker has said. The Table to which you refer is Table 2 in my paper, which shows the papers in this session and the next session which are relevant to methods of analysis; the blank is due to the fact that there are no papers dealing specifically with elastic methods. I was therefore posing the question whether we have said everything we have to say about elastic methods, with a note of regret, rather than anything else. Indeed I was trying to urge that we still have a use for elastic methods even if we are thinking of applying them to situations which have plastic behaviour involved eventually.

Leslie A. Clark. If I could just comment on what Dr. Braestrup said. I am very glad he has raised this because it is a point which is not appreciated by many people. If one carries out an elastic analysis and then uses an ultimate load method of design of the section, the result is a safe lower bound design. We are not using the elastic analysis because it predicts what happens at collapse; the only reason we are using the elastic analysis is because it satisfies equilibrium.

P. Bhatt. Basically there are three methods of slab design to consider: plastic, elastic and optimum. As far as I understand it, as Dr. Clark said, the slab design essentially is going to depend first upon the equilibrium forces which we choose, and then upon any yield criteria which we adopt. Now if you assume that the only yield criterion which is correct is that due to Johansen and go on from there, I think that using the elastic moment and Johansen's yield criterion, and following Wood's procedure, you are really going to get almost optimum design of the slab. The only thing that you can discuss is how much of the constraint we can, in fact, release in order that we can try to reduce the amount of reinforcement which is going to be used. In other words, we are almost talking about something similar to the old design of beams where you started off with an elastic design and then attempted to reduce it by 15 or 30 per cent to obtain a much more 'comfortable' moment distribution from which to design. So my basic question is should we really still keep continuing to look at three or four different procedures for the design of slabs, or should we not really treat some of these things as a purely historical process and drop some of them and try to concentrate our attention really on the elastic designs, use a proper yield criterion and then go on and see how much of the constraint you can really reduce in order to get a much more equitable moment distribution or reinforcement distribution?

Peter G. Lowe. In a sense I think your question is begging some of the questions which we are trying to raise today. I would dispute the point about elastic solutions, when interpreted as a lower bound, representing a possible approach to an optimum solution; I think that has now been shown to be true only in certain cases. While personally, since I have an interest in one of the camps, I am not trying to decry the elastic solution, which in fact is obviously a very important solution, the point I would make is that when people use the word elastic they should qualify it by what sort of elastic they are talking about. This is a point I was going to put to Professor Cusens, but he is a little too close to ask at the moment. I would argue that virtually every time the word elastic has been used today it should be prefaced by the words *isotropic homogeneous*, and really what we are lacking is solutions to non-isotropic, non-homogeneous elastic problems before we can start making a choice between the possibilities offered to us. I shall just leave it at that.

Anthony R. Cusens. I agree with Professor Lowe. We know the answers to a fairly wide range of practical problems, but they are answers to a wide range of fairly easy problems. I think that before we can throw out particular methods or particular approaches we need to be much more sure of our ground. This is one of the reasons that I made a plea at the end of my paper for people to look at more complex problems, more difficult problems and for more experimental work, please.

Leslie A. Clark. I am not sure if I have interpreted the question correctly, but I think that what you were implying is that elastic moment fields tend to be very 'peaky' and it would be nice if we could smooth these peaks out. Well this is what designers tend to do anyway, but they do it by intuition to a large extent. I think this is the area where non-linear analysis can be very useful in that we can justify this intuition and I see the greatest use of non-linear analysis in calibrating other types of analyses which we can do more simply. I am doing some work at present with Dr. Cope at Liverpool, where we are attempting to do this in the field of bridges. We are trying to come up with lower bound moment fields which smooth out the stress concentrations which one gets in obtuse corners of skew slabs, for example. We are doing this by non-linear analysis, but then hoping that we can come up with some simple analyses afterwards.

D.N. Trikha. I would like to make a point in respect of the comments by Professor Cusens in his keynote address. He stated that the finite element method is too uneconomical to be used in the design office and that he favoured experimental work. Well, I think quite differently. I believe that the finite element method is the method which has come to stay now; it has proved its efficiency for all types of slabs: any shapes, any loading, and any boundary condition. Experimental research workers should now try to think differently, instead of thinking of parametrical studies, changing the amount of steel reinforcement, slab shapes and so on, they should now rely on the finite element method and give us data so that we can prove that in all situations and in all circumstances the finite element method does stand up.

Anthony R. Cusens. You can always crack a nut with a sledgehammer, but you do not always need a sledgehammer. That is really my comment on that one. The finite element method is a very powerful technique, but you do not want to use it every time you are designing a slab. By all means use it, as I said in my paper, as a research tool, but it is not something you want to use for slab design in a design office. By all means use it to make parameter studies and then use the results of those parameter studies to produce tables or design curves. That is using it as a research tool, which is excellent, but I do not think it is an everyday design office instrument.

R. Colin Deacon. Mr. Chairman, I would like to sling a spanner in the works if I may because I found reading the papers and listening this afternoon to this session and reading the papers for tomorrow morning's session most depressing. It seems to me that the gulf between the academic and the practical engineer is colossal, judging from these papers. I wonder if any of these gentlemen have ever had gum boots on their feet, paddled around in mud with the weather falling upon them, contractors falling upon them, resident engineers falling upon them. I am sure this work is of first-rate quality, I do not decry it at all. All I say is for heaven's sake do not stop there, bring it down to reality. We have been shown some slides this afternoon of reinforcement arrangements. One can take Professor Cusen's view that it will never get to site in that form, but even to conceive those arrangements of steel is incredible. I sometimes wonder whether research workers should not start from reality, put a few construction joints in their models and see how these affect their theory because I suspect that the real structure is so divorced from theory that the theory is probably not appropriate. I did not realise that we had so many problems with slab design; I have always been brought up with the theory of three moments: $WL^2/8$, $WL^2/10$ or $WL^2/12$ and these have covered all my problems.

Peter G. Lowe. It was with some trepidation that I wrote a paper at all, largely because I thought that this might possibly be one of the reactions. First of all shall I say that although I am not at the present time caught up in actual construction, I have been. That does not prevent me from daring to discuss some of these problems in other ways. Let us bring it down to earth and talk about optimum solutions; I think that unfortunately the whole field has had bad press in the sense that some of the earliest work was perhaps over-reported and subsequently the situation has not quite been put right yet. Given the spirit of the question, however, I would argue that there is a place for some amount of activity along these extreme lines. Whether or not they are the lines we are talking about at the moment, does not necessarily matter, but I would argue that only by having some active work going on, on the fringes if you like, can one be stirring up some of the questions which personally I feel we have got to discuss at some level, in some place. The expression $WL^2/8$ is fine in many cases but I think we would be in a worse state now had we not considered certain other alternatives earlier. Thus

I have no hesitation in stating that this work has some relevance. Possibly it is not of the type you were looking for today, for which I am sorry, but personally I am not in the least bit deflected, I am not going to stop working on it because comments of this sort are made. I appreciate the reasons for them being made, but I feel that somebody has to do something in these areas.

Leslie A. Clark. I would just like to comment on two aspects of that question. First of all $WL^2/10$ was mentioned; now this is rather interesting because $WL^2/10$ does not satisfy equilibrium, so Mr. Deacon was very happy to carry on designing slabs which did not satisfy equilibrium. He will answer that by saying that they stood up so why should he care? The point is that they stood up because he was ignoring certain things and if I was in his position I would like to know why they stood up. I like to think that academics can provide that information. So I do not think you can decry this type of work because it has justified a lot of things that have been done intuitively in design in the past. Coming to the second point, which was an attack on academics in general; I have recently been involved on the periphery of the development of a code of practice. I personally think that that code is unsatisfactory and one of the reasons, I think, it is unsatisfactory is that the code committee that produced it consisted mainly of practising engineers who were no longer designing. If there had been some academics actually involved with structural behaviour through research and, equally, if there had been some practising engineers who were actually designing involved with the code, I think a better code would have been produced. So I cannot accept anything Mr. Deacon has said; if he wants to carry on in ignorance, not knowing why he is doing what he is doing, let him do so.

Chairman. Clearly we believe that the best form of defence is attack.

John D. Peacock. While the academics are doing their job, and one recognises that it is helpful although it does take rather longer than it should do to become practically useful, can they please remember that nearly all the slabs we have to design in practice have holes and openings in them. Can they also please remember that most slabs are not square or rectangular but often have skew sides and that the edges are neither free nor continuous nor clamped. The practical problems are rather different to the ones at which the academics look.

Leslie A. Clark. Well I have done quite a bit of work on skew slabs so I think I have helped the practising engineer out to a certain extent. The way I would answer that, however, is that if you cannot analyse something with a nicely defined edge, you have got no chance of analysing the type of slab which you have just mentioned. You have got to start somewhere with a simple slab and then you can progress on to unusual restraint conditions, etc. It is no good just jumping in at the deep end.

D.N. Trikha. I think we have the method itself, the finite element method. It can be used for all the types of problem which the gentleman has just mentioned: slabs with holes or without holes, free boundary conditions, clamped boundary conditions. Anything you like, and we have the solution.

Anthony R. Cusens. I only want to echo Dr. Clark's words in that what I was trying to say was that we have tackled a number of simple problems, now let us get on to some of the more complicated ones. However, as Dr. Clark rightly says you have to start with the easy ones first; if they work out correctly then you move on to something else. As I understand it we are in the process of moving on.

Mikael W. Braestrup. When dealing with punching shear with most codes of practice one has to consider certain critical sections around the column, with or without rounded corners and at a certain distance from the column, which is usually some number times the half depth of the slab, this number being chosen from among the small integer numbers. This so called shear stress is then compared with some strength measure of the concrete, usually some measure of the tensile concrete strength. My question to the authors who presented papers on punching shear is what in your opinion is the relationship between this method and the actual phenomenon of punching?

Adrian E. Long. Since I have not had a chance to get a word in edgeways perhaps I could try to answer that question. I think this is a very good question in that there is a tremendous amount of debate as to where the critical section should be. If you go back to Moe's original paper in 1961, he considers the critical section to be in the immediate vicinity of the column and I would be inclined to agree with him that, that location is, as far as the real failure is concerned, the critical section. In our work we have been considering the critical section to be at 1.5h to tie in with the British code or 0.5d to tie in with the American code. However, I would agree that the critical section is almost at the periphery of the column, or slightly outside it.

Hans Gesund. I might add that I doubt that it makes any difference where you take your critical section as long as you adjust your allowable ultimate strength accordingly. If you take a lower allowable strength further out or a higher allowable strength closer in, you will get the same result. However, I do not believe in shear punching in normal slabs at all; I believe most of them are really local flexural failures, or at least failures triggered by local yielding of the reinforcement.

Brian E. Clark. Mr. Cleland discussed two aspects of importance in the design of post-stressed slabs: the shear resistance of the slabs and the shear to which the slab is subjected. I wonder if I might comment on the second of these questions.

Consider the two-bay slab shown in Figure 1. The principal bending moments and shear forces, calculated using the sub-frame procedure of C.P.110 and the equivalent frame method of A.C.I.318:77, are shown for the ultimate limit state. C.P.110 requires that the central column reaction be increased by an enhancement factor equal to (1 + 12.5 M/VL) which, deducting the tendon component, gives a net shear force to be resisted by the slab of 690.7 kN. A.C.I.318:77, using a different procedure, requires the slab to be designed for a shear force equal to 586.8 kN. Should these two values be different in view of the different shear perimeters in the two codes?

Figure 2 shows comparable values for the same slab assuming it to have many equal spans. Clearly the A.C.I. code would give no enhancement factor since the moment transferred to the column is zero. C.P.110, on the other hand, specifies a minimum enhancement factor of 1.25 so that the shear for which the slab must be designed will differ by 25 to 30 per cent.

I would welcome the comments of Mr. Cleland on these points.

C.P. 110:1972

4750 | 6950

$1.4G_k+1.6Q_k=96.6$ kN/m

	ext. cols. 300x300 @3.5m cr.		int. cols. 500x500 @ 7.0m cr. 200mm slab	
Secondary Moments	+7.0	-14.0	+63.0	-50.4
(1.4 DL + 1.6 LL)	-67.2	+245.5	-438.2	+240.8
TOTAL BENDING MOMENT, kNm	-60.2	+231.5	-375.2	+190.4
SHEAR, kN (due to 1.4 DL + 1.6 LL)	175.7	255.5	364.0	308.0

619.5

enhancement factor = 1 + ([12.5 x 144.2]/[619.5 x 6.95]) = 1.42

COLUMN REACTION V = 619.5 kN
1.42 V = 879.7 kN
less tendon component = -188.6 kN

691.1 kN

A.C.I. 318:77

$1.4G_k+1.7Q_k=98.6$ kN/m

Secondary Moments	-0.7	-34.7	+56.0	-44.1
(1.4 DL + 1.7 LL)	-35.7	+332.5	-420.7	+214.9
TOTAL BENDING MOMENT, kNm	-36.4	+298.2	-364.7	+170.8
SHEAR, kN	153.3	286.3	371.7	312.9

658.0

enhancement factor equivalent to $\alpha M_t c_3/J = 1.25$

COLUMN REACTION V = 658.0 kN
less tendon component = -188.6 kN

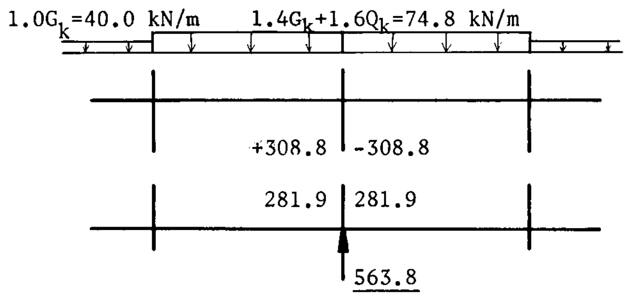
469.4 kN
x1.25 = 586.8 kN

Figure 1 Two Bay Slab

C.P. 110:1972

Bending Moment, kNm

Shear, kN



minimum enhancement factor = 1.25

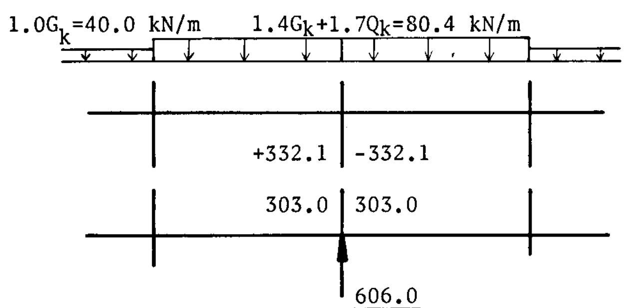
COLUMN REACTION V = 563.8 kN
1.25 V = 704.7 kN
less tendon component =-188.6 kN

516.1 kN

A.C.I. 318:77

Bending Moment, kNm

Shear, kN



equivalent enhancement factor is zero

COLUMN REACTION V = 606.0 kN
less tendon component =-188.6 kN

417.4 kN

Figure 2 Slab with Several Equal Spans

David J. Cleland. The figures which Mr. Clark has presented highlight the difference between the British and American codes in the analysis of connections subject to moment transfer. If the two methods are theoretically correct one would expect that the net shear force should be equal in each case. However there are at least two reasons why these would differ due to varying assumptions in the approaches:

1. The level of moment assumed to be transferred to the column is greater in C.P.110 (1972) since the frame analysis assumes full fixity at the slab/column connection.
2. In C.P.110 (1972) all of the out-of-balance moment is assumed to cause out-of-balance shear stresses while in A.C.I. 318:77 only about 40 per cent of the moment is assumed to be transferred by uneven shear.

For the equal span case the inclusion of a minimum magnification factor of 1.25 (in conjunction with the uniform loading case) in the C.P.110 1975 amendment appears not wholly logical, though rather conservative. However there is a limit to the value of comparing notional shear forces, as Mr. Clark has done, since different shear perimeters and different ultimate allowable shear stresses are also important factors and the overall interaction of these factors tends to confuse the issue.

As a consequence, comparisons of the ultimate predicted loads have been considered and from these it has been shown in Table 3 of our paper that in cases of interior columns subject to moment transfer the C.P.110 method is more conservative than the A.C.I. approach. The important fact which Table 3 attempts to highlight is that for edge columns the reverse is the case - primarily due to the use of the rather arbitrarily chosen 1.25 magnification factor in C.P.110.

Walter Thorpe. There is perhaps one other person here who has been on the Concrete Society Working Party producing the Flat Slab Design Recommendations but, in case he is not in the hall, I shall stand up and take the responsibility myself. I have also had some involvement with the F.I.P. Working Party preparing flat slab design recommendations.

I think that the problem which one always faces in such work is to prepare a document which will be acceptable to the authorities in the particular territory and which also results in safe and economic structures. In many cases the only way that this can be done is to work through examples completely using a design method which is currently in use in some territory, to work through the same examples completely by another proposed design method and, by comparison, to make sure that the results obtained are safe.

I certainly would not disagree in principle with the modifications which are proposed but there are aspects which I would like to discuss; mainly perhaps because design methods easily become very complicated. As I said earlier, the fundamental rule is that the design recommendations must be safe and they must be acceptable to the authorities. Thus the question of 0.5 or 1.5 to the critical perimeter becomes a leading question. I appreciate many of the advantages of adopting 0.5 but, with C.P.110 with us, one cannot very well advocate it.

Sami A.R. Ali. I have a comment for Professor Long about the critical section for punching shear. He said that it is very close to the face of the column, but actually we did 13 slabs in punching at Sheffield University and we found the critical section for punching shear occurred at between 1h and 1.5h. In none of the thirteen slabs did the critical section occur at the face of the column.

Adrian E. Long. I have tested many more than 13 slabs and I would agree with you entirely that the shear perimeter, as it actually projects up on to the top surface, is at a distance of 1.5d from the column face. However as far as I am concerned the mechanism which is initiating failure takes place at the junction of the slab and the column, right beside and fairly close to that junction, perhaps d/4 would be the maximum distance out, but very close to the column face.

Hans Gesund. Might I add just one quick note on that. I think it depends on the kind of shear failure you are talking about. Actually there is no such thing as a shear failure, it is a principal tension stress failure or principal compression stress failure; concrete does not fail in shear, it fails in principal tension or principal compression. A principal tensile stress failure may occur in the slab due to shear outside the immediate periphery of the column or it may occur due to combined shear and flexure at the face of the column. If it is a principal compression stress failure then it will almost certainly occur at the face of the column. Thus I think perhaps we should not be talking about shear failure, we should be talking about principal tension or principal compression stress failures.

Session 3

Structural Design

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STRUCTURAL DESIGN

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INTRODUCTION

The papers of this session are divided broadly into four topics: (a) flat slab design, (b) slabs with shear walls, (c) optimisation and cost implications in slab design, and (d) ground slabs.

Flat Slab Design

The first two papers relate to the Hillerborg method of design of slabs and have been mentioned by the keynote speaker of Session 2. The paper by Gardner and Faltaous on design of multiple panel flat slab structures compares experimental results with methods given in C.P. 110 (1) and A.C.I. 318:77 (2). A number of discrepancies are pointed out and useful recommendations given.

Goldstein has presented a critical review of two existing A.C.I. (2) methods of design for flat slabs in light of a new comprehensive method. The equivalent frame method is examined and the expression for the stiffness of the torsion member is criticised. A new concept of analysis is described based on fixed support moments. This combines with a dual member (beam/slab) concept which is extended to a dual frame concept. The results of using the dual member compare well with three dimensional elastic analysis. The moment distribution is used to replace the concept of equivalent width which results in more comprehensive design for lateral forces.

Slabs with Shear Walls

The paper by Atimtay deals with the design of concrete slabs for infinite in-plane stiffness. A method is derived for predicting when floor flexibility should be taken into account in the analysis for lateral loading on multi-storey buildings. It is suggested that for structures over ten storeys high the floors can usually be assumed to have infinite in-plane stiffness. The author also gives a method for determining the load in external and internal walls, for cases when the floor flexibility must be taken into account.

Gulvanessian and Sims' paper on recent developments in apportioning reinforcement in concrete slabs subjected to bending and membrane forces is really on its own and does not fit under the headings I have given. Nevertheless it fills an important gap in the designers tools by providing simple methods for determining the reinforcement parameters to resist membrane forces. It also provides a method for determining reinforcement parameters to resist a combination of membrane forces and moments. These methods which can be mounted on desk top computers will, I am sure, be readily used.

The paper on structural behaviour of floor slabs in shear wall buildings by Wong and Coull provides a set of non-dimensional design curves for the evaluation of the effective width of floor slabs coupling with a wide range of shear wall shapes which are based on a finite element model.

Optimisation and Cost Implications in Slab Design

The paper by Loov and Khalil is on optimum design of reinforced concrete and prestressed concrete slabs. The authors have derived an expression for the optimum slab thickness both for reinforced and prestressed concrete slabs. The results indicate that some design constraints affect the cost more than others. Future research should be directed towards these.

In his paper on some economic implications in reinforced concrete slab design, Salinas gives a number of interesting statistics. One possibly unexpected conclusion given is that high strength reinforcement and low strength concrete give low cost results.

Ground Slabs

Jalil's paper describes the future French recommendations concerning concrete pavements design. The paper gives information about both reinforced and unreinforced concrete pavements, and it is concerned with a number of practical points which affect the behaviour of such slabs.

The paper by Holland and Walsh is on behaviour and design of residential slabs on the expansive clays of Melbourne. This paper reports on a detailed study of the effect of building integral slab and beam foundations on expansive soils in Melbourne. It has led to a practical design procedure for such slabs.

In their paper Wood, Burley and Harvey have dealt with the design of ground bearing slabs in warehouse construction. This paper describes the study of analysis and design of ground bearing warehouse slabs. Plate bending finite elements were used in a three-dimensional model. A layered continuum model for the soil was found to give a closer approximation to the observed values than the conventional beam on elastic foundation analogy. It was found that the width of aisle was the most important parameter controlling the settlement pattern and the maximum bending moments in the slab.

THE STATE OF THE ART

My own interest lies in development of the U.K. Code of Practice, C.P. 110 (1) towards a more sensible and useful document for engineers. For some time now an anomaly relating to the analysis and detail design of slabs has existed in C.P. 114 (3) and C.P. 110 (1). Simplified methods for determining the bending moments and shear forces are included in C.P. 114 and C.P. 110. These may be used provided the spans do not differ by more than 15 per cent and the live-dead load ratio does not exceed 1. The detailing rules for the simplified method in C.P. 110

curtail the top bars at 0.3 x span, which is in accordance with practice for many years.

Engineers have become accustomed to designing structures to these rules wherever possible and their use has generally resulted in safe, serviceable and economical structures. No reports of structural failures resulting from the use of these methods have to my knowledge come to light. However, if the same slabs were designed to the more rigorous rules of these Codes a very different set of reinforcement details would be required. The three bending moment diagrams in Figure 1 show a comparison of using: (a) the simplified method of C.P. 110, (b) C.P. 114 pattern loading, and (c) C.P. 110 pattern loading. The moment diagram for the simplified method was prepared in a manner which relates closest to the rules, i.e. the hogging moment parts are produced by live load on adjacent spans only and the sagging moment parts are produced by alternate spans loaded.

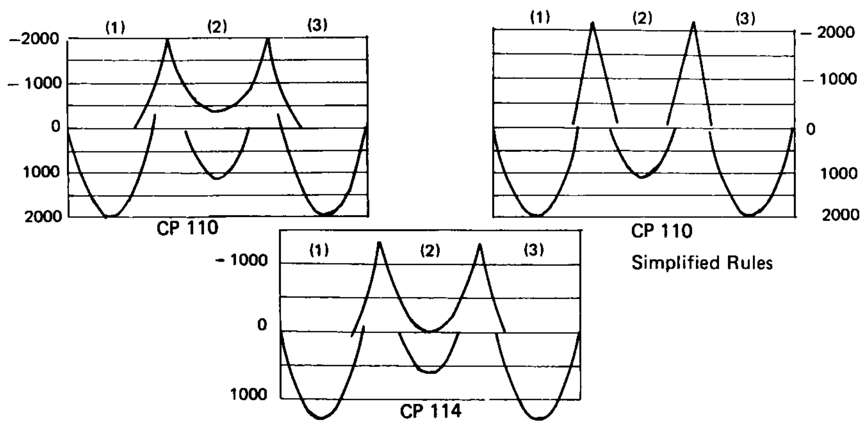

Figure 1 Bending Moment Diagrams

None of the simplified rules either for single-way, two-way or flat slabs take account of hogging in the unloaded spans of the load case with alternate spans loaded. Thus, in theory the simplified methods do not cater for a Type B failure as shown in Figure 2.

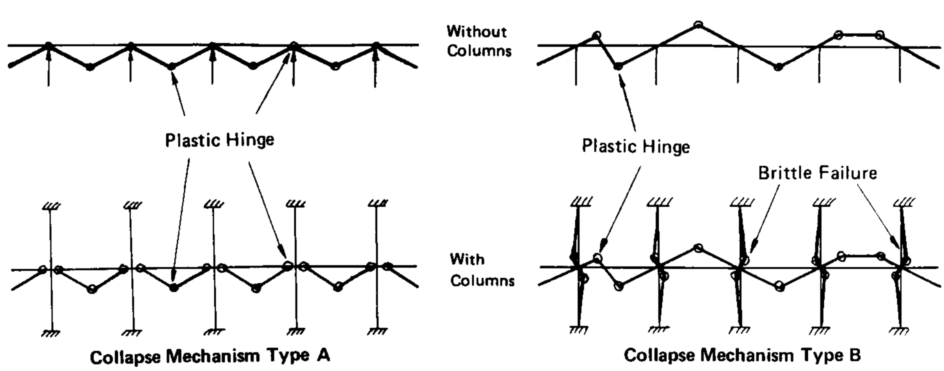

Figure 2 Failure Mechanisms

We are now faced with a dilemma. We could choose any of the three alternatives given below.

(a) Persuade engineers to scrap the present simplified methods and in so doing, by implication, pronounce as unsafe innumerable existing and apparently satisfactory structures.

(b) Alter the load factors to give bending moment diagrams with a closer fit to the simplified rules. Both the A.C.I. Code (2) and the CEB Model Code (4) treat the factored dead loads as fixed loads whereas C.P. 110 treats 40 per cent of dead load as a variable load. Unfortunately even changing C.P. 110 in this respect would not help the situation enough.

(c) Accept the simplified methods and acknowledge a number of factors which are difficult to calculate but which are often of great significance. These include: (i) catenary and other membrane actions, (ii) arching effects, (iii) restraints due to the prestressing effects from the surrounding structure, (iv) tensile strength of concrete, (v) increased safety factor in the strength of a large number of reinforcing bars over a small number, (vi) dispersing effect of secondary reinforcement, (vii) design loads not being achieved, and (vii) live loads not varying from span to span as much as the codes assume.

I would be inclined to choose (c) and alter the load combinations of the more rigorous method of analysis to fit. However, this will require careful stipulation of the limitations of this approach outside which the more rigorous pattern loading combinations should be used.

Over the last few years there has been a strong upsurge in the use of computers, especially desk top computers, for reinforced concrete analysis and detail design. The use of computer programs has tended to highlight problems such as I have described above. I do not advocate Codes of Practice being written for computers, but, nevertheless, I do believe it is important to establish simple methods which relate more closely with the rigorous methods.

The equivalent frame analysis for flat slabs has become popular because it is simple to use and can be readily programmed on small computers. However, it not only suffers from a misfit with the simplified empirical method but also gives rise to added complications in the analysis for shear. The first issue of C.P. 110 in 1972 (1) gave a very conservative estimate of the shear resistance of flat slabs at internal columns. A revision in 1976 altered this to a more reasonable level, but is is still thought by experienced designers that a further reduction is necessary. It is interesting to note that recent research tends to support this.

There are other aspects in C.P. 110 relating to the design of flat slabs which are still quite unsatisfactory. These include:

(a) Inadequate information on the effects of different aspect ratios of slab panels.

(b) Insufficient information on the effect of column size and shape on shear, especially with respect to moment transfer magnification factors.

(c) Information regarding shear perimeters and satisfactory methods of reinforcing for shear.

Both Professor Long in his recent research and Dr. Regan who has recently completed research for a CIRIA report have done much to answer some of these points. It is a pity that the results of Dr. Regan's work were not available for this conference. The CIRIA publication is overdue.

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REINFORCED CONCRETE SLAB DESIGN A GENERALISED STRIP METHOD INCLUDING CONCENTRATED LOADS AND SUPPORTS

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ABSTRACT A generalised strip deflection method of flexural slab design with distributed loading is presented. The load distributions are determined systematically to ensure satisfactory conditions under working loads. The method gives exact solutions for the plastic collapse load with simple banded layouts of reinforcement. Techniques are illustrated for extending the method to deal with concentrated loads and supports which only involve the use of simple load spreader systems.

INTRODUCTION

An objective examination of our present Code of Practice recommendations for the design of reinforced concrete slabs reveals little consistency of approach and a diversity in the amount of design information provided. For commonly occurring rectangular slabs under uniform loading a banded approach is employed with specified maximum moment coefficients. The origin of the method lies in elastic theory modified by tests, but now the moments are determined by yield line theory although the historical background is retained by placing all the active reinforcement in the centre bands.

For other slabs the Codes recommend the use of yield line theory or the Hillerborg strip method. Yet yield line theory, although a most reliable method for predicting the collapse load of a specified slab, leaves much to be desired as a design method. It fails to define the moment field throughout the slab and the difficulties of producing a safe solution grow as the curtailment of the reinforcement increases for economy. Hillerborg's simple strip method is an ingenious and attractive design method but it cannot at present deal with concentrated loads or supports and for more complex slabs the designer has no guidance on choosing the load distributions which will ensure satisfactory behaviour under working loads. For flat slabs the Codes resort to empiricism or approximate elastic analysis and until recently C.P. 114 even recommended moments which were not in equilibrium with the loads.

Of these various approaches only the strip method gives full information on the moment field throughout the slab and the loads transmitted to supporting beams. For rectangular slabs under uniform loading, the familiar triangular trapezium load distribution to supporting beams is recommended but this is only approximately correct for uniformly reinforced slabs at ultimate load. In general, composite action between slabs and supporting beams is inadequately considered in our present Codes.

In current Code recommendations we therefore have a variety of design methods which arise more from history than from logic and none of the methods can be considered ideal for design. The nearest to the ideal is the Hillerborg strip method but this is currently restricted in its application and gives insufficient attention to service conditions. The aim of this paper is to show how the strip method can be generalised to overcome the present limitations and become a basic design approach for all slab problems.

NOTATION

ij	grid element, row i, column j
M_s	support moment for column strip
M_{xy}	twisting moment per unit length
q	intensity of distributed loading
q_x, q_y	intensity of distributed loading in x and y directions respectively
R	column reaction
W	total load
Δx_{ij}, Δy_{ij}	flexural deflections of x and y strips at centre of grid element ij

HILLERBORG SIMPLE STRIP METHOD

The simplicity and power of the simple strip method first proposed by Hillerborg (1) is due to the elimination of the twisting moments, M_{xy}, and the consequent facility of choosing the distribution of the intensity of loading, q, into two orthogonal directions so that $q_x = \alpha q$ and $q_y = (1 - \alpha)q$. The plate continuum problem is thereby reduced to the simple analysis of beam strips.

When Hillerborg first proposed the method, his aim was to produce a safe lower bound solution. A critical examination of the strip method by Wood and Armer (2) lead them to conclude that if the reinforcement is precisely in accordance with the slab strip moments then'Hillerborg's method produces an exact solution with an unlimited number of simultaneous modes'. However Kemp and Fernando (3) have demonstrated that this conclusion is not generally true but is restricted to the cases where the two principal moments are everywhere of the same sign. They show that although it is not possible to give a general proof of uniqueness, the cases where the strip method does not lead to at least one unique solution mechanism are exceedingly rare.

The simple strip method therefore becomes very close to being an ideal method of slab design. It produces full information on the moment field using only simple computations, usually provides an exact solution for the plastic collapse load and with a suitable choice of the strips gives economical reinforcement in simple banded layouts. Its only serious disadvantages are that the designer may choose load distributions which depart too far from working load conditions leading to unserviceability due to early cracking and particularly the difficulty of dealing simply with concentrated loads and supports. Hillerborg has devoted considerable effort and ingenuity to solve the latter problem but his solutions involve complex moment fields, including twisting moments, and the simplicity of the strip method is lost.

The authors have recently proposed a generalised approach (4) to the strip method aimed at overcoming these disadvantages and producing a systematic treatment which maintains the attractiveness of the original concept. The basis of this new approach which has been called the strip deflection method will first be described and it will then be shown how it can be extended to accommodate concentrated loads and supports.

THE STRIP DEFLECTION METHOD

A simple example will be used to illustrate the basic approach of the strip deflection method. Figure 1 shows a rectangular slab simply supported on the two short sides, fixed on one long side and free on the other and subjected to a distributed loading. The slab is divided into 16 rectangular grid elements by choosing four strips in each orthogonal direction.

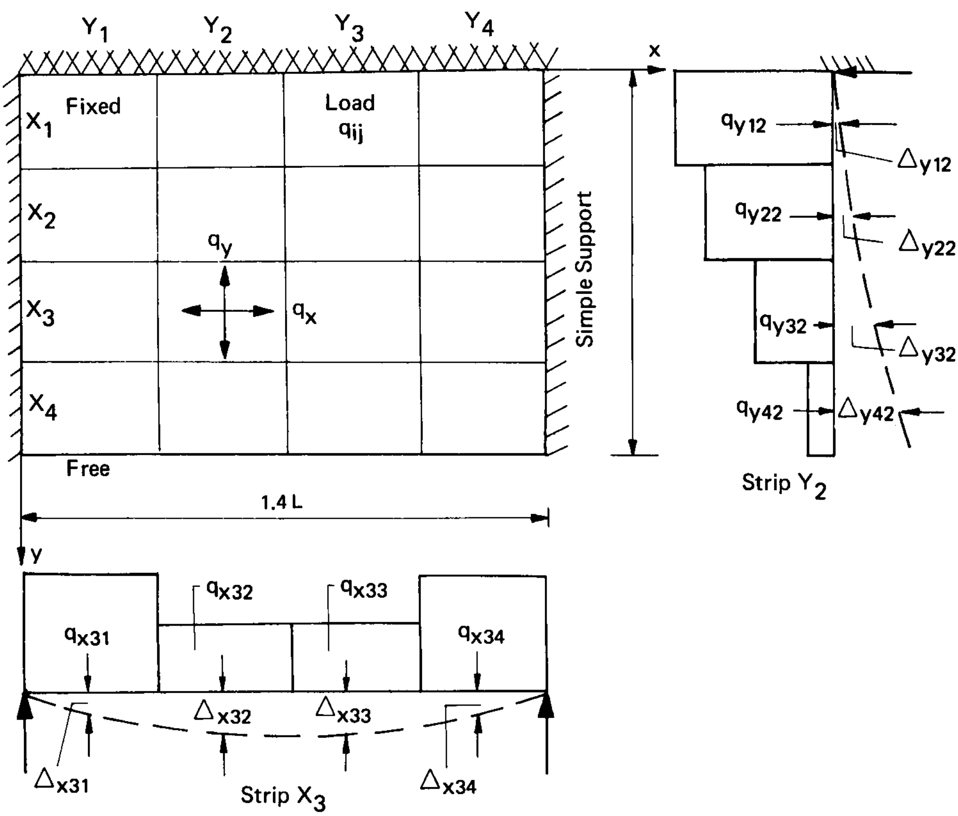

Figure 1 Example of Strip Deflection Method

The key assumptions are that the twisting moments, M_{xy}, are zero everywhere and the intensity of loading, q, is constant over each grid element, although it may vary from element to element. For any grid element, ij, the equilibrium required is $(q_x)_{ij} + (q_y)_{ij} = q_{ij}$ where $(q_x)_{ij}$ and $(q_y)_{ij}$ are the unknown load distributions in the x and y directions. Due to the one axis of symmetry in this example, there will be only eight independent load distributions $(q_x)_{ij}$ to be found.

In Hillerborg's strip method these values of $(q_x)_{ij}$ are selected by the designer intuitively and although they will generally lead to a unique value of the plastic collapse load, they do not necessarily provide serviceability for cracking. In the strip deflection method the values of $(q_x)_{ij}$ are determined systematically so that they will not depart far from working load (elastic) conditions. This is achieved by requiring compatibility of the elastic flexural deflections of the orthogonal strips at the intersection of their centre lines when loaded by the unknown patch loads $(q_x)_{ij}$ and $(q_y)_{ij} = q_{ij} - (q_x)_{ij}$. There will be one such compatibility equation $\Delta x_{ij} = \Delta y_{ij}$ for each grid element, exactly equal to the unknown load distributions $(q_x)_{ij}$.

The compatibility equation can be readily formulated using simple elastic beam theory and the solution of the eight simultaneous linear equations can be performed on a small computer. The load distributions obtained for the simple slab in Figure 1 when subjected to a uniformly distributed load q = 1.0 are shown in Figure 2. The wide variation in the values of $(q_x)_{ij}$, including one negative value, will be observed and such values would be very difficult to predict intuitively even by an experienced designer.

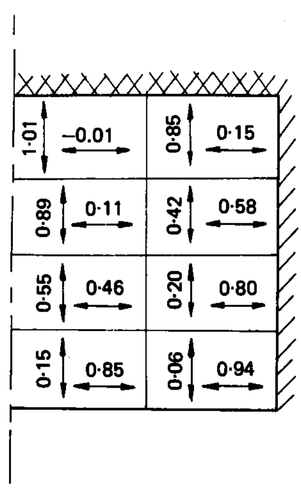

Figure 2 Load Distribution, q = 1.0

It must be emphasized that elastic deflections are only being used to determine serviceable values of the load distributions. The strip deflection method remains a plastic strip design method, closely similar to Hillerborg's approach, and no great precision is required in the calculated values of $(q_x)_{ij}$. In essence the method is a generalisation of the Rankine-Grashof method and is also closely related to the torsionless grid analogy method of plate analysis. Indeed the authors would recommend the use of computer programmes based on torsion free grids for the analysis of the load distributions wherever these are available.

Once the load distributions are known, the bending moments in the various strips can be readily determined by simple statics. For an exact plastic solution the loads must be uniformly distributed over the grid areas which is the only important difference between the strip deflection method and the torsionless grid analogy

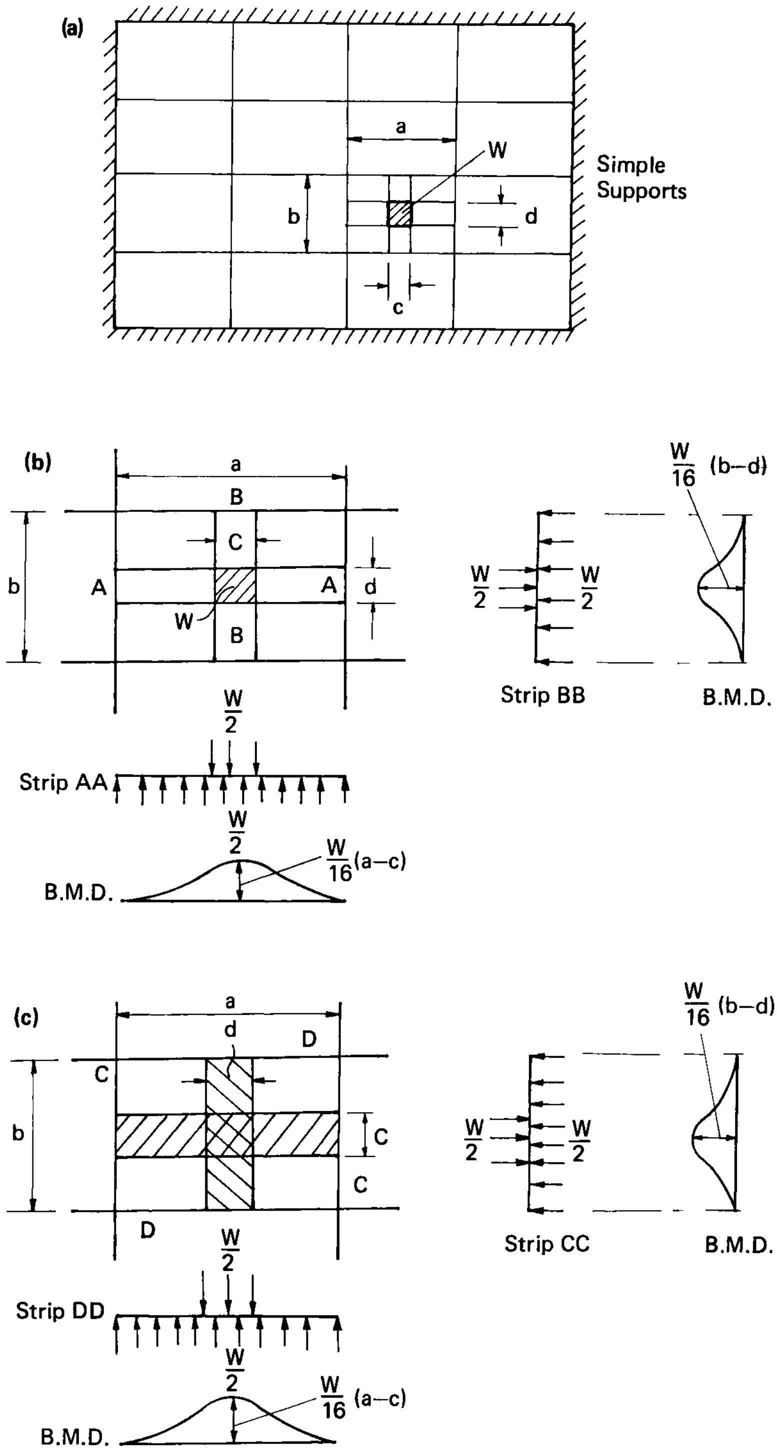

Figure 3 Spreader System for Concentrated Load

which assumes point interaction between the intersecting grids. In practice the differences between the moments due to assuming point interactions rather than distributed load interactions over grid elements are quite small with four or more strips in each direction.

A fuller description of the strip deflection method with distributed loading is given in Reference (4). The procedure with other boundary support conditions is described there and it is shown that partial composite action with supporting beams can be readily included in determining the load distributions since the supporting beams just become narrow edge strips. The strip deflection method will therefore provide full information on the moments throughout the slab and any supporting beams leading to an exact solution for the plastic collapse load. The reinforcement required is in a simple banded layout and will approach minimum weight design as the number of strips increases. The use of elastic deflections in determining the load distributions should avoid unsatisfactory behaviour under working loads. The whole procedure can be computerised if desired once the designer has chosen a strip layout. It now remains to be shown how the method can be extended to deal with concentrated loads and supports.

PATCH AND CONCENTRATED LOADS

If a patch load covers a sufficiently extensive area of the slab, the strip system can be chosen so that the patch load is contained entirely within one grid area. In these circumstances the procedure for analysis is identical to the one described for uniformly distributed loading.

If, however, the concentrated loaded area is small, a strip system should be chosen so that the concentrated load is centrally positioned within a grid rectangle as shown in Figure 3(a). The concentrated load is then assumed initially to be uniformly distributed over the whole grid area and the analysis for load distributions and bending moments would proceed exactly as for distributed loading.

The bending moments so derived do not, however, satisfy equilibrium in the local region of the grid area containing the concentrated load due to the initial assumption of spreading the load over the whole grid area. To obtain an exact solution for the plastic collapse load, additional moments must be added within the grid element containing the concentrated load. These additional moments depend only on the geometry of the concentrated loaded area and the grid rectangle and the magnitude of the load. They can be readily calculated in a general form by using a simple equilibrium spreader system.

Such a system is shown in Figures 3(b) and 3(c). The concentrated load is first uniformly distributed equally to the two strips AA and BB in Figure 3(b), giving the bending moments shown within these two strips. The load from the two strips AA and BB is then distributed uniformly to the whole grid area which produces the bending moments shown in Figure 3(c) within the grid area. By the use of this simple spreader system, equilibrium is satisfied within the grid element containing the concentrated load and if additional reinforcement is provided in accordance with the moments in both of the two spreader systems, the design moment field will lead to an exact solution for the plastic collapse of the slab.

INTERNAL CONCENTRATED SUPPORTS

Unique plastic solutions for the collapse load can also be obtained by the strip deflection method for slabs with concentrated internal supports by the use of simple spreader systems. Consider the square slab shown in Figure 4 carrying a

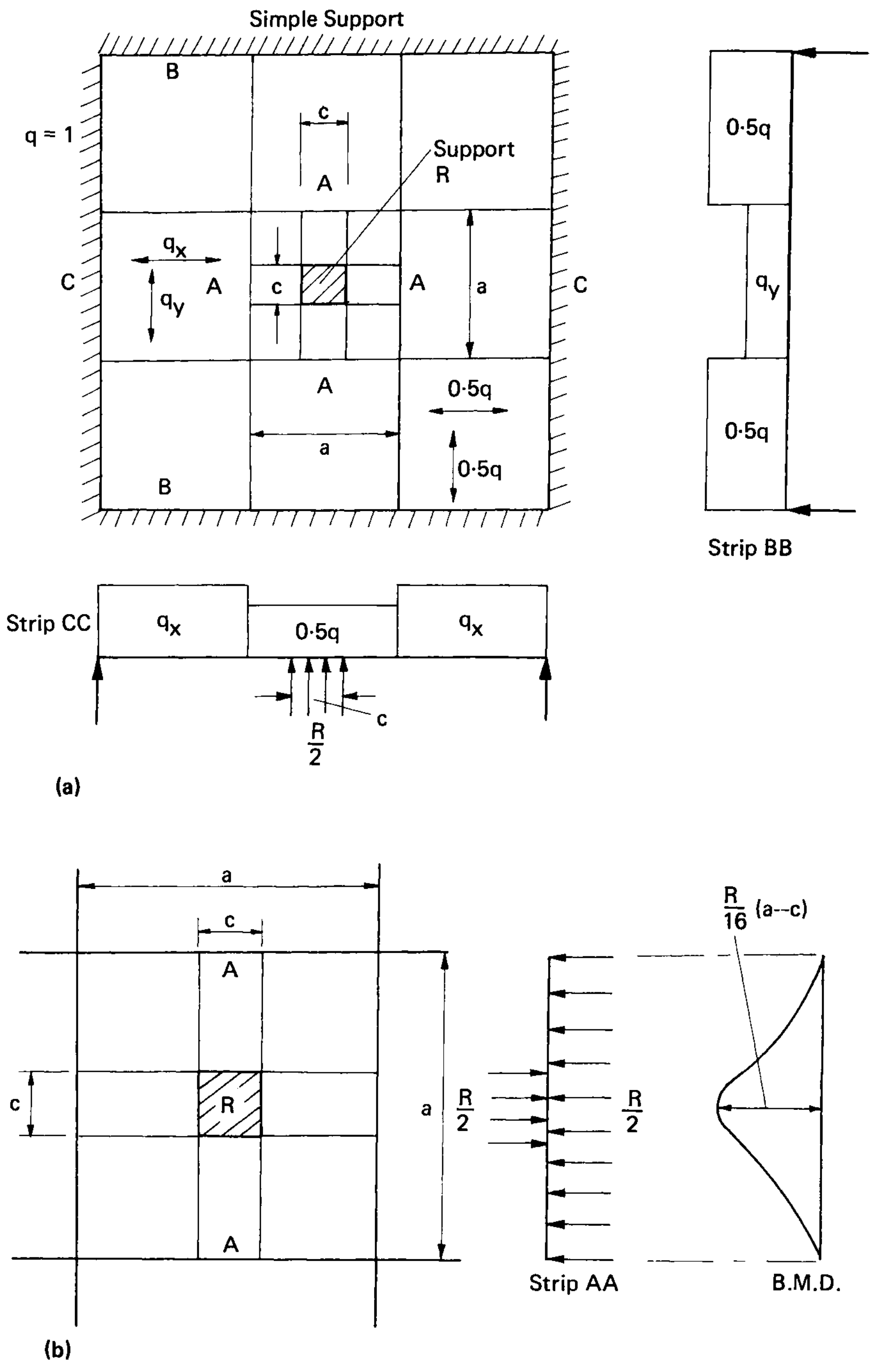

Figure 4 Spreader System for Internal Support

uniformly distributed load q = 1.0, simply supported at the boundaries and with a central concentrated support. For the strip system shown, there is only one unknown load distribution to be found, q_x, since due to symmetry all the others are $q_x = q_y = 0.5$. This load distribution can be found by equating the elastic flexural deflections at the intersection point of the centre lines of strips BB and CC and for this stage the reaction R at the internal support could be assumed concentrated at the centre point since only approximate load distributions are needed.

The bending moments in the strips BB and CC can then be determined by statics and the reactions R/2 at the centre support should not strictly be assumed to be uniformly distributed over the column width c as in Figure 4(a). These bending moments will satisfy equilibrium except within the grid area containing the internal support, where it has been assumed that the strip CC is supported across the full width of the strip a, whereas the column width is only c. To satisfy local equilibrium and ensure a unique solution, the column reaction R must be distributed uniformly over the two spreader strips AA as shown in Figure 4(b). If additional reinforcement is provided in strips AA in accordance with those moments in the spreader system, the total moment field will give a unique plastic collapse load for the slab.

A practical problem is the choice of the width of the column strip AA which will ensure satisfactory serviceable behaviour with regard to cracking. Tests on large scale slabs designed by the strip deflection method are needed to answer this question satisfactorily but until this information is available, the accumulated experience contained in codes of practice can be utilised. For flat slabs C.P. 110 (1972) recommends a column strip width of half the mean span or the drop width and this rule could be employed in the use of the strip deflection method until more experimental evidence is available on serviceability and column strip width.

CORNER SUPPORTS

The treatment of corner supports is illustrated in Figure 5. This shows a square slab with four square column supports of plan dimension c carrying a patch load W in the centre. The strip system chosen has column strips BB of width b and centre strips AA of width (L - 2b) and because of symmetry the load distribution throughout the slab is determined entirely by statics. The loading and bending moments for the centre and column strips are shown in Figure 5(a). It should be noted that the column strip BB is assumed to be supported over the full width of the strip on a support of length c. To satisfy equilibrium it is necessary to introduce spreader strips of width c which cantilever from the columns for the full width of the column strips b. The loading and moments in these spreader strips are shown in Figure 5(b) and the moment transmitted to the column is W(b-c)/16. Field line analysis will show that the solution is exact which is dependent on the introduction of the spreader system.

In this example the support moments for the column strips were assumed to be zero, so there is no torsional moment in the column spreader strips when analysing a diagonal mode collapse mechanism. If, however, it is wished to transmit support moments to the columns this can be achieved. The support moments could be determined by elastic frame analysis of the column strips and columns, or by using the results of current research programmes on the moments actually transmitted by flat slabs to supporting columns. These negative support moments would then be added to the bending moment diagram for the column strips BB, reducing the positive moments in the strip by the same amount. The support moments assumed to act uniformly across the full width of the column strip b will cause torsional moments in the column spreader strips. If the total support moment is M_s, then

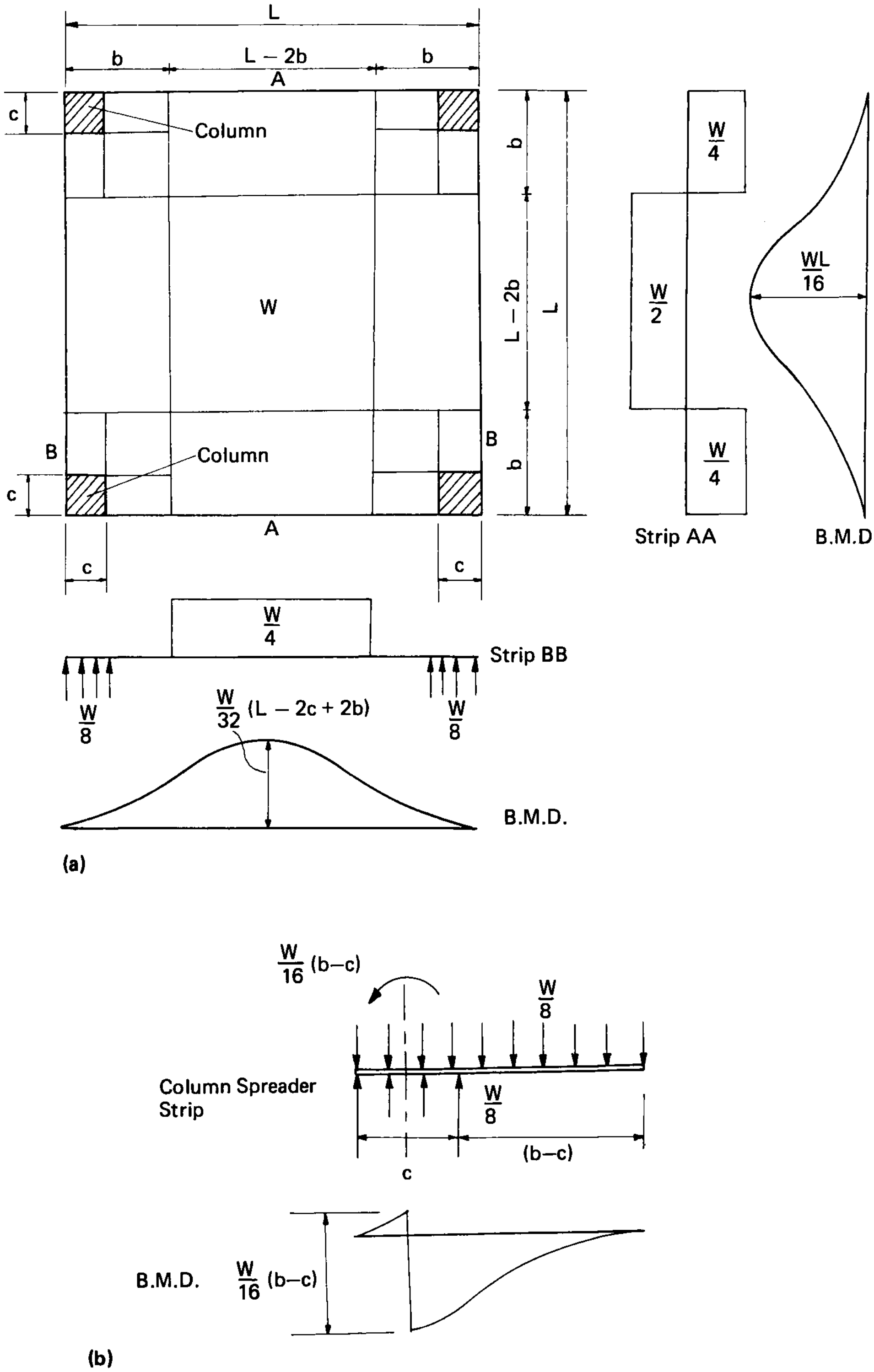

Figure 5 Spreader System for Corner Support

$M_s c/b$ will be transmitted directly to the column and the column spreader strips will need to be reinforced for torsional moments which increase linearly from zero at the end to M_s (b-c)/b at the junction with the column. If such reinforcement is provided the moment field will again lead to an exact solution for the collapse load.

CONCLUSIONS

A generalised strip method of slab design has been proposed. For uniform or patch loading the designer chooses an orthogonal strip system to give uniform loading over each grid rectangle. The load distributions in each grid rectangle are then determined systematically by insisting on compatibility of the elastic deflections at each of the intersection points of the centre lines of the orthogonal strips. A torsion free grid analogy programme would be a convenient procedure for this analysis.

For an exact solution for the plastic collapse load, the moments in the strips must be calculated assuming the load distributions to be uniformly distributed over grid lengths but with more than four strips in each direction, a grid analogy programme would give close approximations to the moments. The same procedure has been shown to be applicable with concentrated loads and supports but it is then necessary to introduce simple spreader systems. These involve additional moments in the grid element containing the load or support but they can be readily calculated by simple statics.

The strip deflection method thus becomes applicable to any form of slab and loading, and gives an exact solution for the plastic collapse load. The load distributions will not depart far from working load conditions so that satisfactory serviceability against cracking should be ensured. The reinforcement required is in a simple banded layout and economical in amount. The method would appear to be an ideal method for the design of all types of slab and could be the basis of consistent recommendations in Codes of Practice. Further tests are needed on large scale slabs to determine the optimum width of strips with concentrated loads and supports to ensure satisfactory serviceability, but until this is available current Code of Practice recommendations could be used.

ACKNOWLEDGEMENTS The authors wish to acknowledge the Science Research Council for the financial support provided.

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HILLERBORG'S SIMPLE STRIP METHOD OF DESIGN FOR REINFORCED CONCRETE SLABS

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ABSTRACT. A simple, practical and economical design method for two-way reinforced concrete slabs, based on Hillerborg's Strip Method, is presented. Tests on three model slabs incorporating minimum reinforcement ratio, minimum depth and maximum bar spacing permitted by the Australian Concrete Structures Code showed satisfactory behaviour at service loads and ultimate loads despite the fact that bending moment fields at ultimate load (and, therefore, reinforcement layouts) were different to elastic distributions. A saving of approximately 40 per cent of reinforcement, compared with conventional code requirements, was achieved.

INTRODUCTION

In what Hillerborg called his 'Strip Method' he attempted to formulate a practical and simple method of design for two-way spanning reinforced concrete slabs, based on the lower bound theorem of plasticity. The twisting moments in the slab equilibrium equation:

$$\frac{\partial^2 M_x}{\partial x^2} - 2\frac{\partial^2 M_{xy}}{\partial x \partial y} + \frac{\partial^2 M_y}{\partial y^2} = -w$$

were given a value of zero so that the problem was reduced to the design of one-way-spanning orthogonal beam strips, see Figure 1, each carrying a proportion of the loading, and represented by the equations:

$$\frac{\partial^2 M_x}{\partial x^2} = -\alpha w \quad \text{and} \quad \frac{\partial^2 M_y}{\partial y^2} = -(1-\alpha)w$$

This method, as published in 1956 (1), applied to uniformly loaded, continuously supported slabs and became known as the 'Simple Strip Method' when Hillerborg later developed a method, the 'Advanced Strip Method', to deal with column supports (2).

The division of the load into αw and $(1-\alpha)w$ is arbitrary and therefore the elastic deflections of any two strips at their intersection points would generally be dissimilar. However, under ultimate load conditions it is assumed that plasticity enables deflections to become compatible. The strips are reinforced to carry exactly their assigned proportion of the ultimate load and, usually, the load is defined by an arbitrary load dispersion pattern dividing the slab into zones so that all the load in each zone is assumed to be carried either wholly in the x-direction or wholly in the y-direction (i.e. $\alpha = 0$). For slabs with fixed edges, lines of contraflexure (usually straight) are arbitrarily positioned to define the lengths of positive and negative bending moment zones in the strips. Typical load dispersion lines and lines of contraflexure are shown in Figure 2, together with the resulting loading on typical strips.

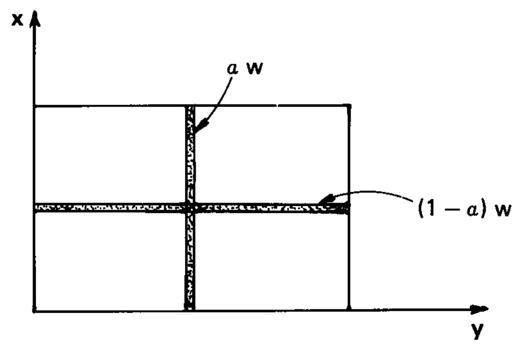

Figure 1 Loading on Strips

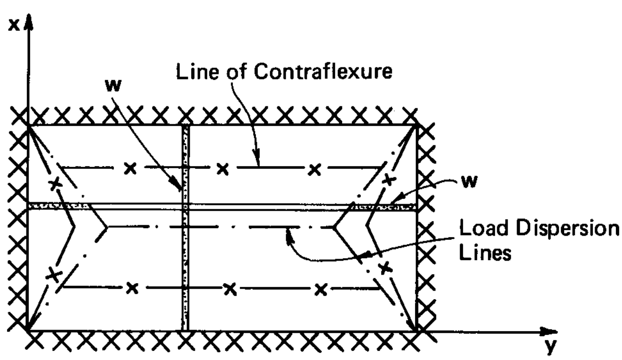

Figure 2 Typical Load Dispersion and Contraflexure Lines for a Slab with Fixed Edges

For a design to be regarded as satisfactory the behaviour of the slab must be acceptable at both ultimate load and service load. The arbitrary nature of the assignment of load (and thus of reinforcement) to the strips means that the behaviour at one or other load could be unsatisfactory; brittle behaviour could occur in some sections at ultimate load or excessive cracking or excessive deflection could occur at service load.

In the conventional code design of two-way slabs, based on bending moment coefficients, the calculated reinforcement ratios are frequently less than the minimum ratios demanded by the code for shrinkage control or other reasons. Considerable quantities of reinforcement thus have to be added both in the spanwise direction and in the secondary direction. The extent of such extra reinforcement is governed by the position of the lines of contraflexure determined from elastic analysis or from simplified code rules and the designer thus has no opportunity to optimize the reinforcement layout.

This situation may be criticised on the one hand because the additional spanwise reinforcement is not required from ultimate strength considerations and is irrelevant to shrinkage control since shrinkage cracks do not occur when flexural action exists (3). On the other hand, in the secondary direction the code requirements for shrinkage reinforcement are generally much too small to provide satisfactory control of direct tension cracking (3). With the Hillerborg method the designer can more efficiently utilize his reinforcement simply by a suitable choice of load dispersion lines and lines of contraflexure.

In this paper the behaviour of three one-third scale model slabs is reported. The slabs had an aspect ratio of 1.6 and were designed on the basis of full utilisation of the code minimum sagging moment reinforcements at mid-spans and, where possible, of support hogging moment reinforcement. Effective depth and maximum bar spacing were those permitted by the serviceability criteria of the Australian Concrete Structures Code (4). The behaviour of the slabs was satisfactory at both working load and ultimate load and it is suggested that the design basis used may be regarded as simple, economical and practical.

SLAB DESIGN BASIS

The three slabs considered all had both long edges fixed, whilst the short sides were i) both fixed, ii) one fixed and one pinned and iii) both pinned. The prototype size was 7.32m x 4.57m (24ft x 15ft) and a working live load of $4kN/m^2$ was assumed. The slabs were designed in the following way.

(i) Reinforcement for the short span sagging moment was determined by the minimum ratio (0.0015) permitted by the Code (4).
(ii) Effective depth was determined by deflection control as exercised by limiting the short span/effective depth ratio to the minimum value permitted by the Code.
(iii) Ultimate strength requirement was determined from factored live and dead loads.
(iv) Lines of contraflexure in the short span were positioned to equate the ultimate sagging moment and ultimate strength.
(v) Support hogging ultimate moments were calculated and reinforcement determined.
(vi) Reinforcement for the maximum long-span sagging moment was set at the Code minimum.
(vii) Loaded length in the sagging moment zone was chosen to equate ultimate sagging moment and ultimate strength.
(viii) Contraflexure point (if edge fixed) was positioned to equate support hogging moment to ultimate strength, with minimum reinforcement ratio.

Design Examples

Slab 1 : All Edges Restrained

<u>Short span</u> = 4.570m

Gross p = 0.0015
Steel yield strength, f_{sy} = 635N/mm^2
Span/effective depth = 26 x 1.22 = 31.7
∴ Effective depth = 4570/31.7 = 144mm

Bar diam. = 12mm
Slab depth = 144 + 20 + 6 = 170mm
∴ Required A_{st} = 0.0015 x 1000 x 170 = 255mm^2/m

Bar spacing = 3d = 3 x 144 = 432mm
∴ Actual A_{st} = 260mm^2/m

Self weight = 4kN/m^2
Live load = 4kN/m^2
∴ Ultimate load = 1.5 x 4 + 1.8 x 4 = w = 13.2kN/m^2

Effective strength of section = $M' = \phi[A_{st} f_{sy} d(1 - 0.6 \frac{A_{st}}{bd} \cdot \frac{f_{sy}}{F'_c})]$ = 20.7kNm

Length of simply supported span (see Figure 3) = ℓ_2,
where $w\ell_2^2/8$ = 20.7, i.e. ℓ_2 = 3.54m
∴ $R_B = R_C = w\ell_2/2$ = 23.36kN

$\ell_1 = \ell_3$ = (4.57 - 3.54)/2 = 0.515m

$M_A = M_D = w\ell_1^2/2 + R_B\, \ell_1$ = 13.78kNm

13.78 < 20.7 ∴ Use minimum p = 0.0015

<u>Long span</u> = 7.32m

Make sagging moment = 20.7kNm
Then ℓ_5 (= ℓ_7) is given by $w\ell_5^2/2$ = 20.7, i.e. $\ell_5 = \ell_7$ = 1.66m

Make support hogging moment M_E (= M_K) = 20.7kNm
Then ℓ_4 (= ℓ_8) is given by $w\ell_4^2/2 + R_F\, \ell_4$ = 20.7, i.e. $\ell_4 = \ell_8$ = 0.73m
∴ ℓ_6 = 7.32 - (2 x 1.77) - (2 x 0.73) = 2.32m

Slab 2 : 3 Edges Restrained, 1 Short Edge Pinned

Short span design as for Slab 1

Long span design differs only in that right hand edge is pinned and ∴ ℓ_8 = 0

Slab 3 : Long Edges Restrained, Short Edges Pinned

Short span design as for Slab 1

In long span design, $\ell_4 = \ell_8$ = 0

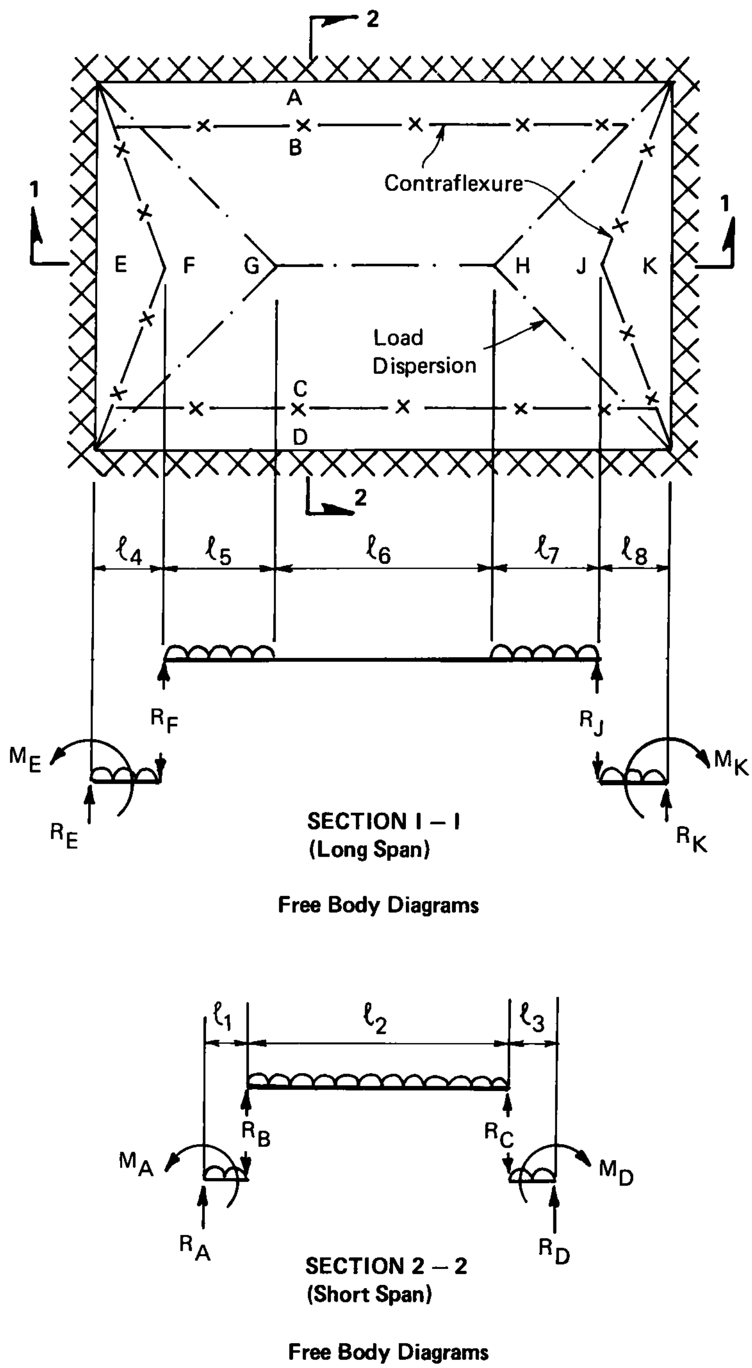

Figure 3 Loading on Typical Strips of Slab 1

Reinforcement Adopted

For the size of slab adopted, curtailment of bars was considered inappropriate and Figure 4 shows the simple reinforcement layout used for Slab 1. Reinforcement ratios were the same as would be required by the conventional code design but more efficient utilization resulted in a substantial saving in reinforcement quantity of approximately 40 per cent.

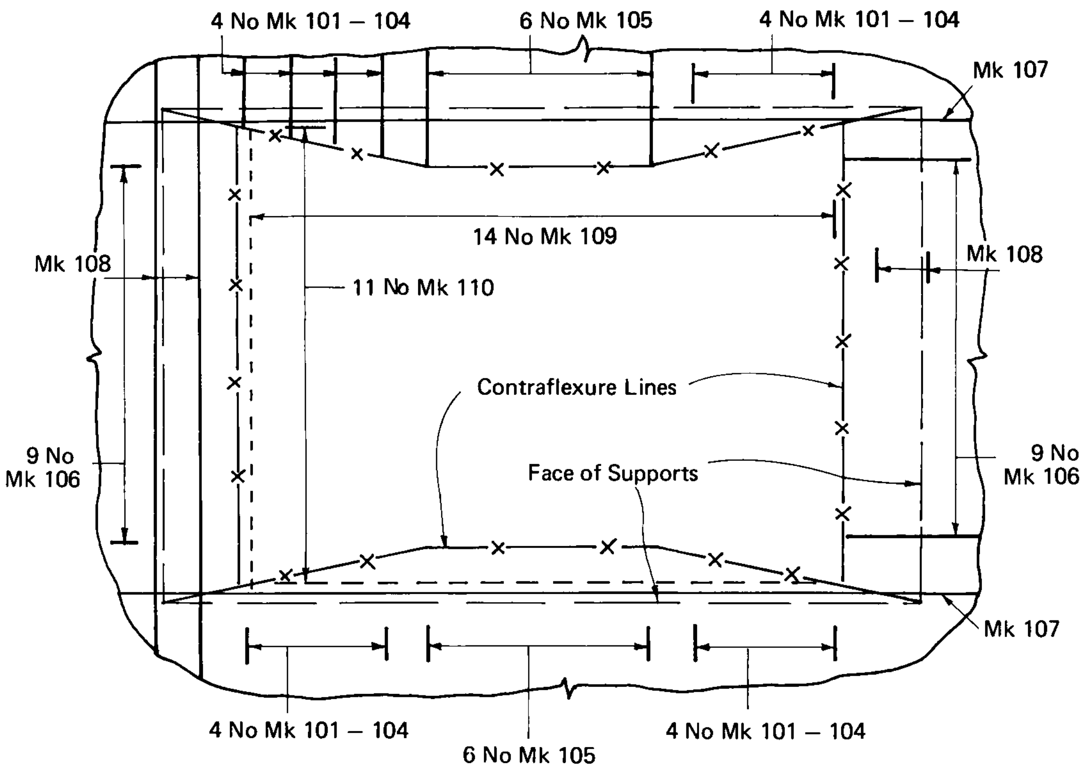

Figure 4 Reinforcement of Slab 1

DETAILS OF MODEL SLABS

Testing Arrangement

The size of the model slabs was dictated by the size of the water bags that were already available for application of the load to the slabs. The scale factor of three was chosen so that 12mm diameter reinforcement bars in the full size slabs could be directly replaced by 4mm wires in the models.

The slabs were tested upside down, with the water bags reacting against the laboratory strong floor and surrounded by a reinforced concrete plinth, as shown in Figure 5. The fixed edges of the slabs were clamped between the plinth and a steel channel which was prestressed to the floor by 25mm diameter bolts, see Figure 5(a). The slab's pinned edges reacted against the steel channel through steel rollers and were free to translate and to rotate slightly, see Figure 5(b).

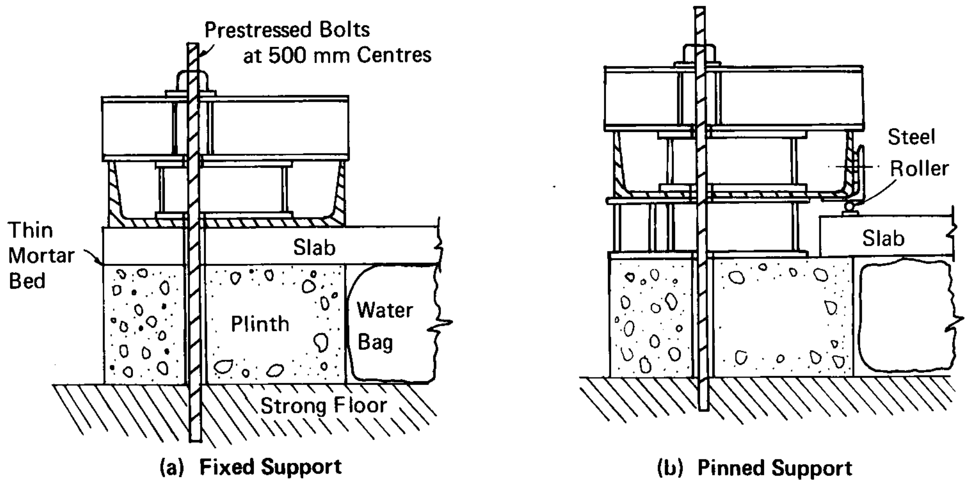

Figure 5 Details of Holding Down Assembly Units

Pressure was supplied to the water bags from the water main via a pressure reducing valve which could be set to maintain the required pressure. This was measured by a conventional pressure gauge and also by a transducer incorporating a digital voltmeter. A pump was used to extract water quickly when unloading.

The deflection of the slabs was measured by dial gauges and by inductance transducers at various points on both centre lines. Strains were measured on the 'soffit' of the slabs, with 100mm Demec gauges applied to lines of overlapping gauge lengths. All results were transferred to punched tape either via a data logger or by a manually operated teletype.

Materials

To simplify analysis, and interpretation, of test results, it was considered important that the model reinforcement should have a well-defined yield point and a long yield plateau. To obtain this, 4mm diameter hard drawn wire, which was available in straight lengths, was heat treated at 475°C. From ten random test samples it was found that the yield stress ranged from 620 to 645N/mm^2, with a mean of 635 N/mm^2. The concrete was made with ordinary Portland cement, 10mm maximum size crushed basalt coarse aggregate and coarse and fine natural sands. Tests carried out on standard cylinders gave the results shown in Table 1.

RESULTS

Service Load Behaviour

In the early stages of testing of the slabs, small increments of load were applied within the working load range; deflections and strains were measured and the onset of cracking studied.

Table 1 Concrete Test Results

SLAB NUMBER	W/C RATIO	A/C RATIO	SLUMP mm	TEST AGE Days	STRENGTH, N/mm^2 Compressive	STRENGTH, N/mm^2 Indirect Tensile	ELASTIC MODULUS kN/mm^2
1	0.61	5.30	180	24	29.0	3.2	25.5
2	0.57	4.78	130	31	26.3	2.7	22.4
3	0.60	4.78	140	14	24.9	2.0	21.1

At the dead load condition ($4kN/m^2$) none of the slabs was cracked on the soffit. With dead load plus service live load ($8kN/m^2$) slabs 2 and 3 were still uncracked but in Slab 1 minor cracking was observed which started at a load of $6.3kN/m^2$.

The maximum deflections of the three slabs at the service load were respectively span/1670, span/2470 and span/2670, see Table 2. These values are not strictly comparable because the loading rates and histories were not identical; they give only an indication of short term deflection. Long term deflection was not investigated.

Table 2 Approximate Maximum Deflections

SLAB NUMBER	DEFLECTIONS AT VARIOUS STAGES, mm				
	$4\ kN/m^2$	$8\ kN/m^2$	$13.2\ kN/m^2$	Yield Pattern Complete	Ultimate Load
1	0.35	0.92	1.4	10	25
2	0.23	0.62	1.4	15	22
3	0.29	0.57	1.8	14	18

From the strain measurements along the centrelines and along several lines of gauge lengths normal to the slab edges, it was shown that the position of the lines of contraflexure corresponded to elastic behaviour. For example, for Slab 1 the lines of contraflexure were approximately 0.22 x short span and 0.18 x long span from the faces of the supports.

Ultimate Load Behaviour

As loading was increased incrementally beyond the service load, cracking propagated in all the slabs, although even at loads in excess of the theoretical ultimate load ($13.2kN/m^2$), the crack patterns were not fully developed, see Figure 6. At the theoretical ultimate load the maximum deflections, see Table 2, were approximately span/1000 for Slabs 1 and 2 and span/850 for Slab 3 but, again, these values are not strictly comparable and only indicate the smallness of the deflections.

As the crack patterns became fully developed, the strain measurements showed that the points of contraflexure moved to approximately the positions predetermined in the design; this was particularly noticeable in the short spans where the movement was from a position at 0.22 x span to one at 0.11 x span. A progressive decrease occurred in the stiffness of the slabs, as illustrated in Figure 7 which shows the load-deflection curve for Slab 1. The other load-deflection curves were similar in form.

Eventually a full yield-line pattern developed in each slab, see Figure 6, and the failure loads were taken as the loads at which all yield lines just reached the slab boundaries. For the three slabs these loads were respectively 2.75, 1.90 and 1.87 times the design ultimate load value of 13.2kN/m^2.

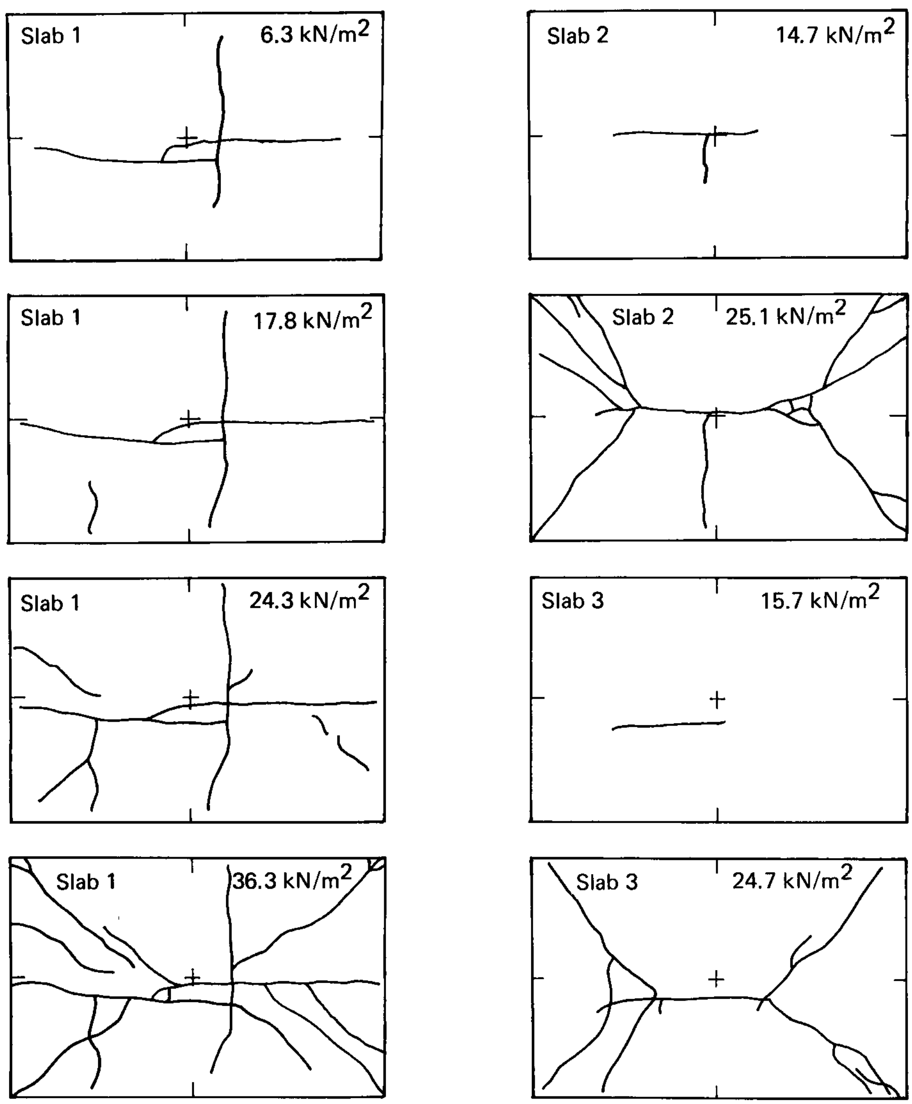

Figure 6 Crack Patterns on Slab Soffits at Various Loads

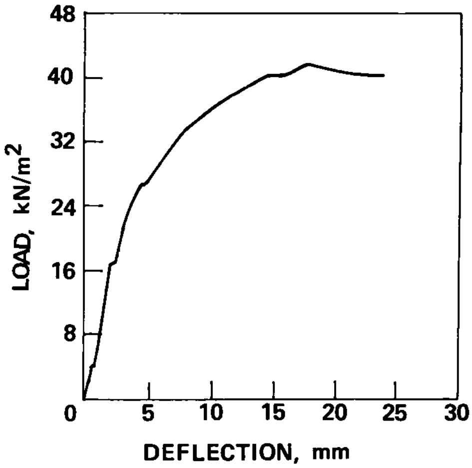

Figure 7 Central Deflection of Slab 1

Analysis of the slabs by Johansen's yield line method, ignoring corner fan effects, predicted ultimate loads somewhat larger than the values at which full patterns actually developed, particularly for Slabs 2 and 3, as shown in Table 3. A significant increase in load occurred with Slab 1 as the slab was deflected further and, eventually, tensile membrane action commenced with cracks penetrating right through the slab. Smaller increases in load occurred in Slabs 2 and 3, the pinned edges presumably reducing the membrane action.

Table 3 Comparison of Calculated and Actual Loads

SLAB NUMBER	LOADS, kN/m^2				RATIOS		
	Hillerborg Ultimate H	Johansen Ultimate J	Yield Lines Complete Y	Observed Ultimate U	$\frac{Y}{H}$	$\frac{Y}{J}$	$\frac{U}{J}$
1	13.2	37.0	36.3	42	2.75	0.98	1.14
2	13.2	32.6	25.1	26	1.90	0.77	0.80
3	13.2	29.4	24.7	27	1.87	0.84	0.92

DISCUSSION OF RESULTS

Hillerborg's strip method has been widely acknowledged to be a simple and powerful method for the design of slabs, although the designer is usually cautioned to choose ultimate moment fields that are not too different from elastic distributions (5). On the other hand numerous researchers (e.g. 6,7,8) have investigated optimized reinforcement layouts with unconventional and complex patterns based on sometimes complex design procedures.

This paper investigates in a very limited way, a simple optimization design process, resulting in ultimate bending moment fields which are substantially different from elastic distributions, but simple reinforcement layouts which provide substantial economy compared with conventional code requirements. The calculated savings in reinforcement for seven different slabs designed by the method of this paper are shown in Table 4.

Table 4 Savings in Reinforcement Achieved using the Proposed Design Method

SLAB DIMENSIONS mm	STEEL YIELD STRENGTH N/mm^2	REINFORCEMENT SAVING %
7320 x 4000 x 150	635	43
8000 x 4000 x 150	635	48 *
7000 x 4000 x 110	410	45
8000 x 4000 x 110	410	53 *
6625 x 4240 x 120	410	44
7420 x 4240 x 120	410	48 *
8480 x 4240 x 120	410	53 *

* Indicates that the Code (4) requires more than the minimum reinforcement ratio in the short span. The alternative design utilizes the minimum ratio.

The test slabs exhibited the delayed cracking and extended 'linear' behaviour usually attributed to compressive membrane action, and failure at large load factors by typical yield line action followed by tensile membrane action.

Deflections at the ultimate load were respectively 0.44, 0.39 and 0.32 times the slab depth, but much greater deflections could have been achieved with the slabs acting in their tensile membrane action range.

CONCLUSION

The slab tests described add a little to what is still relatively small and inadequate fund of information on the behaviour of slabs designed by the Hillerborg Strip Method. It is suggested that the design procedure used in this paper is simple, economical and practical. In the slabs tested it resulted in satisfactory behaviour at both service and ultimate loads.

Extension of the design method to include optimization of load dispersion by choosing $\alpha \neq 0$, for larger, more heavily loaded slabs, is currently being investigated experimentally.

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DESIGN OF MULTIPLE PANEL FLAT SLAB STRUCTURES

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ABSTRACT. An extensive model study was carried out to determine the distribution of moments in flat slab structures under extreme pattern loading. The experimental results were compared with the empirical method of C.P.110:72 and the direct design method of A.C.I. 318:77. Analysis showed that both design methods are satisfactory for negative moments but underestimate the magnitude of the positive moments. With the current A.C.I. 318:77 limitations on live load to dead load ratio, reversal of sign can occur for 'nominally' positive moments. C.P.110 should introduce a limit on the live load to dead load ratio.

INTRODUCTION

Flat slab construction, which sometimes incorporates drop panels or edge beams, is a versatile economical method of construction often used for multi-floor office and residential buildings. Its popularity is due to its ease of construction, economy of formwork and flexibility of use as the enclosed space can be rearranged as required. The design of flat slabs is generally carried out using either an approximate method known as the empirical method, C.P.110:Part 1:1972 (1), and the direct design method, A.C.I. 318:77 (2), A.S.1480:74 (3), C.S.A. A23.3M 1978 (4) or the continuous (equivalent) frame method.

The empirical method, C.P.110:72 clause 3.6.6, and the direct design method, A.C.I. 318:77 section 13.3, A.S.1480:74 clause 21.4 and C.S.A. A23.3M:78 clause 11.4,are essentially similar. The empirical method is a modified elastic analysis in which equilibrium is satisfied but compatability is not formally satisfied but is reasonably satisfied provided the limitations on the method are met, namely: (i) the slabs should comprise a series of rectangular panels of approximately constant thickness with at least three rows in two directions and the ratio of the length of a panel to its width should not exceed 4:3, (ii) the stability of the structure is provided by bracing or shear walls designed to resist all the lateral force, (iii) the lengths and widths of any two adjacent panels should not differ by more than 15 per cent of the greater length or width, except in no case shall an end span be larger than the adjacent interior span, (iv) where drops are provided they should be rectangular in plan and have a length in each direction not less than one-third of the panel length in that direction.

The limitations on the direct design method are: (i) a minimum of three continuous spans in each direction, (ii) rectangular panels with the ratio of longer span to shorter span not greater than 2, (iii) successive span lengths in each direction shall not differ by more than 1/3 of the longer span, (iv) columns should not be offset by more than 10 per cent of span and (v) live load should not exceed three times dead load.

It is of interest to consider the differences between C.P.110:72 limitations and those required by A.C.I. 318:77 et al. The American, Australian and Canadian codes all have a limit on the maximum live load/dead load ratio for which the empirical method can be used. Further, the panel aspect for which the empirical method can be used are more restrictive in C.P.110:72 than the other three codes. Finally, the American, Australian and Canadian codes all have provision for edge beams whereas Section 3.6 of C.P.110:72 does not permit incorporation of edge beams. The A.C.I.-A.S.-C.S.A. codes have a requirement for increasing the positive moments for pattern loading by a coefficient which is a function of live load/dead load ratio and relative column stiffness [A.C.I. 318:77 clause 21.4.6, C.S.A. A23.3M:77 clause 11.4.6].

The continuous frame method, C.P.110:72 clause 3.6.5, and the equivalent frame method, A.C.I. 318:77 section 13.7, A.S.1480:74 rule 21.5 and C.S.A. section 11.5, are identical. The structure is analyzed as equivalent frames on the column lines taken longitudinally and transversely through the building bounded laterally by the centre lines of the panels. The loading patterns to be considered are dead load on all spans and live load on alternate spans or two adjacent spans. Analysis is performed by a conventional elastic analysis.

Both the empirical method and the continuous frame method being essentially elastic analyses give lower bound solutions to the flexure problem and shear still needs careful consideration.

The moments determined at a section have to be proportioned across the width of the slab. All codes consider the slab to be divided into column strips and middle strip. The moment acting at a section is proportioned to give the moment on the column strip and middle strip. The American, Australian and Canadian Codes use the same allocation of section moment between column strips and middle strips for both the empirical method and the continuous frame method. The British Code, C.P.110:72, has insignificantly different cross slab distributions for the two methods. The Australian Code, clause 9.2.7, also permits analysis by yield line theory. However, distribution of reinforcement is complicated in yield line analysis in that the collapse mechanism is a function of the reinforcement distribution.

This study was undertaken as the authors thought the code distributions of moment between column strips and middle strips across a panel could be in error. Further, for the empirical method two additional potential problems appeared possible, namely: (i) the distribution of the total static design moment into its positive and negative components under pattern loading and (ii) the possibility that the normally positive moment at midspan could be negative under high live load/dead load ratios.

To investigate these potential problems a model investigation was carried out using a 1/12 scale perspex model of a three bay by three bay flat slab structure.

MODEL INVESTIGATION

The perspex model used in this investigation had two storeys each of three bay by three bay configuration. The model was designed to be a 1/12 scale model of the flat slab structure shown in Figure 1, with a slab thickness of 152 mm (6 in) and bay dimensions of 4.88 m x 4.27 m (16 ft x 14 ft). Equivalent full scale columns would be 305 mm (12 in) diameter with 3.05 m (10 ft) clear height between floors.

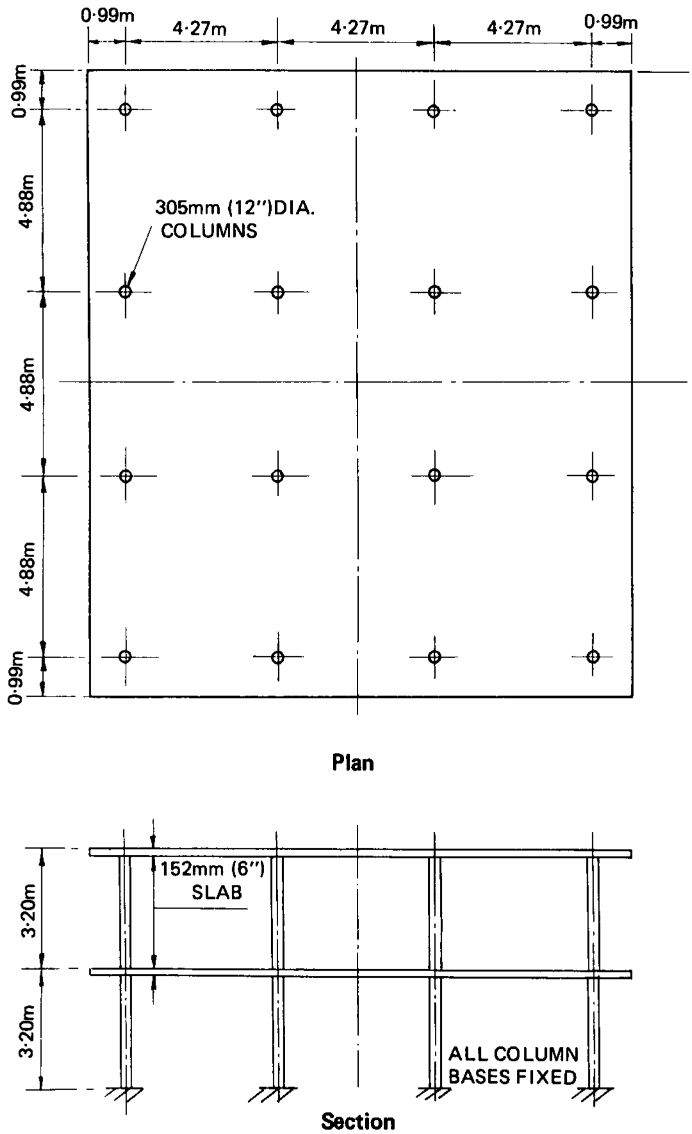

Figure 1 Layout of the Prototype

Model response was measured by foil type electrical resistance strain gauges for surface strains and dial gauges for deflections.

Uniformly distributed loading, using an air bag, was applied one panel at a time successively to the various bays. By summation of the relevant one panel responses the response of the model to any pattern of loading could be determined. The patterns of loading considered are those shown in Figure 2, plus the case of the live load on all panels.

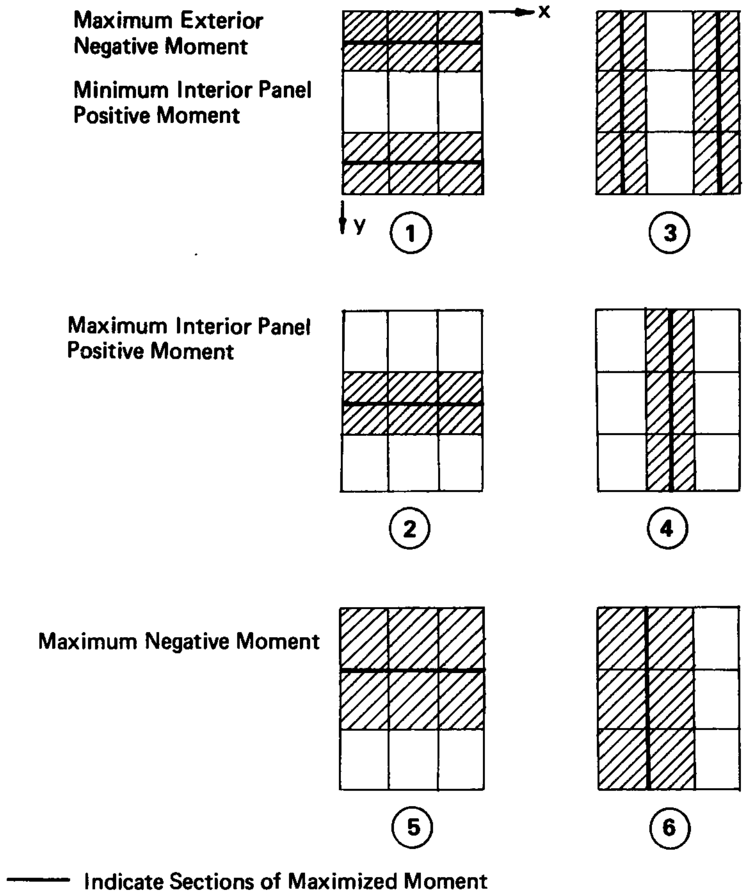

Figure 2 Loading Patterns

The results presented for the various load patterns are based upon the live load being three times the dead load with the loads factored by 1.4 for the dead and 1.7 for the live load (A.C.I.-C.S.A. limiting condition).

DISCUSSION OF RESULTS

Variation of Interior Negative Moment across Panel

A typical variation of negative moment across an interior bay is shown in Figure 3. It can be noted that the design moment distribution is not similar to the experimentally measured distribution. The design moments appear to account for pattern loading effects satisfactorily in terms of magnitude of section moment.

Variation of Exterior Negative Moments across Panel

A typical distribution of exterior negative moment is shown in Figure 4. The positive moment over the middle strip at the end of an end panel is an aberation due to the model slab overhanging past the edge column line. In this case the design

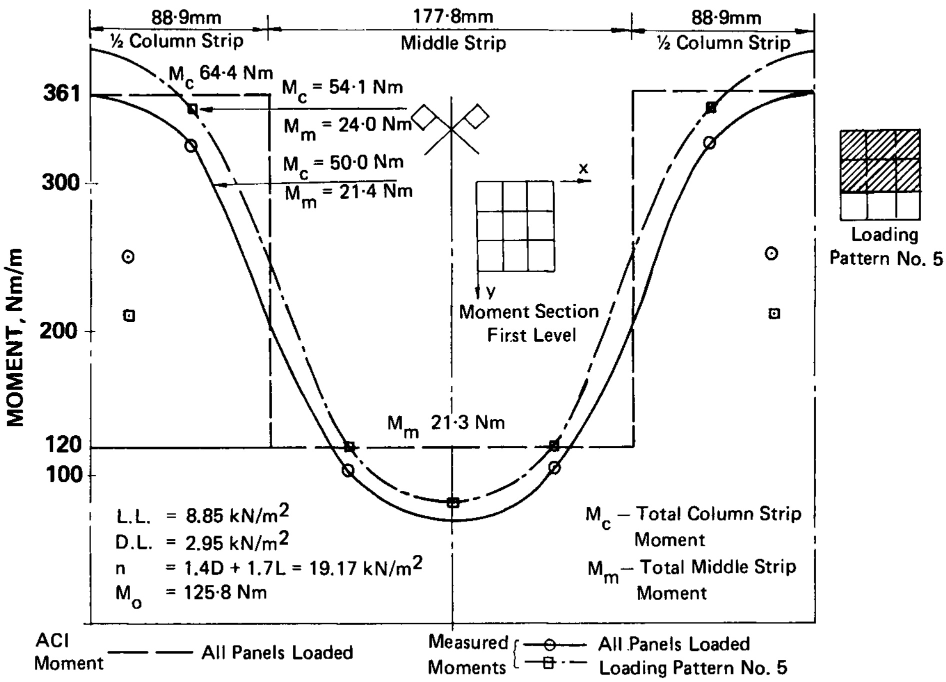

Figure 3 Comparison of A.C.I. Moments and Measured Moments for an Interior Span (moment variation across width of maximum negative moment section)

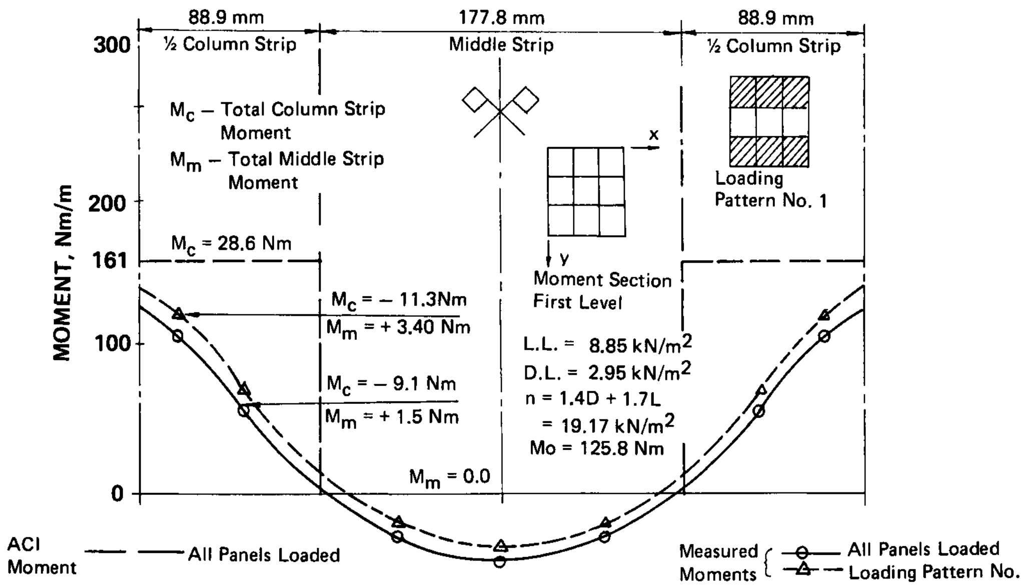

Figure 4 Comparison of A.C.I. Moments and Measured Moments for an End Span (moment variation across width of maximum negative moment section)

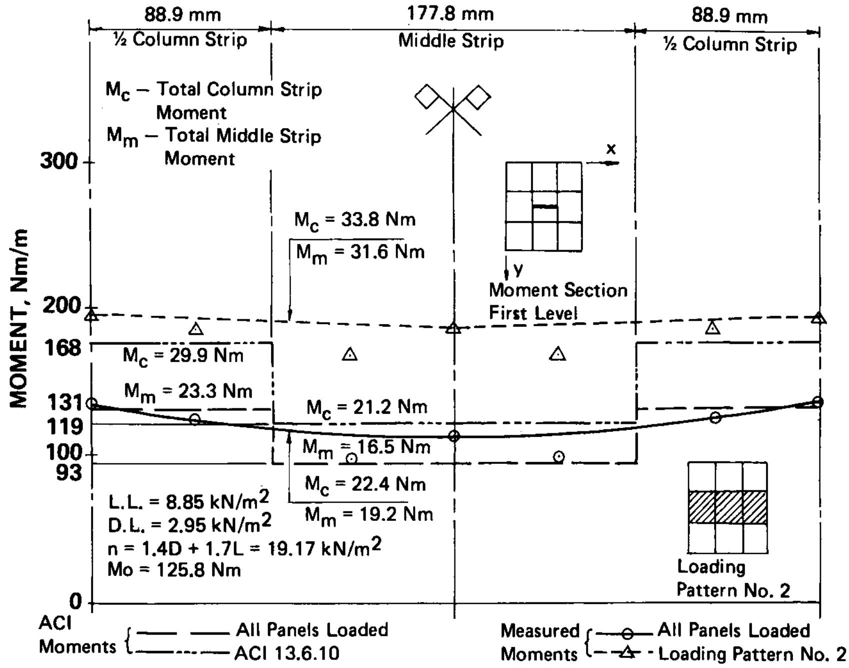

Figure 5 Comparison of A.C.I. Moments and Measured Moments for an Interior Span (moment variation across width of maximum positive moment section)

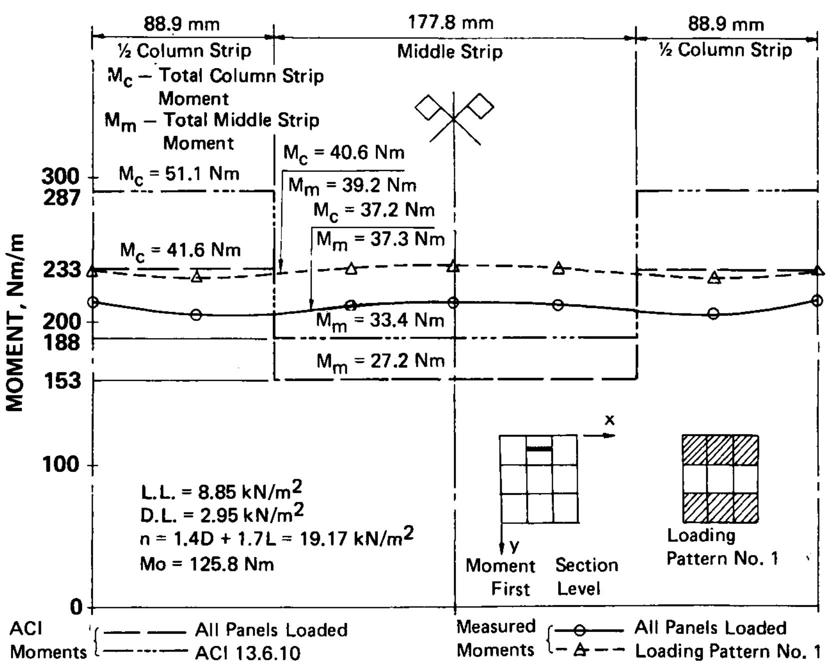

Figure 6 Comparison of A.C.I. Moments and Measured Moments for an End Span (moment variation across width of maximum positive moment section)

moments are again greater than the measured moments for all patterns of loading.

Variation of Positive Moments across an Interior Panel

Figure 5 shows that for all practical purposes the variation of positive moment is uniform across the panel under both uniform loading and pattern loading and that the design moments are less than the measured moments even using Clause 13.6.10 of A.C.I. 318:77. Further, under certain conditions of pattern loading the moment can become negative.

Variation of Exterior Panel Positive Moment

Again in Figure 6, the positive moment distribution can be seen to be approximately constant. In this case, no loading pattern gives rise to a reversal of sign.

Table 1 Comparison of Measured Moments with Design Moments for 'y' Direction Level 1 as Fractions of M_o*

	LOAD PATTERN	EXTERIOR COLUMN 1/2 STRIP	EXTERIOR PANEL MIDDLE STRIP	INTERIOR COLUMN 1/2 Strip	INTERIOR COLUMN 1/2 Strip	INTERIOR PANEL MIDDLE STRIP
Exterior negative moment	Expt All **	-0.10	+0.03	-0.04	-0.04	+0.04
	Expt (1) **	-0.10	+0.03	-0.05	-0.05	+0.02
	A.C.I. 318:77	-0.11	0.00	-0.23		0.00
	C.P.110:72	-0.21	-0.10	-0.41		-0.10
End panel positive moment	Expt All **	0.18	0.27	0.15	0.15	0.30
	Expt (1) **	0.21	0.31	0.16	0.16	0.32
	A.C.I. 318:77	0.17	0.22	0.33		0.22
	A.C.I. 318:77 (Clause 13.6.10)	0.20	0.26	0.41		0.26
	C.P.110:72	0.14	0.20	0.28		0.20
Interior negative moment	Expt All **	-0.27	-0.07	-0.21	-0.25	-0.12
	Expt (5) **	-0.30	-0.09	-0.23	-0.26	-0.14
	A.C.I. 318:77	-0.26	-0.17	-0.51		-0.17
	C.P.110:72	-0.23	-0.16	-0.46		-0.16
Interior positive moment	Expt All **	0.14	0.19	0.10	0.09	0.16
	Expt (1) **	-0.03	-0.04	-0.02	-0.03	-0.06
	Expt (2) **	0.20	0.25	0.14	0.13	0.26
	A.C.I. 318:77	0.09	0.13	0.19		0.16
	A.C.I. 318:77 (Clause 13.6.10)	0.11	0.18	0.24		0.17
	C.P.110:72	0.11	0.16	0.22		0.16

* $M_o = \frac{nL_2}{8}(L_1 - h_c)^2, \quad n = 1.4D + 1.7L$

** Experimental load patterns defined in Figure 2

Effect of Pattern Loading and the Empirical/Direct Design Method

For interior panels both the empirical method and the direct design method insist that the sum of the positive and negative moments equal the equilibrium moment. Thus if one moment is conservative then the other moment is unconservative and moment redistribution must be assumed to occur. For exterior panels C.P.110:72 specifies positive and negative moments whose sum is somewhat greater than the equilibrium moment. The empirical method does not consider pattern loading effects and the direct design method only considers pattern loading effects by increasing the mid panel positive moments.

Table 1 shows that the direct design method overestimates the negative moments and underestimates the positive moments, even taking advantage of the pattern loading clause. The empirical method is sensibly exact for the negative moment but underestimates the positive moments.

No code considers that the normally positive mid span moments can become negative. Either these sections should be reinforced for negative moments or the live load/dead load ratio limited to unity. The positive moment should be divided equally to the column and middle strips.

RECOMMENDATIONS

Empirical Method C.P.110:72 and Direct Design Method A.C.I. 318:77, A.S.1480:74, C.S.A. A23.3M:78

1) Live load/dead load ratio should be limited to unity unless nominal negative moment steel is provided in mid panel regions.
2) Positive moments under extreme pattern loadings are significantly greater than those predicted by the design methods. Hence the design positive moment should be increased to 50 per cent of M_o for interior panels.
3) The proportion of the positive moment taken by the middle strips should be increased to 50 per cent for slabs without edge beams.
4) The sharp discontinuity inherent in having a much higher steel ratio in the column strip compared with the adjacent middle strip at their common boundary could cause severe cracking parallel to the column strip and other methods of patterning reinforcement should be considered.

Empirical Method C.P.110:72

5) The range of application of the empirical method should be expanded to include edge beams and panels of larger aspect ratio.
6) A clause to increase the interior panel positive moments under high live load/dead load ratios should be incorporated.

Direct Design Method A.C.I. 318:77, A.S.1480:74, C.S.A. A23.3M:78

7) The present insistence that the sum of the positive and negative moments equals the static moment needs reconsideration. The concentration of negative moment steel in the column strips and the high punching shears may inhibit the moment redistribution needed to satisfy equilibrium.
8) The intent of A.C.I. 318:77 clause 13.6.10 would be satisfied if the positive moments were increased as in recommendation 2) above.
9) Minimum negative moment steel be provided for the middle strips of end panels.

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A CRITICAL REVIEW OF TWO EXISTING METHODS OF DESIGN FOR FLAT SLABS IN LIGHT OF A NEW COMPREHENSIVE METHOD

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ABSTRACT Both methods of design of A.C.I. 318:71 are analysed by means of a new method of analysis. Serious weaknesses are revealed in the calculation of the negative design moments. The statics of the total design moment, M_o, are substantiated and it acquires major theoretical importance. The results of theoretical and experimental studies are interpreted by the new method and the concept of the dual frame is evolved. This concept is applied to design for lateral forces in the form of the split moment distribution.

INTRODUCTION

The purpose of this study is to examine by means of a new method of analysis both methods of flat slab design of A.C.I. 318:71 (1), and to interpret the results of theoretical and experimental studies (2,3). As a result, a new comprehensive method of moment distribution is evolved: the dual frame method.

The new method of analysis is grapho-analytical. It is simple, visually meaningful and gives answers in moments rather than mathematical equations, the meaning of which need not be immediately apparent. It is based on two concepts of the moment area method of analysis (4). Firstly, the concept of the transformed moment diagram. This diagram represents graphically the equation $M/IE = f(x)$. Secondly, the principle of equivalence of moment area and rotation expressed thus: 'If in a simply supported loaded beam the transformed moment diagram is superimposed as an imaginary load, then the reaction of this load at a support (divided by the basic IE) equals the rotation of this end relative to the line of the beam before deflection'.

NOTATION

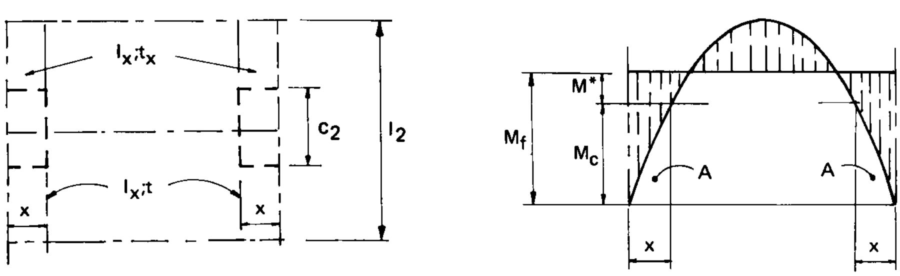
B	Fraction of the basic end moment, M_B, contained in the width, c_2, of the support.
B'	Fraction of the fixed end moment of the slab beam contained in the width, c_2, of the support.
B''	Fraction of the fixed support moment contained in the width, c_2, of the support.
FM, FM_a, FM_b	Fixed support moment.
FM', FM'_a, FM'_b	Final support moment after moment distribution.

$\dot{K}$	Absolute stiffness equals: $\dot{k}IE/l$.
K	Stiffness as fraction of the total stiffness of the joint.
l_{1n}	Clear span in the direction of moments.
l_{2n}	Clear span in the transverse direction to that of the moments.
M, M_a, M_b	Fixed end moment of a slab beam end.
M_c, M_{ca}, M_{cb}	Positive moment at the face of the support: $0.5(x-x^2)$
M_f	Fixed end moment assuming an intersection of theoretical centre lines of the members of a joint.
M_B, M_{Ba}, M_{Bb}	Fixed end moment of a 'beam member' end; the basic end moment.
M_S, M_{Sa}, M_{Sb}	Fixed end moment of a 'slab member' end.
M', M'_a, M'_b	Final end moment of a slab beam end after moment distribution measured on the centre line of the column.
M'', M''_a, M''_b	As above but measured on the face of the support.
M^*, M^*_a, M^*_b	Negative design moment of a slab beam end.
M^*_e, M^*_d	As above calculated by the equivalent frame and by the direct design method respectively.
M^*_i, M^*_s	Negative design moment from Table 5(2) and scaled from Figures 6 and 7(3), respectively.

Note. The span $l_1 = 1$, and all lengths are expressed as fractions of l_1.
The total uniform load: $wl_1l_2 = 1$.

THE EQUIVALENT FRAME METHOD OF DESIGN

The basic assumptions of this method (1) are as follows. Firstly, that the moment of inertia, I_x, of the transverse sections of the ends of the slab beam, from the centre of the support to its face, see Figure 1(a), is equal to the moment of inertia, I, at the face of the support divided by the quantity $\rho = (1 - c_2/l_2)^2$. In the case of a flat plate this assumption amounts to an imaginary increase in the thickness of the slab, to $t_x = t/\rho^{1/3}$, in the transverse strip between the faces of the support. The second assumption places the critical section for the negative design moment at the face of the support. The third assumption concerns the equivalent column which consists of the actual columns plus an attached transverse torsional member.



(a) Plan **(b) Moment Diagram**

Figure 1 Details of Slab Beam

The calculation of the negative moments in this method commences with the fixed end moments. The design moments are not immediately available. The new method of analysis facilitates a simple and direct calculation. Referring to Figure 1(b) the moment area A for the ordinate x is equal to $\{(x^2/4)-(x^3/6)\}$. Thus for x = 0.5, A = 0.04167 (5). For zero rotation at the fixed ends, see Figure 2,

$$M_f[0.5 - x(1 - \rho)] = 0.04167 - A(1 - \rho)$$

and therefore

$$M_f = [0.04167 - A(1 - \rho)]/[0.5 - x(1 - \rho)]$$

The critical section being at the face of the support, $M_e^* = M_f - M_c$.

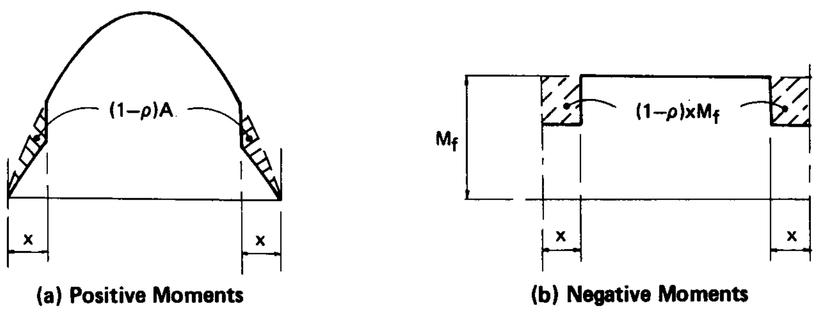


Figure 2 Transformed Moment Diagrams

Tables 1 and 2 show a comparison of moments thus calculated with moments calculated by the direct design method and also with moments calculated by elastic plate theory and verified experimentally (2,3). The differences between the moments calculated by both methods of the Code (M_e^* and M_d^*) increase with the c_1/l_1 ratio, reaching a maximum of 52 per cent in Table 1 and 170 per cent in Table 2. The maximum differences between the moments M_e^* and the theoretical moments are, correspondingly, 37 per cent and 135 per cent.

Table 1 Comparison Moments for $c_1 = c_2 = c$; $l_1 = l_2$

c	M_e^*	M_d^*	M_i^*	M_d^*/M_e^*	M_i^*/M_e^*
0.10	0.0610	0.0658	0.0632	1.08	1.03
0.20	0.0430	0.0520	0.0464	1.21	1.08
0.25	0.0354	0.0457	0.0401	1.29	1.13
0.35	0.0225	0.0343	0.0310+	1.52	1.37

\+ Extrapolated

Table 2 Comparison Moments for $c_2 = 0.05$; $l_1 = l_2$ (Ref 5)

c	M_e^*	M_d^*	M_s^*	M_d^*/M_e^*	M_s^*/M_e^*
0.05	0.0715	0.0733	0.072	1.02	1.00
0.15	0.0496	0.0587	0.053	1.18	1.07
0.25	0.0300	0.0457	0.040	1.52	1.33
0.35	0.0127	0.0343	0.030	2.7	2.35

Referring to Figure 1(b) it can be seen that the ratio M_f/M_e^* is greater than t_x/t. There is no reason, therefore, for locating the critical section at the face of the support. Seemingly, the demand of some researchers for a 'full' negative design moment is justified.

The Code (1) deserves great credit for highlighting the torsional aspect of moment transfer in flat slabs. However, the theoretical expression for the stiffness of the torsional member, K_t, required a correction factor of 3 to bring it into agreement with experimental results (6). In a subsequent section it will be shown that no correction factor is needed in the dual frame method.

THE DIRECT DESIGN METHOD

If infinite moment of inertia is assumed for the transverse sections of the slab beam ends, from the centre of the column to the face of the support, referring to Figures 1 and 2, $I_x = \infty$, $\rho = 0$ and the areas A and the areas $M_f x$ are cancelled. For zero rotation of the fixed ends:

$$0.5 l_{ln} M_f = 0.5 l_{ln} M_c + 0.333 l_{ln} M_o$$

therefore:

$$M_f = M_c + 0.667 M_o.$$

Since

$$M_d^* = M_f - M_c$$

therefore:

$$M_d^* = 0.667 M_o \tag{1}$$

The corresponding expression in the Code is rounded off to $M = 0.65M_o$. Thus it is obvious that this method is based on the assumption of infinite moment of inertia for the end parts of the slab beam between the faces of the support. This assumption differs completely from that of the previously discussed method. What is more it appeared in the rules for design by elastic analysis of a previous Code (7) but was intended to be dispensed with for several weighty reasons (5).

THE STATICS OF THE SLAB BEAM

The expression for the total design moment stems for experimental work (8,9,10). To prove its statics a scheme of load components is assumed, see Figure 3(a), that follows the pattern of the yield lines of an experimental slab, see Figure 69 (11). The summed reactions are shown, in elevation, in Figure 3(b). If the two opposite forces of 0.5 at each end of the member are added, a positive and a negative moment diagram are obtained. The balance of these diagrams is the total design moment, $M_o = 0.125 l_{ln}^2$, see Figure 3(c).

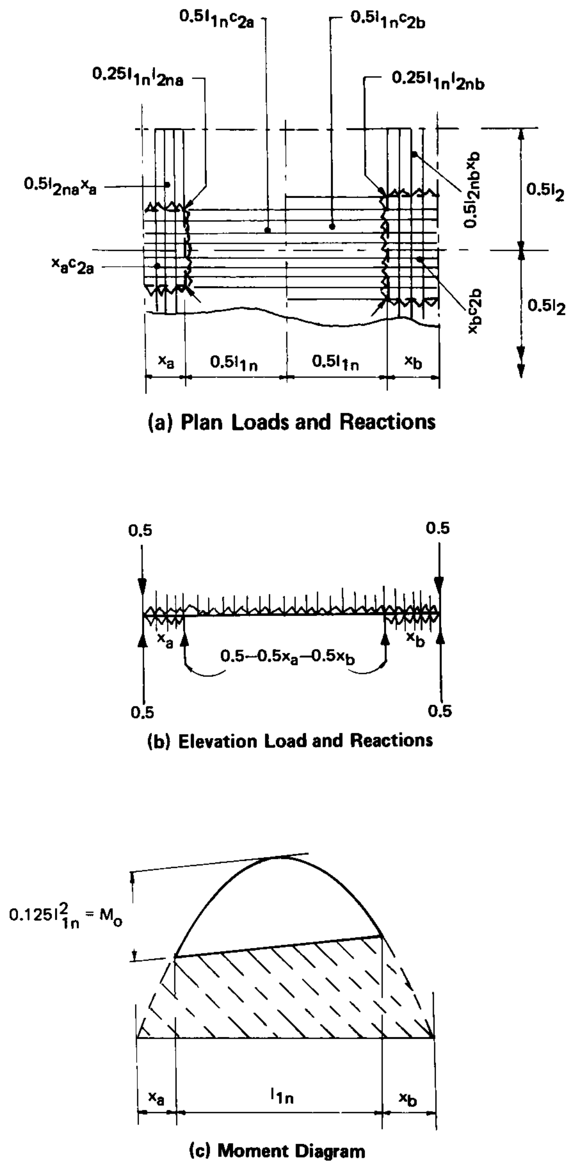
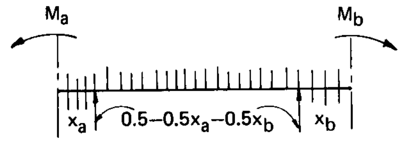
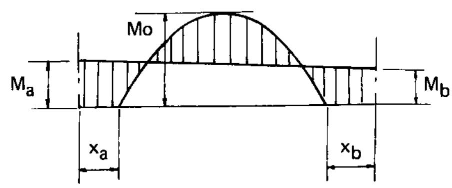


Figure 3 Total Design Moment

To calculate the fixed end moments of the slab beam it is considered as a free body. Figure 4(a) shows, in elevation, the loaded member and its reactions, as in Figure 3(b), and also the fixed end moments (5). The positive and negative moment diagrams are shown in Figure 4(b).



(a) Load and Reactions



(b) Moment Diagram

Figure 4 Fixed End Moments

The fixed support moment is a new concept. Figure 5(a) shows the support at the end a of a slab beam a - b as a free body. Taking moments to the centre of the column:

$$FM_a = M_a + 0.5x_a^2 + 0.5x_a - 0.5x_a^2 - 0.5x_a x_b$$

Now

$$0.5x_a - 0.5x_a^2 = M_{ca} ,$$

$$0.5x_b - 0.5x_b^2 = M_{cb} ,$$

and

$$0.5x_a^2 - 0.5x_a x_b = (M_{ca} - M_{cb})x_a/l_{ln} .$$

Designating

$$M'_{ca} = M_{ca} + (M_{ca} - M_{cb})x_a/l_{ln} \tag{2a}$$

and

$$M'_{cb} = M_{cb} + (M_{cb} - M_{ca})x_b/l_{ln} \tag{2b}$$

then

$$FM_a = M_a + M'_{ca} \tag{3a}$$

and

$$FM_b = M_b + M'_{cb} \tag{3b}$$

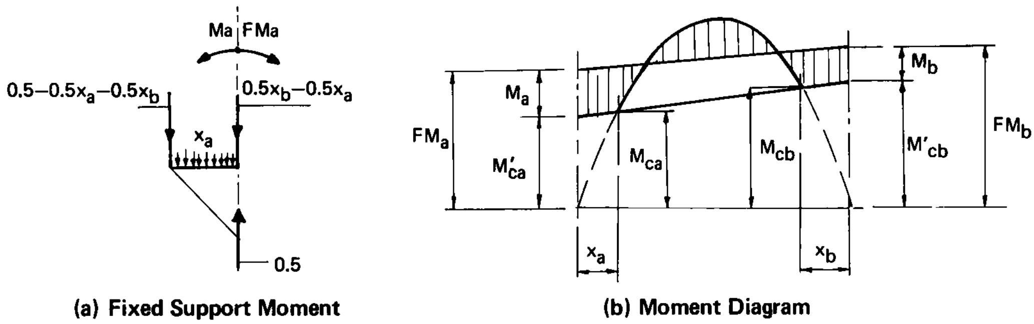


Figure 5 Fixed Support Moment

The equations (2a) to (3b) are illustrated in the moment diagram of Figure 5(b). If an intersection of theoretical centre lines of the members of a joint ($x_a = x_b = 0$) is assumed then the diagram becomes identical with that of Figure 1(b). A comparison of these two diagrams proves the exceptional importance of Figure 5(b): it resolves a controversy of long standing and provides rigorous proof for reduced negative design moments.

THE DUAL MEMBER CONCEPT

The diagrams produced by elastic plate theory, see Figures 24 to 26 (2), showing the transverse distribution of the fixed end moments, Figure 6, display two features of considerable interest. Firstly, discontinuity: from a low value M_m the moment curve rises gradually then drops to zero at the side of the support. Secondly, high concentration of moment over the width, c_2, of the support. This points clearly to two distinctly different modes of action: that of a rigid 'beam member' and that of a less rigid 'slab member'. The action of both 'members' is superposed in the slab beam. Both 'members' are geometrically identical with the slab beam. For the 'beam member' we assume infinite moment of inertia for the transverse strips between the faces of the support. Its contribution to the fixed end moment is BM_B. For the 'slab member' we assume knife edge supports on the transverse centre lines of the columns. Its contribution is the remainder of the basic end moment downgraded by slab action to $(1 - B)M_B r_m$, where $r_m = M_S/M_B$. The total fixed end moment of the slab beam is therefore:

$$M = M_B[B + (1 - B)r_m] \tag{4}$$

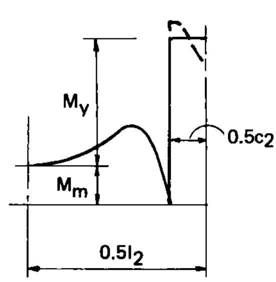


Figure 6 Transverse Moment Distribution

In the case of square symmetrical slab beams without drop, if $c_1 = c$ and $c_2 = c$, it can be assumed that $B = 2c$. From Figures 7(a) and (b) $M_B = 2M_o/3$, while from Figures 7(a) and (c), $M_S = 2M_o l_n/3$ and $r_m = l_n$.

Therefore

$$M = 2M_o[2c + (1 - 2c)(1 - c)]/3,$$

hence

$$M = 2M_o(l_n + 2c^2)/3 \qquad (5)$$

For symmetrical, rectangular slab beams see (5). The agreement with the theoretical moments as shown in Tables 3 and 4 is excellent.

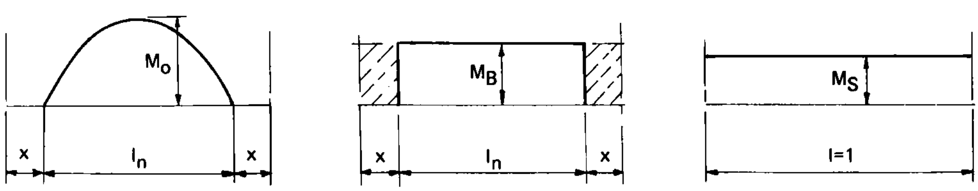
Table 3 Comparison Moments for $c_1 = c_2 = c$; Eq. (5)

c	M_i^*	M^*
0.10	0.0632	0.0621
0.20	0.0464	0.0469
0.25	0.0401	0.0410
0.35	0.0310$^+$	0.0315

+ Extrapolated

Table 4 Comparison Moments for $c_2 = 0.05$ (Ref. 5)

c_1	M_s^*	M^*
0.05	0.0720	0.0718
0.20	0.0465	0.0457
0.35	0.0300	0.0300



(a) Positive Moment Diagram

(b) Beam Member, Transformed Negative Moment Diagram

(c) Slab Member, Negative Moment Diagram

Figure 7 Fixed End Moments 'Beam Member' and 'Slab Member'

THE ELASTO PLASTIC STAGE OF BEHAVIOUR

By definition $BM_B = B'M$. From equations (4) and (5), $BM_B = B'M_B(1 + 2c^2)$ and $B' = B/(l_n + 2c^2)$. The average value of the peak moment at the support, see Figure 6, is $B'/c = 2.2$. The column strip of the Code (1) allows for a maximum moment of $0.75/0.5 = 1.5$. Unless a considerable increase in the thickness of the drop or slab is acceptable, a certain limited re-distribution of peak moments must be allowed for (12). We are thus dealing with elasto-plastic behaviour. For this stage several assumptions must be made. It must be assumed, firstly, that the value of the minimum moment, M_m, will not be affected by the redistribution. Secondly, that the curve of the transverse distribution of the fixed end moment is expressed as $y = M_y(X/l_{2n})^2$, see Figure 8. Thirdly that the slab beam as a whole will behave elastically for the purpose of moment distribution by the Cross method.

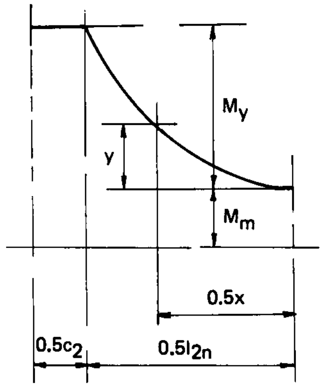


Figure 8 Transverse Distribution End Moment

For the minimum moment the suggested expression is (5):

$$M_m = 0.45k_1k_2k_3M \qquad (6)$$

where k_1 expresses the influence of the ratio l_{1n}/l_{2n}, k_2 that of the ratio c_1/c_2 and k_3 that of the stiffness of the exterior edge, where it applies (5). Referring to Figure 8, designating

$$k_n = c_2/l_2 + l_{2n}/3l_2 \qquad (7)$$

then

$$M_y = (1 - M_m)/k_n.$$

Since

$$B' = (M_m + M_y)c_2/l_2\ ,$$

therefore

$$B' = [M_m + (1 - M_m)/k_n]c_2/l_2. \qquad (8)$$

From equation (4),

$$B = [B + (1 - B)r_m]B'$$

therefore

$$B = B'r_m/[1 - B'(1 - r_m)] \qquad (9)$$

THE DUAL FRAME CONCEPT

The concepts of 'beam member' and 'slab member' are now extended to those of 'beam frame' and 'slab frame'. The 'beam frame' consists of the 'beam member' and the columns acting with the full stiffness $\Sigma\dot{K}_c$. The fixed support of this frame is $FM_B = M_c^{\prime} + BM_B$, see Figure 5(b). The 'slab frame' consists of the 'slab member' and columns of reduced stiffness $\dot{K}_c^e$. The fixed end moment of the 'slab member' is transferred torsionally to the column ends and their stiffness is reduced to $\dot{K}_c^e = 1(\theta_t + 1/\Sigma\dot{K}_c)$, where θ_t is the torsional rotation of a member c_1 wide by the depth of the slab including the stem of a transverse beam, see Appendix II of reference (5). The fixed support moment of this frame is $FM_S = (1 - B)M_b r_m$ and $FM_S = M_S(1 - B)$. The fraction of the fixed support moment contained in the width c_2 of the support is:

$$B'' = FM_B/(FM_B + FM_S) \tag{10}$$

All the required information is now available for the moment distribution in each of the 'frames'. For simplicity it is combined into one. Figure 9 (a) and (b) shows the absolute stiffnesses of the member ends of a joint in the 'beam' and 'slab' frames respectively. Figure 10 (a) and (b) shows the corresponding fractional stiffness $K_B = \dot{K}_B/\Sigma\dot{K}_B$ and $K_S = \dot{K}_S/\Sigma\dot{K}_S$. The combined fractional stiffness is $K_{sb} = B''K_B + (1 - B'')K_S$. The combined carry over factor is $\dot{c}_{sb} = [B''\dot{c}_B K_B + (1 - B'')\dot{c}_S K_S]/K_{sb}$. In all cases the value of B'' is taken from the side of the joint of the unbalanced moment. The fixed support moment is obviously $FM_{sb} = FM_B + FM_S$.

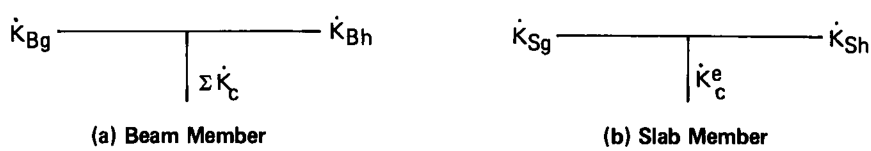


Figure 9 Absolute Stiffnesses

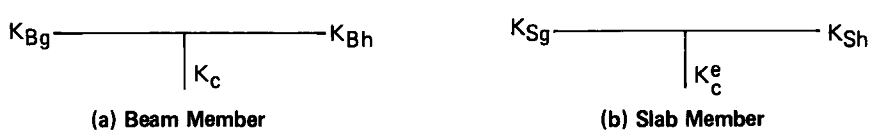


Figure 10 Fractional Stiffnesses

After completion of the moment distribution the final end moments of the slab beam are obtained as follows: $M' = FM' - M_c'$, see Figure 5(b). The design moment on the side of the lesser M' is equal to M'', whereas on the side of the greater M' it is given by $M^* = B'M'' + (1 - B')M'$. The torsional moment is calculated as the difference of $0.5(1 - B')M'$ on both sides of the joint.

The basis of these calculations is the diagram of Figure 8. If further yield is to be safeguarded against then the width of the column strip must equal $0.75/(M_m + M_y)$ for 75 per cent of the negative design moment or $1/(M_m + M_y)$ for the full moment. The strength of the concrete in the drop or slab must be calculated for the moment $M_m + M_y$. Examples of practical calculations are given in reference (5).

DESIGN FOR LATERAL FORCES

Figure 11 shows the assumed transverse distribution of the end moment. The following equations are suggested for the fractions of the end moment B_e' contained in the width of the column c_2, see Figures 11 and 12.

For $l_1 = l_2$, $\qquad B_e' = c_1 + 0.325$ (11a)

$l_1 > l_2$, $\qquad B_e' = (c_1 + 0.325)(l_1 + c_1)/(l_2 + c_1)$ (11b)

$l_1 < l_2$, $\qquad B_e' = (c_1 + 0.325)(l_1 - c_1)/(l_2 - c_1)$ (11c)

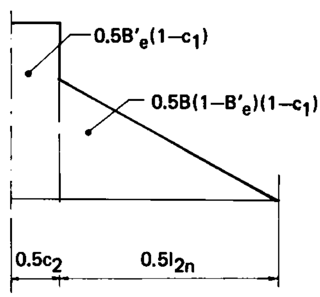


Figure 11 Transverse Moment Distribution

As in the case of vertical forces the reactions are at the face of the column and therefore the moment there is $M_e^c = c_1$ and the total moment contained in the width of column is, from Figure 12:

$$B_e'' = c_1 + B_e'(1 - c_1) \tag{12}$$

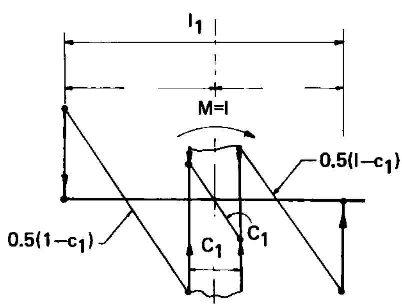


Figure 12 Slab Beam Joint, Far Ends Hinged

In the cases where the moment is cranked in from the column, it is assumed to be very short and therefore much stiffer than the slab beam. It is thus possible to express the stiffness of the slab beam as (15):

$$\dot{K}_{sb} = B_e''\Sigma\dot{K}_B + (1 - B_e'')/(\theta_t + 1/\Sigma\dot{K}_S) \tag{13}$$

where $\theta_t = l_{2n}/6C$, see Appendix II of reference (5). The equivalent width is $Y_e = K_{sb}/\Sigma K_S$. The values of Y_e thus calculated are presented in Figure 13; the agreement with the theoretical values (13,14) is very good.

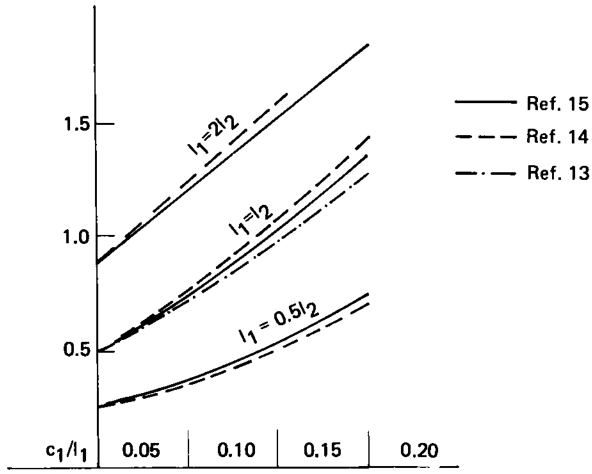


Figure 13 Slab Beam Equivalent Width

THE SPLIT MOMENT DISTRIBUTION

The concept of 'equivalent width' is now discarded in favour of a more comprehensive method, that of moment distribution. The ordinate of the point of contraflexure of the slab beam a - b, see Figure 14, is as follows:

$$0.5l' = B''_a/(1 + \dot{c}_{Ba}) + (1 - B''_a)/(1 + \dot{c}_{Sa}) \tag{14}$$

Designating $c' = c_1/l'$ and $l_2' = l_2/l_1$, the value of B'' is calculated by adopting a value of B''_e slightly greater than the one for c_1 and l_1, and rechecked from either of the equations (11) and then from equations (12) and (13). Graphs for B'' are given in reference (15).

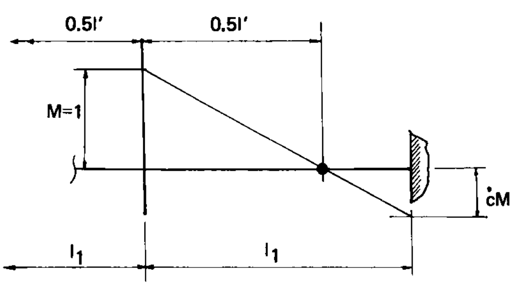


Figure 14 Slab Beam Joint, Far Ends Fixed

The joint receives two kinds of unbalanced moments from the columns, see Figure 15. Firstly, the components $(\bar{M} + \dot{M})\bar{m}/\Sigma\bar{m}$, see Appendix I reference (15). Secondly, the carried over moments from the far ends of the columns. The total effective stiffness of the 'slab member' ends is $\dot{K}^g_S = 1/[\theta_t + 1/(\dot{K}_{Sg} + \dot{K}_{Sh})]$. The effective stiffness of each end is:

end g $$\dot{K}^{e}_{Sg} = \dot{K}^{e}_{S}\dot{K}_{Sg}/(\dot{K}_{Sg} + \dot{K}_{Sh}) \qquad (15g)$$

end h $$\dot{K}^{e}_{Sh} = \dot{K}^{e}_{S}\dot{K}_{Sh}/(\dot{K}_{Sg} + \dot{K}_{Sh}) \qquad (15h)$$

The stiffnesses of the 'beam frame' are obvious. The rest of the procedure is as for vertical forces except that the fractions B''_g and B''_h are used for the slab ends and the average fraction B''_{av} for the column ends, where $B''_{av} = (B''_g K_{Bg} + B''_h K_{Bh})/ /(K_{Bg} + K_{Bh})$. The actual carry over factors are used for the column ends. The combined stiffnesses of the member ends are entered into the Figure 15 and marked K^c.

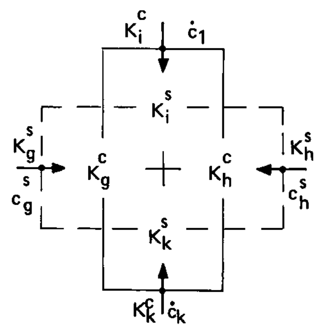


Figure 15 Split Moment Distribution

The joint also receives the carried over moments from the far ends of the slab beams. The total effective stiffness of the column ends, see Figure 15, in the 'slab frame' is $\dot{K}^e_S = 1/[\theta_t + 1/(\dot{K}_i + \dot{K}_k)]$. The effective stiffness of each column end is:

end i $$\dot{K}^{e}_{Si} = \dot{K}^{e}_{S}\dot{K}_{i}/(\dot{K}_{i} + \dot{K}_{k}) \qquad (16i)$$

end h $$\dot{K}^{e}_{Sk} = \dot{K}^{e}_{S}\dot{K}_{k}/(\dot{K}_{i} + \dot{K}_{k}) \qquad (16k)$$

The stiffness of the 'beam frame' are obvious. The conversion into fractional stiffness and the combining of the two frames is as before: the fractions B''_g and B''_h are used for the slab beam ends and the fraction B''_{av} for the column ends. The combined fractional stiffnesses are marked K^S and entered into the Figure 15. The carry over factors of the slab beam ends, see Appendix I, reference (5), are $c^S = \dot{c}_B B'' + \dot{c}_S(1 - B'')$ where the corresponding values of $\dot{c}_B$, $\dot{c}_S$ and B'' are used for each end.

The unbalanced moments from the columns are distributed between the stiffnesses marked K^C and those from the slab beam between the stiffnesses marked K^S. The results at each end are added and used for further distribution.

CONCLUSION

The two dimensional concept of 'dual member' yields results giving excellent agreement with those produced by three dimensional elastic analysis. The 'dual frame' method of moment distribution is, similarly, more accurate than the method of the Code (1).

The split moment distribution method of design for lateral forces is more comprehensive than the portal method implied in the concept of 'equivalent width'.

ACKNOWLEDGEMENT The encouragement and assistance given by Dr. G.D. Base, Senior Lecturer, Civil Engineering Department, Melbourne University, is gratefully acknowledged.

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DESIGN OF CONCRETE SLABS FOR INFINITE IN-PLANE STIFFNESS

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ABSTRACT The assumption that floor slabs are infinitely rigid is very common in the analysis of multi-storey structures. A designer therefore requires design guides and conditions to enable him to design a floor slab which can safely be assumed to be infinitely stiff. The governing equations of lateral load partitioning between end and intermediate shear walls are derived considering a flexible slab. In the derived equations, the height of the structure, slab properties, relative rigidities of shear walls and spacing of shear walls are found to be the main parameters. A relationship is obtained which gives the necessary slab rigidity to make valid the assumption of infinitely stiff floors. A crude conclusion is made on the limiting number of stories after which the importance of floor flexibility is minimized. It has been observed that when 'height-slab property' parameter, z, is greater than 2.5, the floor can be assumed as infinitely stiff. For small values of z, the partitioning of the lateral load is strongly affected by the ratio of flexural rigidities of end and middle shear walls. The spacing of the intermediate shear walls, for a fixed ratio of flexural rigidities, plays a minor role in lateral load partitioning. The results are applied to a practical design example.

INTRODUCTION

A multi-storey building is a three dimensional structure composed of many bents held together by the floor slab and longitudinal beams. When the floor slab is a flat plate, which is common in such structures, the only element to bind the bents together and form a three dimensional structure is the floor slab itself. In the lateral load analysis of buildings, the floors are usually assumed to be infinitely stiff in their plane. This assumption enables the design engineer to partition the lateral load to individual bents in proportion to their bending stiffnesses.

When the floor slab is infinitely stiff, the structure has a fixed center of rigidity; this is not true when the floor flexibility must be considered in the solution. This fact complicates the structural analysis, particularly when eccentric action of the lateral load, producing floor torsion, must be considered. The problem is further complicated by the fact that loads on the slab cannot be concentrated into a single resultant without introducing large errors when the floor slab is flexible. Therefore, to eliminate such complications and achieve three dimensional behavior of the structure, it becomes mandatory to design a tall building to give slabs which can be considered as infinitely stiff in their plane.

Whether the classical assumption of infinitely stiff floors is satisfied or not is almost never checked in structural design. This assumption may be automatically satisfied for framed structures, but for the case of shear wall-frame combinations, the slabs cannot always be assumed as infinitely stiff in their plane, because shear walls may be as stiff as (or stiffer than) the slab. Then, the designer must choose the slab thickness in such a way to yield a slab stiffness which can be assumed as infinitely stiff.

Murashev et al (1) have developed the basic relationship between the shear wall and slab interaction for the case of two end and one intermediate shear walls. However, the influencing parameters have not been studied in sufficient detail to propose guides for design. Muto (2) has considered the case of a deflecting floor affecting the shear force distribution factors of the individual frames. Again, no conditions have been put forward to enable practical design of an infinitely stiff floor slab.

EQUATION OF IN PLANE SLAB DEFLECTION

Consider a slab having three shear walls, two at the ends and one in the middle. This slab is part of a multi-storey structure with equally spaced stories of height h, as shown in Figure 1. The total structure deflection at a certain height is y and this is uniform along the slab length L. However, due to the lateral load, ph, the slab will undergo an additional deflection, thus causing the middle wall to increase its deflection by δ_m relative to the end shear walls. This relative deflection δ_m will result in increased shear forces being carried by the middle wall.

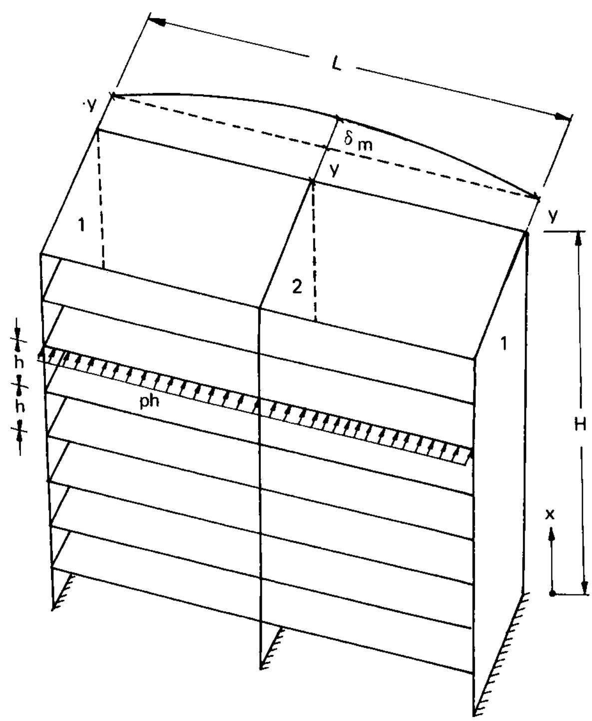

Figure 1 Multi-Storey Structure with Deflecting Floor

The floor slab loaded in its plane by a lateral load of magnitude ph can be considered as a beam on an elastic middle spring, see Figure 2. The deflection of the slab will thus be equal to the deflection under the load ph and the deflection of the concentrated reaction p_2h, that is:

$$\delta_m = \delta_o - \delta_s \tag{1}$$

$$\delta_o = \frac{5}{384}\frac{(ph)(L^3)}{K_f} \tag{2}$$

$$\delta_s = \frac{1}{48}\frac{(p_2h)(L^3)}{K_f} \tag{3}$$

where δ_m = maximum deflection of the considered slab,
δ_o = deflection under a uniformly distributed load of magnitude ph,
δ_s = deflection under a concentrated reaction force of p_2h,
K_f = stiffness of the floor,
L = length of the slab
p = lateral pressure, kN/mm^2,
p_1 = lateral pressure carried by wall (1),
p_2 = lateral pressure carried by wall (2).

Substituting equations (2) and (3) into equation (1) and considering that $p = p_1 + p_2$, an expression for δ_m in terms of p and p_1 is obtained, such that:

$$\delta_m = \frac{p_1 - \alpha p}{S} \tag{4}$$

$$S = \frac{48\ K_f}{hL^3} \tag{5}$$

where $\alpha = 0.38$,
S = stiffness parameter of slab

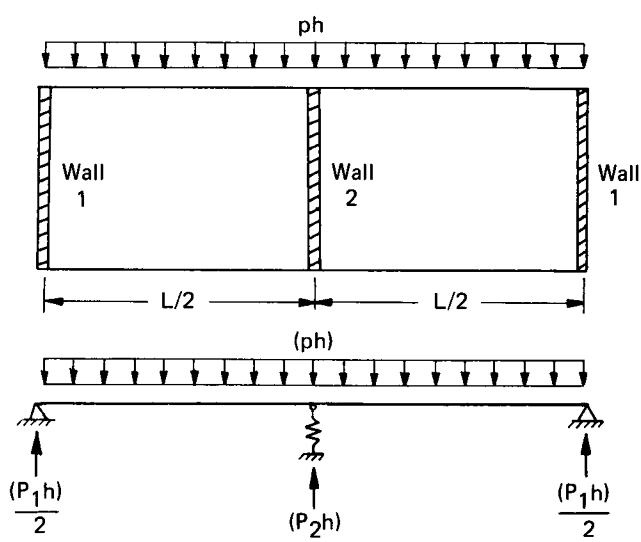

Figure 2 Mathematical Model of a Slab with Three Shear Walls

The same mathematical treatment can be applied to different cases where the number of intermediate walls is increased, the only change being in the coefficient α as shown in Table 1.

Table 1 Slab Deflection with Various Wall Arrangements

Shear walls				
	$n = 3$	$n = 4$	$n = 5$	$n = 6$
α	0.380	0.266	0.210	0.173

GENERAL EQUATION OF BENDING

Consider a slab with n number of shear walls subject to a distributed lateral load of magnitude p, see Figure 3.

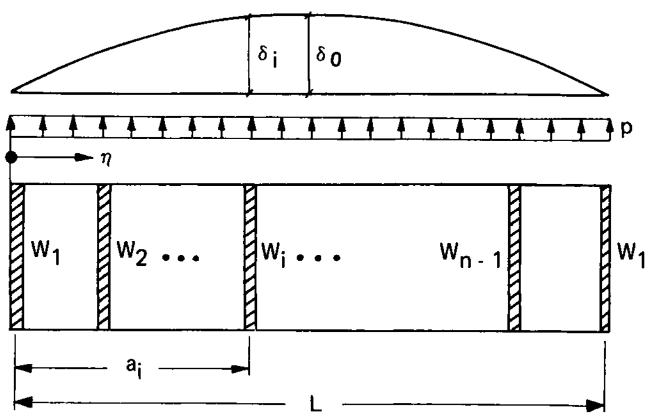

Figure 3 General Floor-Shear Walls Arrangement

The general equation of shear force equilibrium can be written as follows:

$$-K_1 y''' - K_2(y''' + \delta''') + GA(y' + \delta') = Q_o \tag{6}$$

where K_1 = sum of flexural rigidities of end shear walls
K_2 = sum of flexural rigidities of all interior shear walls
GA = shear rigidity of the structure
Q_o = total shear force at the story level considered

If the flexural rigidities of the shear walls are much greater than the shear rigidities of the frames, then the term $GA(y' + \delta')$ can be neglected. This is the common assumption which considers all the shear force to be carried by the shear walls. Expressing equation (6) for n shear walls, equation (7) is obtained as:

$$-\frac{K_1}{2}y''' - K_2(y''' + \delta_2''') - \dots - K_i(y''' + \delta_i''') - \dots - \frac{K_1}{2}(y''' + \delta_n''') = Q_o \quad (7)$$

Thus

$$-K_1 y''' - \sum_{i=2}^{n-1} K_i(y''' + \delta_i''') = Q_o \quad (8)$$

where δ_i = floor deflection at $\eta = a_i$.

Differentiating once more with respect to x, an equation for the distributed lateral load equilibrium is obtained as:

$$K_1 y^{IV} + \sum_{i=2}^{n-1} K_i(y^{IV} + \delta_i^{IV}) - p = 0 \quad (9)$$

Remembering that $p_1 = K_1 y^{IV}$ and substituting this in equation (1) and solving for y^{IV}, equation (10) is obtained:

$$y^{IV} = \frac{S\delta_m + \alpha p}{K_1} \quad (10)$$

Substituting equation (10) in equation (9), an equation in terms of δ_m is obtained as:

$$K_1\left(\frac{S\delta_m + \alpha p}{K_1}\right) + \sum_{i=2}^{n-1} K_i\left(\frac{S\delta_m + \alpha p}{K_1} + \delta_i^{IV}\right) - p = 0 \quad (11)$$

If a second degree parabola is assumed for the floor deflections, δ_i can be expressed in terms of δ_m.

$$K_1\left(\frac{S\delta_m + \alpha p}{K_1}\right) + \sum_{i=2}^{n-1} K_i\left(\frac{S\delta_m + \alpha p}{K_1} + \frac{4a_i^2}{L^2}\delta_m^{IV}\right) - p = 0 \quad (12)$$

Re-arranging equation (12) yields equations (13) and (14):

$$\left\{\frac{4}{L^2}\sum_{i=2}^{n-1} K_i a_i^2\right\}(\delta_m^{IV}) + S\left\{\frac{\sum_{i=2}^{n-1} K_i}{K_1} + 1\right\}\delta_m + \left\{\alpha\frac{\sum_{i=2}^{n-1} K_i}{K_1} + \alpha - 1\right\}p = 0 \quad (13)$$

$$\frac{c^4}{4}\delta_m^{IV} + \delta_m + bp = 0 \quad (14)$$

where $c = 2\left[\frac{\sum_{i=2}^{n-1} K_i a_i^2}{SL^2(K/K_1)}\right]^{1/4}$,

$b = \frac{1}{S}(\alpha - \frac{K_1}{K})$,

$K = K_1 + \sum_{i=2}^{n-1} K_i$

Equation (14) is the familiar equation of beams on elastic foundations. For slabs encountered in practice, in which in-plane deflection is expected, the slab can be assumed as a long beam on an elastic foundation. Thus, the solution of equation (14) becomes as follows:

$$\delta_m = A_1 e^{-z} \cos z + A_2 e^{-z} \sin z - bp \tag{15}$$

where $z = \frac{x}{c}$

The boundary conditions of a fixed-end cantilever wall can be applied, i.e. (a) $\delta_m(0) = 0$ and (b) $\delta_m'(0) = 0$.

The solution of equation (14) thus becomes:

$$\delta_m = \beta \frac{p}{S} \{e^{-z} \cos z + e^{-z} \sin z - 1\} \tag{16}$$

where $\beta = \frac{K_1}{K} - \alpha$

Re-arranging equation (16) yields equations (17) and (18) as:

$$\delta = \frac{\beta}{S} (1 - s)p \tag{17}$$

$$s = e^{-z} (\cos z + \sin z) \tag{18}$$

Equating equations (14) and (17) gives the relationship between the lateral load carried by the end walls, p_1, and the lateral load carried by the interior walls, p_2:

$$\frac{p_1}{p} = \{\beta(1 - s) + \alpha\} \tag{19}$$

OBSERVATIONS ON INFINITE IN-PLANE STIFFNESS OF FLOORS

If the floor slab is infinitely stiff, the ratio of the lateral loads carried by the end walls and interior walls must be equal to the ratio of the flexural rigidities of the corresponding walls. In order to design a slab which can be assumed as infinitely stiff, the parameters of equation (19) must be carefully studied.

The variation of p_1/p as a function of z and K_1/K is shown in Figure 4, (a) to (d), in which the vertical line represents the case of lateral load partitioning with infinitely stiff floors. It can be observed that almost independently of the value of the ratio K_1/K, the value of z which enables an infinitely stiff floor to be assumed is $z > 2.5$. Greater values of z causes a very slight increase in the lateral load carried by the end walls, but this increase is quite negligible. When $z > 2.5$, the floor may be assumed to be infinitely stiff regardless of the number and arrangement of intermediate shear walls.

When the value of $z < 2.0$, the floor slabs can no longer be taken as infinitely stiff. In this case, the load carried by the end walls is less than the value which the ratio of flexural rigidities, K_1/K, would suggest, and the intermediate shear walls are additionally loaded. If the value of $z < 2.0$, then the lateral load attributed to shear walls must be calculated using equation (19).

The value of the ratio p_1/p deviates from the value of the ratio K_1/K for lower values of α. This means that when $z < 2.0$, the greater the number of intermediate shear walls, the greater will be the error introduced by assuming the floor as

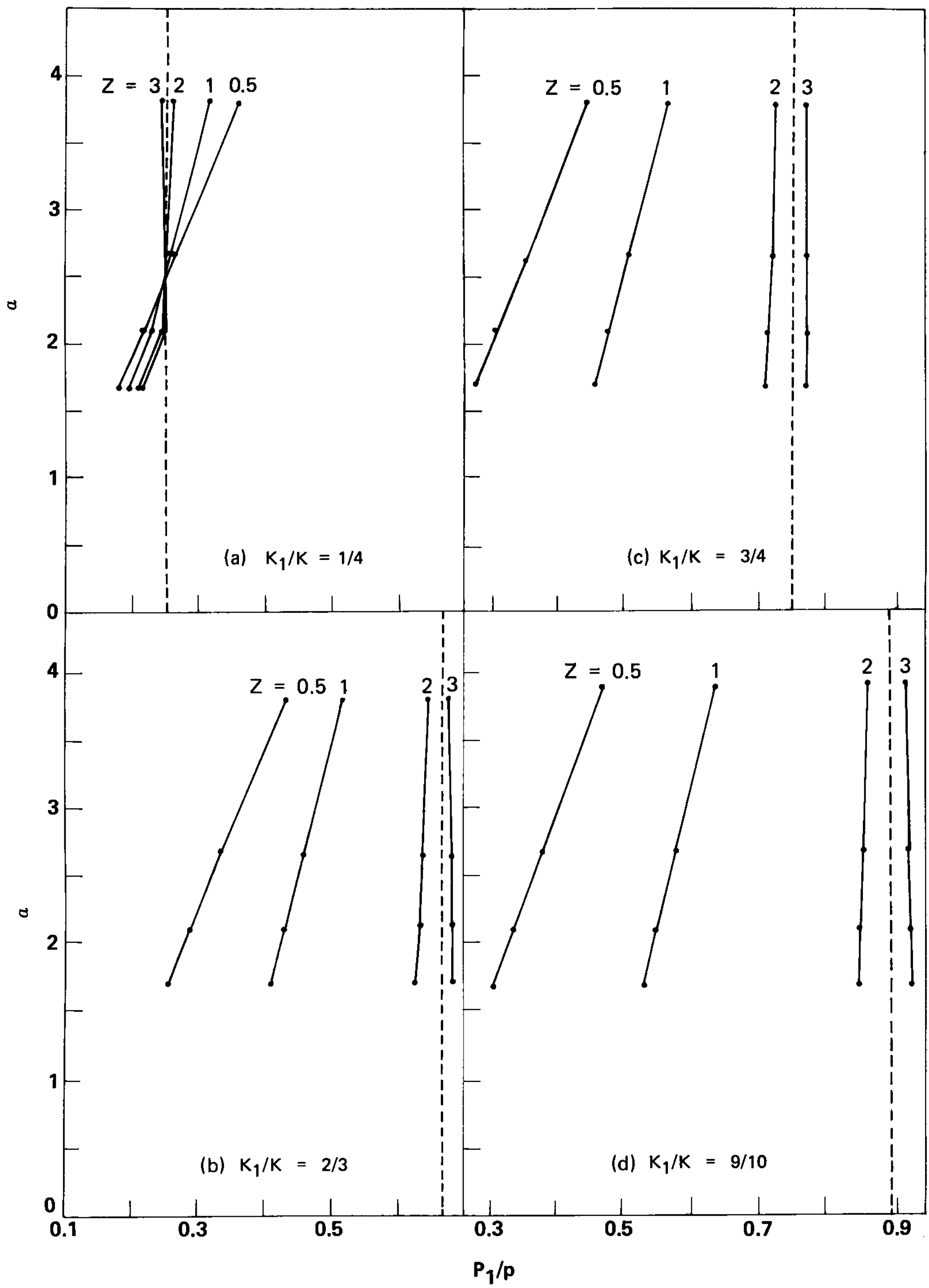

Figure 4 The Variation of (p_1/p) as a function of (K_1/K)

infinitely stiff.

The relative rigidities of end and interior shear walls have a pronounced effect on the partitioning of lateral load, especially when $z < 2.0$. At large values of K_1/K, which means end shear walls stiffer than the interior shear walls, the end walls unload a good part of their share of the lateral load to the interior walls. A typical plot of this phenomenon is shown in Figure 5, for a chosen value of $z = 1.0$. In Figure 5, for the infinitely stiff floor action, all the curves must coincide with the 45^{o} line which would indicate a one to one correspondence of K_1/K and p_1/p ratios. As suggested by Figure 5, the best choice of K_1/K value, resulting in behavior closest to that of an infinitely stiff floor, would be smaller values such as 0.25-0.40.

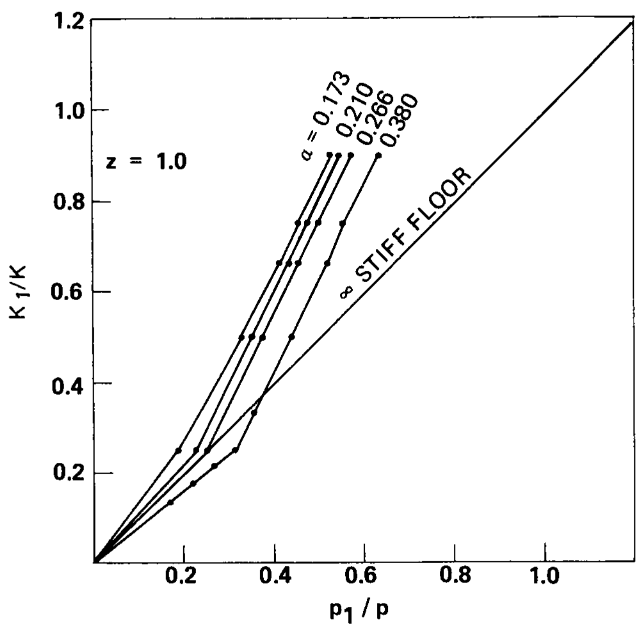

Figure 5 The Effect of End Wall Rigidities on the Partitioning of the Lateral Load

Since the values of z to enable a tall building to behave as having infinitely stiff floors are $z > 2.5$, see Figure 4, the required slab stiffness (or the thickness) can be calculated for a structure height of $x = H$ as:

$$z = \frac{H}{c} = \frac{H}{2\left[\dfrac{\sum_{i=2}^{n-1} K_i a_i^2}{SL^2\ \{K/K_1\}}\right]^{1/4}} \tag{20}$$

$$S = \frac{48K_f}{hL^3} \tag{21}$$

$$K_f = \frac{z^4 hL \{K_1/K\}}{3.0\ H^4} \sum_{i=2}^{n-1} K_i a_i^2 \qquad (22)$$

If the layout of the plan and height of a structure are known, the required slab stiffness can be calculated from equation (22). From a designer's point of view, the value of c and the variation of the influencing parameters reflect the desired layout of the structure.

It is evident from equation (22) that as the height of the structure increases, z also increases in direct proportion, thus making the structure behave as if it had infinitely stiff floors, reducing the importance of the floor thickness. A designer may want to know the limiting height of a tall building, after which he should not worry if his slab is sufficiently stiff to justify his assumption of an infinitely stiff floor while partitioning the lateral load to the shear walls. To answer the question, a structure with quite favourable geometric properties can be considered as follows:

Structure. A multi-storey structure with end shear walls of rigidity K_1 and one intermediate shear wall of rigidity K_2. $(K = K_1 + K_2)$.

$\frac{K_1}{K} = 0.9$

Slab length = 40000 mm
Slab width = 6000 mm
Slab thickness = 80 mm
Storey height = 3000 mm
Wall thickness = 250 mm
Chosen z = 3.0

Solving equation (22) for H, equation (23) is obtained, from which the limiting structure height, H, is calculated.

$$H = \left[\frac{z^4 hL\{K_1/K\}}{3.0\ K_f} \sum_{i=2}^{n-1} K_i a_i^2\right]^{1/4} \qquad (23)$$

For the above data the value of H = 27900 mm, the limiting value of H thus corresponding to about 10 stories. So, a general but crude conclusion can be drawn that if a structure is greater than 10 stories high, its floors can be assumed to be infinitely stiff. However, special structures, although higher than 10 stories, should always receive special consideration.

CONCLUSIONS

The following conclusions have been reached as a result of this study:

When the parameter $z > 2.5$, the floor slab can be considered as infinitely stiff, and the lateral load can be partitioned between the shear walls in direct proportion to their flexural rigidities.

For structures which are greater than ten stories high, the effect of floor flexibility is minimized.

When $z < 2.0$, the ratio of the rigidities of the end shear walls to the total number of shear walls (K_1/K) plays an important role in the partitioning of the lateral load among the shear walls. As K_1/K becomes greater than 0.4, errors introduced by assuming an infinitely stiff floor become very important.

For the same slab length, L, the number and distribution of intermediate shear walls have a minor influence on lateral load partitioning for a given K_1/K ratio.

If an infinitely stiff floor is assumed in design, the necessary slab thickness can be calculated using equation (22).

If the value of the parameter z is less than 2.0 and cannot be changed, the lateral load which goes to end and intermediate shear walls can be calculated from equation (19).

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APPENDIX: DESIGN EXAMPLE

An eight storey structure with elevation and floor plan as shown in Figure 6 has the following data:

Slab length	40000 mm
Slab width	6000 mm
Storey height	3000 mm
Wall thickness	250 mm
All columns	500 x 500 mm
All beams	400 x 750 mm
Concrete strength	25 N/mm^2

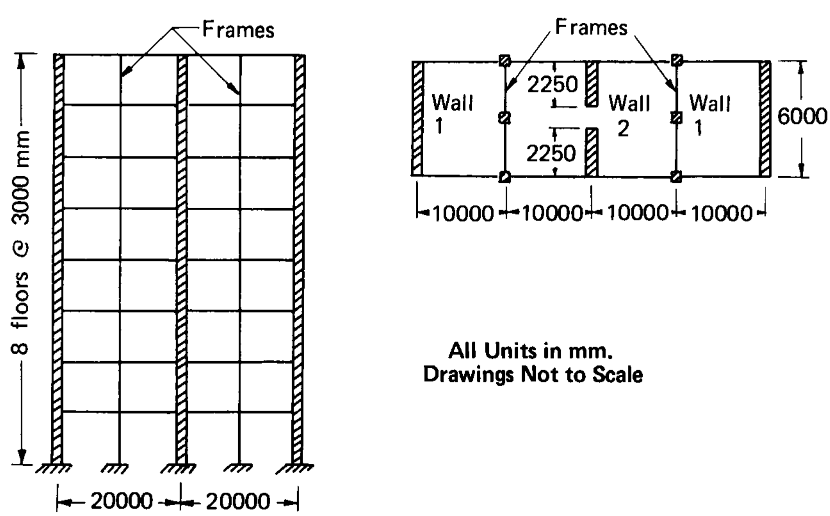

Figure 6 Elevation and Plan Views of the Example Structure

Modulus of elasticity (CEB-75) : $E_{cj} \simeq 30$ kN/mm^2

Flexural rigidities of walls :

$$\text{Wall 1 : } EI_1 = K_1 = (30)\ (\frac{1}{12} \times 250 \times 6000^3)2 = 0.27 \times 10^{15} \text{ kN-mm}^2$$

$$\text{Wall 2 : } EI_2 = K_2 = (30)\ (\frac{1}{12} \times 250 \times 2250^3)2 = 0.14 \times 10^{14} \text{ kN-mm}^2$$

Neglecting the rigidities of the frame elements

$$K = (2.7 + 0.14) \times 10^{14} = 2.84 \times 10^{14} \text{ kN-mm}^2$$

$$\frac{K_1}{K} = \frac{2.7}{2.84} = 0.95$$

$$\sum_{i=2}^{n-1} K_i a_i^2 = 0.14 \times 10^{14} (20000)^2 = 0.56 \times 10^{22} \text{ kN-mm}^2$$

Choose $z = 3.0$

Necessary slab stiffness, K_f

$$K_f = \frac{(3)^4}{3.0} (3000)(40000)(0.95)(0.56 \times 10^{22}) \frac{1}{(24000)^4} = 5.195 \times 10^{13} \text{ kN/mm}^2$$

$$(30)\ \frac{1}{12}\ (t)(B^3) = 5.195 \times 10^{13}$$

$$t = 5.195 \times 10^{13}\ (12)\ \frac{1}{(6000)^3}\ \frac{1}{(30)} = 96.2 \text{ mm}$$

In order to be able to assume the floor slab as infinitely stiff, the slab thickness must be at least 100.0 mm.

RECENT DEVELOPMENTS IN APPORTIONING REINFORCEMENT IN CONCRETE SLABS SUBJECTED TO BENDING AND MEMBRANE FORCES

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ABSTRACT Previous analytical approaches for the determination of reinforcement to resist membrane forces in slabs are examined and an alternative graphical procedure is presented. It is shown how this latter approach can be used to achieve solutions other than those employing point optimisation. The membrane solutions are then applied to the problem of determining reinforcement to resist a combination of bending and membrane forces by consideration of sandwich models. Sections designed by such an approach are then examined with regard to serviceability criteria specified in the current codes of practice. Examples illustrating these points are also presented.

INTRODUCTION

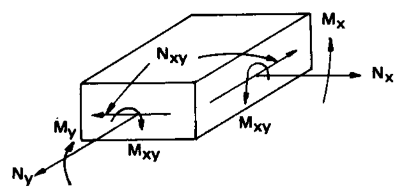
In the design of concrete structures such as slabs, box-girders or shells, the basic distribution of forces within the structures, for given external loading at the ultimate limit state, is usually determined by elastic analysis. The problem then arises of how to reinforce each element of the structure to resist the moments and forces on it. Often there will be practical constraints on the directions in which reinforcement may be placed, or on the proportion of steel allowed, but in many cases it is worthwhile to try to minimise the total amount of reinforcement in given orthogonal or skew directions within the slab.

Slab elements subject to membrane forces only have been examined by Nielsen (1) who gives simple explicit equations for optimum tensile steel proportions. Clark (2) has extended this approach to cover the possibility that compression steel may be required. For slab elements subject to moments only, Morley (3), Wood (4) and Armer (5) present equations of varying complexity for both orthogonal and skew steel proportions. These approaches consider both cases independently, but invariably combinations of moment and membrane forces act together on structural elements and a computerised sandwich procedure for determining the reinforcement for this case has been presented by Morley and Gulvanessian (6).

This paper presents an alternative graphical approach for the determination of reinforcement to resist membrane forces which will allow the designer more choice in the determination of the reinforcement parameters. Finally the ultimate and serviceability aspects of a simplified sandwich approach for combined moment and membrane loading of a slab element are considered.

NOTATION

N_x, N_y, N_{xy}, M_x, M_y, M_{xy} — Membrane forces and bending moments per unit length of slab, as shown below:



$$n_x = \frac{N_x}{f_c D}; \quad n_y = \frac{N_y}{f_c D}; \quad n_{xy} = \frac{N_{xy}}{f_c D};$$

$$a_x = \frac{A_x \sigma_Y}{f_c D}; \quad a_y = \frac{A_y \sigma_Y}{f_c D}$$

f_c, σ_Y yield stress of concrete and steel respectively
σ, θ concrete principal stress and principal angle
D slab thickness
c cover to centre of reinforcement layers
A_x, A_y area of reinforcement per unit width of slab
ε strain in concrete at required depth
ρ concrete strength factor
$K = |\tan\theta|$

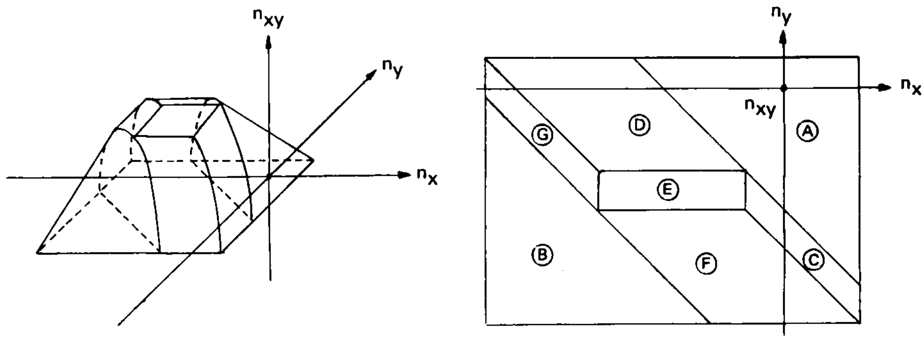
REINFORCEMENT OF SLABS FOR MEMBRANE FORCES ONLY

The sandwich method for determining the reinforcement to resist a combination of membrane (N_x, N_y, N_{xy}) and bending forces (M_x, M_y, M_{xy}), described in a later section, requires a procedure for determination of the reinforcement to resist membrane forces in a slab. Nielsen (1) and Clark (2) have presented equations based upon lower bound plasticity theory with Clark allowing for the possibility that compression reinforcement may be required. Clark's equations must be used in conjunction with a 'case boundary graph' which identifies the relevant set of equations to be used for a particular stress triad. Apparently this graph must be constructed for different values of concrete strength and thus knowledge of the design concrete strength is required before the graph can be constructed.

Nielsen's approach, which considers only tensile reinforcement, differs in that it does not use any auxiliary charts, since the concrete strength is determined directly from his equations, see Appendix 1. This means that for every point under consideration Nielsen is effectively defining a yield criterion enclosing or passing through the point, such that the tension reinforcement regions are applicable. Thus for points for which Clark's equations indicate compression reinforcement, Nielsen's equations require either concrete of a higher strength or a thicker section.

Graphical Procedure

A graphical procedure can be derived to determine the reinforcement parameters corresponding to Clark's approach. The origin of this procedure is Nielsen's orthotropic yield criterion (1), see Figure 1, the derivation of which is the inverse problem of the provision of reinforcement. Thus by definition of a suitable transformation of variables a 'reinforcement' criterion can be obtained. Part of such a reinforcement criterion, when plotted as a surface, was first presented by Braestrup (7) who considered pure shear (n_{xy}) in the webs of reinforced concrete T-beams. In extending Braestrup's idea to any combination of tensile membrane forces, the influence of the additional direct forces is to move the surface obtained for pure shear around a fixed global co-ordinate system, a_x, a_y, $|n_{xy}|$, and not to alter the basic relationship between the shear force and reinforcement parameters, see Figure 2.



(a) General View of Yield Surface (b) Projection of Yield Surface on Plane $n_{xy}=0$

Figure 1 Yield Criterion for Orthogonally Reinforced Panel Elements

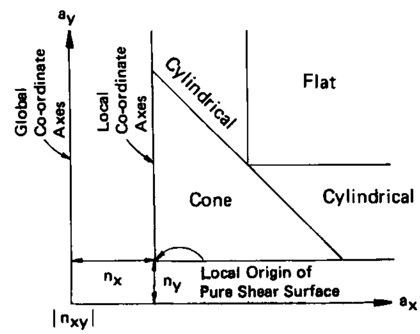
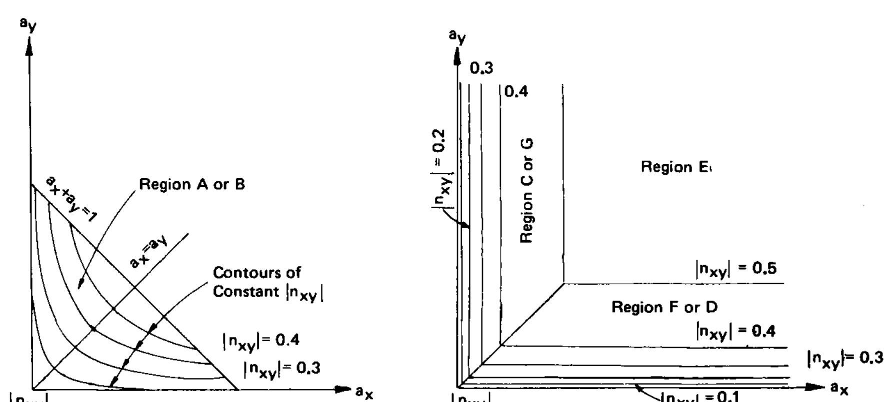


Figure 2 Relation between Shear Forces and Reinforcement Parameters

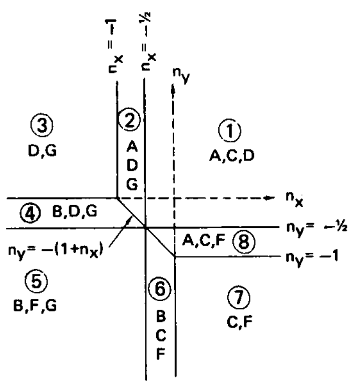
Further investigation showed that the general surface for any combination of membrane forces could be constructed from two basic surfaces; one being a cone, the other being composed of two cylindrical surfaces at right angles, Figure 3.



(a) Cone (b) Cylindrical

Figure 3 Basic Surfaces

These basic surfaces are the parts of the orthotropic yield criterion designated A, C and D in Figure 1. Should the construction require both surfaces to overlap, the cone surface takes precedence and is placed over the cylindrical surface. This is illustrated in Appendix 2. The detailed mathematical derivation of this approach is contained in unpublished reports of Building Research Station. The mathematically constructed reinforcement criterion now requires some means of indicating the correct physical conditions which are applicable to the constituent regions. This is achieved by construction of an applicability diagram, see Figure 4, similar in concept to Clark's case boundary graphs but much simpler in appearance and, more importantly, requiring constructing only once. Figure 4 also shows the positions in the global co-ordinate axes at which the origins of the basic surfaces should be placed to construct the reinforcement criterion. The steps in the use of the graphical procedure are listed below and an example of this procedure is contained in Appendix 2.



Legend:-

- Y_T - Tensile yield stress
- Y_c - Compressive yield stress
- σ_E - Elastic range, $Y_c \leqslant \sigma_E \leqslant Y_T$
- f_c - Concrete crushing stress
- σ_1 σ_2 - Numeric values of the concrete principal stresses

Co-Ordinates for Positioning Basic Surfaces

ZONE	SURFACE	CO-ORDINATES OF GLOBAL POSITION OF LOCAL ORIGIN
1	cone cylindrical	$\left\{\begin{array}{l} a_x = n_x \\ a_y = n_y \end{array}\right.$
2	cone cylindrical	$a_x = n_x;\ a_y = n_y$ $a_x = -(1 + n_x);\ a_y = n_y$
3	cylindrical	$a_x = -(1 + n_x);\ a_y = n_y$
4	cone cylindrical	$a_x = -(1 + n_x);\ a_y = -(1 + n_y)$ $a_x = -(1 + n_x);\ a_y = n_y$
5	cone cylindrical	$\left\{\begin{array}{l} a_x = -(1 + n_x) \\ a_y = -(1 + n_y) \end{array}\right.$
6	cone cylindrical	$a_x = -(1 + n_x);\ a_y = -(1 + n_y)$ $a_x = n_x;\ a_y = -(1 + n_y)$
7	cylindrical	$a_x = n_x;\ a_y = -(1 + n_y)$
8	cone cylindrical	$a_x = n_x;\ a_y = n_y$ $a_x = n_x;\ a_y = -(1 + n_y)$

Physical Conditions Relevant to Each Region of the Orthotropic Yield Surface

REGION	X-STEEL	Y-STEEL	CONCRETE	
			σ_1	σ_2
A	Y_T	Y_T	0	$0 < \sigma_2 < f_c$
B	Y_c	Y_c	$0 < \sigma_1 < f_c$	f_c
C	Y_T	σ_E	0	f_c
D	σ_E	Y_T	0	f_c
E	σ_E	σ_E	0	f_c
F	σ_E	Y_c	0	f_c
G	Y_c	σ_E	0	f_c

Figure 4 Applicability Conditions

Step 1. Select f_c and hence obtain n_x, n_y and $|n_{xy}|$ from analysis.
Step 2. From Figure 4 check the location of (n_x, n_y); note the zone number; calculate the global co-ordinates at which the origins of the basic surfaces are to be placed.
Step 3. Position the basic surfaces, cylindrical before cone if both are required; the reinforcement criterion is the resulting surface lying in the positive a_x, a_y quadrant.
Step 4. Locate the relevant $|n_{xy}|$ contour and as any point on this contour represents a valid solution, read off the chosen values of a_x and a_y.
Step 5. From Figure 4 note the correct physical interpretation to these values.

Quantification of Different Concrete Strengths

The graphical procedure can be used to quantify the differences in concrete strength required in certain cases by the approaches of Nielsen and Clark. The question to be answered is by how much does the concrete strength need to be increased to obviate the need for compression reinforcement? Figure 4 shows that the only zones in which tensile reinforcement solutions are possible are 1, 2 or 8. If ρ is defined to be the factor by which the original concrete strength must be multiplied, then from the definition of n_x and n_y, it can be seen that the effect of ρ is to move points towards the positive n_x, n_y quadrant in Figure 4, ie attracting points towards the zones 1, 2 and 8. Thus the required value of ρ is that which will just move the point into one of these zones in which the tensile reinforcement equations apply. The analytical expressions deduced from the graphical approach are presented in Appendix 2 together with a numerical calculation.

Reinforcement Parameter Adjustment - Variable K

The statement in Step 4 of the graphical procedure is due to the singularity of the equilibrium equations corresponding to the cone regions of the yield criterion. Nielsen and Clark remove this singularity by considering the minimisation of the reinforcement at a point. However by not imposing this condition, it is possible to adjust the reinforcement parameters to achieve alternative solutions so that possibly patch minimisation may be attempted.

Utilising the basic cone equation, $a_x a_y = n_{xy}^2$, obtained by eliminating K from the two equilibrium equations:

$$a_x = K|n_{xy}|; \quad a_y = |n_{xy}|/K$$

it can be seen that contours of constant $|n_{xy}|$ are rectangular hyperbolae and that lines from the origin have a gradient of $1/K^2$, see Figure 5. The point optimisation solution corresponds to K = 1 and in Figure 5 is represented by the point at the intersection of the line at $\pi/4$ to the axes and the $|n_{xy}|$ contour under consideration. If one moves away from this point along the contour, then it can be seen that one of the reinforcement parameters can be reduced but only at the expense of possibly a disproportionate increase in the orthogonal parameter. This facility will be useful at points which require a high localised value of reinforcement in one direction, as predicted by the point optimisation solution. While the general variable K solution is a complex non-linear mathematical programming problem, it is possible to achieve a solution by trial and error as in the example below.

Consider the provision of uniform reinforcement to resist the given forces at the following four points: A (n_x = 0.6, n_y = 0.167, n_{xy} = 0.333); B (0.667, 0.1,

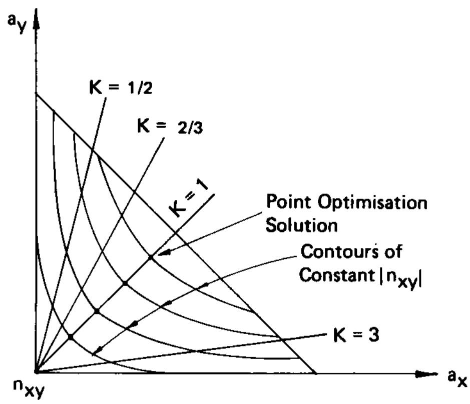


Figure 5 Variable K Representation

0.333); C (0.333, 0.267, 0.167); D (0.1, 0.4, 0.5). The point optimisation solution will be:

Point	n_x	n_y	$\|n_{xy}\|$	$a_x = n_x + \|n_{xy}\|$	$a_y = n_y + \|n_{xy}\|$
A	0.6	0.167	0.333	0.933	0.5
B	0.667	0.1	0.333	1.0	0.433
C	0.333	0.267	0.167	0.5	0.433
D	0.1	0.4	0.5	0.6	0.9

From the above, one might provide $a_x = 1.0$ and $a_y = 0.9$ over the area giving a total reinforcement sum of 1.9.

The results of a trial and error variable K procedure are tabulated below with a_x and a_y assumed to be 0.85 and 0.733 respectively:

	Solution insisting on a_x value			Solution insisting on a_y value		
Point	K	a_x	a_y	K	a_x	a_y
A	0.75	0.85	0.611	0.588	0.796	0.733
B	0.55	0.85	0.706	0.526	0.842	0.733
C	3.1	0.85	0.320	0.357	0.393	0.733
D	1.5	0.85	0.733	1.5	0.85	0.733

Note: $a_x = n_x + K|n_{xy}|$, $a_y = n_y + \frac{1}{K}|n_{xy}|$

Thus, as no conditions are violated at any of the points, the above values represent a valid solution, giving a total reinforcement sum of 1.583. Thus, the point optimisation solution requires a 20 per cent increase in the total reinforcement compared with the variable K solution. This example is merely illustrative, however, and in practice smaller savings should result.

REINFORCEMENT OF SLABS FOR COMBINED BENDING AND MEMBRANE FORCES

For cases when a slab element has to resist a combination of membrane and bending forces acting at the mid-depth of the slab, see Figure 6(c), the sandwich method, briefly described below, may be used.

The sandwich model is shown in Figure 6, where the slab element is regarded as two reinforced outer layers separated by an unreinforced concrete filling. The thickness of each of the outer layers consists of twice the cover to the main reinforcement plus the diameters of the bars, Figure 6(b). The forces in the outer layers are assumed to act at the mid-depths of the layers. Transferring the forces acting at the mid-depth of the slab to the mid-depth of each layer, the following expressions are obtained.

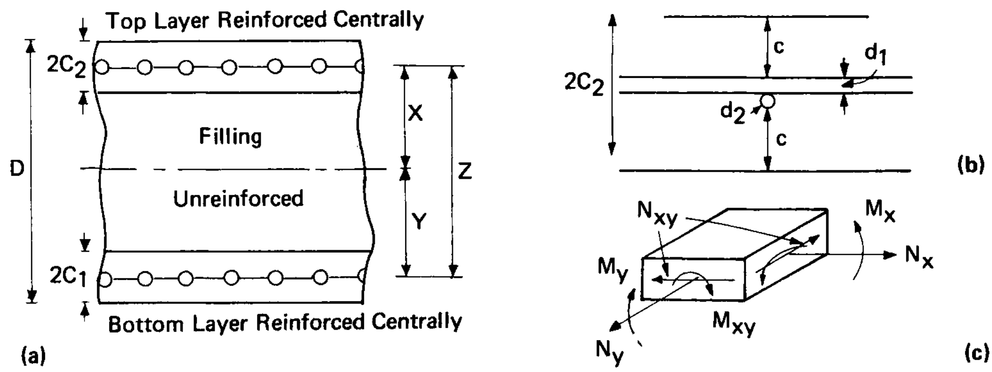


Figure 6 Slab Parameter Definitions

For the top layer:

$$N_{xt} = \frac{N_x Y - M_x}{Z}, \qquad N_{yt} = \frac{N_y Y - M_y}{Z}, \qquad N_{xyt} = \frac{N_{xy} Y - M_{xy}}{Z}$$

and for the bottom layer:

$$N_{xb} = \frac{N_x X + M_x}{Z}, \qquad N_{yb} = \frac{N_y X + M_y}{Z}, \qquad N_{xyb} = \frac{N_{xy} X + M_{xy}}{Z}$$

The procedure for obtaining the required reinforcement using the above equations is as follows:

a. Assume $X = Y$ and $2C_1 = 2C_2 = 2C$. A recommended value for 2C is twice the cover to the main bar plus 40mm.
b. Apply any of the membrane reinforcement procedures, already discussed, to the outer layers.
c. Preliminary values of A_x and A_y are now obtained for both outer layers and the actual values for C_1, C_2, X and Y are determined.
d. Steps b and c are repeated. One iteration is usually sufficient.

The sandwich method has a limitation, which is more likely to occur in thick slabs, where the cover is a small proportion of the total depth. Here the compressive strength of the concrete in the outer layers may be insufficient to resist the applied forces and moments unaided. In such cases it is necessary to make use of the compressive strength of the concrete in the filling. A method of carrying out these calculations using Nielsen's equations is described below.

The case considered here requires reinforcement in the y-direction only, i.e. $N_{xt} \leqslant N_{yt}$, $N_{xt} < |N_{xyt}|$ and $N_{yt} > |N_{xyt}|$, and Nielsen's equations for this case are:

$$A_{xt}\,\sigma_Y = 0$$

$$A_{yt}\,\sigma_Y = N_{yt} + \left|\frac{N_{xyt}^2}{N_{xt}}\right|$$

$$\sigma_t\,2C_2 = N_{xt}\left[1 + \left[\frac{N_{xyt}}{N_{xt}}\right]^2\right]$$

where the suffix t refers to top outer layer.

Providing $\sigma_t \leqslant f_c$, the outer layer can resist the applied forces unaided. However, if $\sigma_t > f_c$, it will be necessary to make use of the compressive strength of the concrete in the filling as follows.

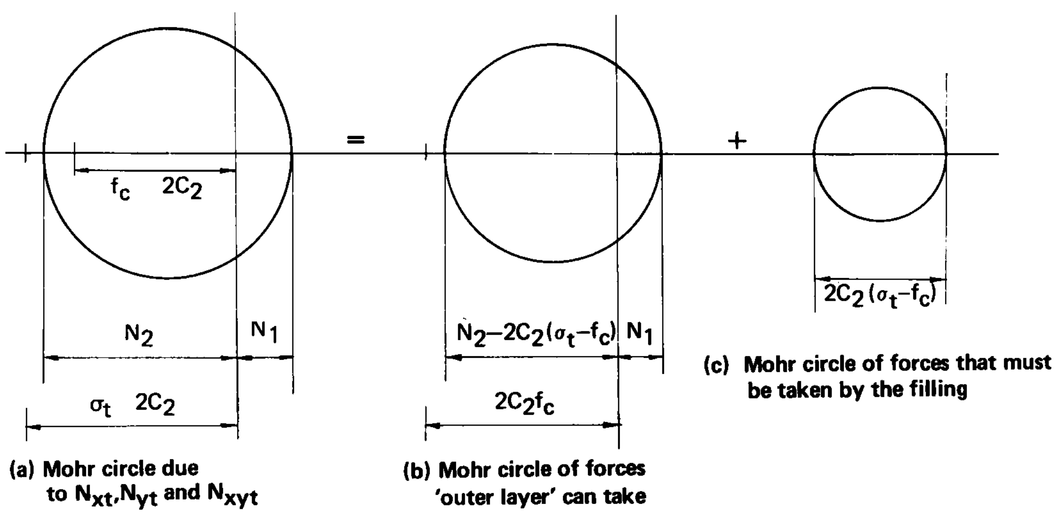


Figure 7 Mohr's Circle Representation of Stresses in Slabs

The excess compressive force is $2C_2\,(\sigma_t - f_c)$. Figure 7(a) shows the Mohr circle due to N_{xt}, N_{yt} and N_{xyt}. Figure 7(b) is the Mohr circle for the forces which the outer layer can resist and Figure 7(c) is the Mohr circle of forces that must be taken by the unreinforced filling. From Figure 7(b), the equations for principal stress are:

$$N_2 - 2C_2\ (\sigma_t - f_c) = \frac{N_x^t + N_y^t}{2} - \left[\left[\frac{N_x^t - N_y^t}{2}\right]^2 + (N_{xy}^t)^2\right]^{\frac{1}{2}}$$

$$N_1 = \frac{N_x^t + N_y^t}{2} + \left[\left[\frac{N_x^t - N_y^t}{2}\right]^2 + (N_{xy}^t)^2\right]^{\frac{1}{2}}$$

and from Nielsen's equations:

$$f_c\ 2C_2 = N_x^t\ (1 + \left[\frac{N_{xy}^t}{N_{xt}}\right]^2)$$

where the superscript t indicates the maximum forces which the outer layer is capable of resisting. In the above three equations N_1 and N_2 are obtained using Figure 7(a). Solution of the above equations yields the remaining unknowns N_x^t, N_y^t and N_{xy}^t. Other cases can be treated similarly.

Considerations of Both Layers Simultaneously

The lower layer is also examined for all cases and the forces the filling is required to take are determined by the following expressions:

$$N_x^f = N_x - N_x^t - N_x^b \qquad M_x^f = M_x - N_x^b\ Y + N_x^t\ X$$

$$N_y^f = N_y - N_y^t - N_y^b \qquad M_y^f = M_y - N_y^b\ Y + N_y^t\ X$$

$$N_{xy}^f = N_{xy} - N_{xy}^t - N_{xy}^b \qquad M_{xy}^f = M_{xy} - N_{xy}^b\ Y + N_{xy}^t\ X$$

So far it has been assumed that these forces and moments can be transferred from the outer layer to the filling. It is necessary to verify that the filling has adequate strength.

SERVICEABILITY CONSIDERATIONS

The methods discussed so far determine the reinforcement with reference to the ultimate limit state. However, the serviceability limit state remains to be considered. To date, little guidance is available for determining crack widths and, hence, spacing of the reinforcing bars for slabs resisting combined loads. A method is shown below.

Determination of Spacing of the Bars

The rules governing crack control are contained within CP110 (8) and BS 5400 (9). In application of the relevant crack width formulae the principal unknown is ε_f.

Consider a slab subject to bending and membrane forces. By applying the sandwich method described earlier, N_{xt}^*, N_{yt}^*, N_{xb}^* and N_{yb}^* can be obtained using serviceability load factors. Figure 8 shows the possible combinations giving rise to either tension or compression at the slab faces. One combination, case b, is considered below. Referring to Figure 9, it is seen that once the depth of the neutral axis is determined ε_f is easily obtained, and nd can be obtained by equilibrium conditions.

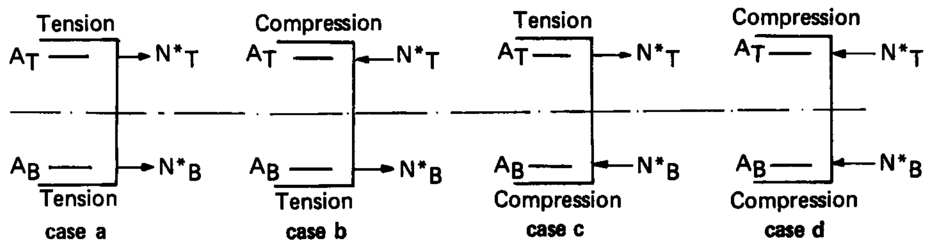


Figure 8 Force Combinations

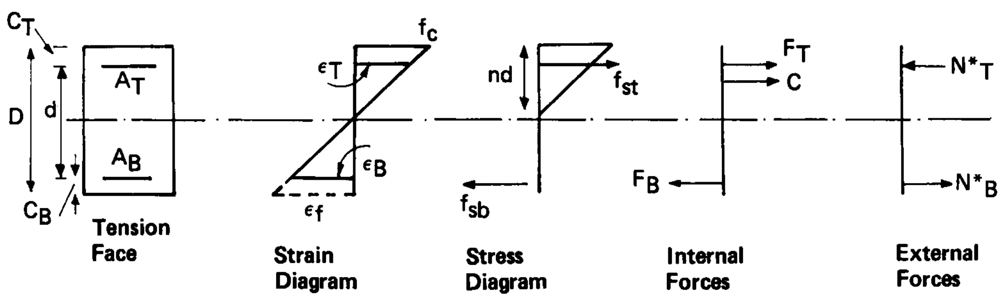


Figure 9 Stress-Strain Diagram for Case b of Figure 8

Thus
$$N^*_T - N^*_B = F_T + C - F_B$$

and
$$N^*_T\ (d - c_B) - C\ (d + c_T - c_B - \frac{nd}{3}) - F_T\ (d - c_B) = 0$$

where
$$C = \frac{nd}{2}\, f_c\,,$$
$$f_{st} = (\frac{nd - c_T}{nd})\ (m - 1)\ f_c,$$

and
$$f_{sb} = \frac{n - 1}{n}\ m\ f_c$$

Solution of the above equations enables n to be determined, f_{st} and f_{sb} are obtained from the expressions:

$$\varepsilon_T = \frac{f_{st}}{E} \quad \text{and} \quad \varepsilon_B = \frac{f_{sb}}{E}\,,$$

and by examining the strain diagram in Figure 9:

$$\varepsilon_f = (\frac{D - nd}{d - nd})\ \frac{f_{sb}}{E}$$

Therefore, by substituting the values obtained above into the relevant crack control equations the spacing of the bars is determined. Similar treatment can be applied to the remaining three cases.

CONCLUSIONS

The approaches of Nielsen and Clark to the determination of reinforcement parameters to resist membrane forces offer a choice of solutions dependent upon the design philosophy chosen and the freedom to vary either the concrete strength or thickness.

A graphical procedure has been outlined for determination of the reinforcement parameters to resist membrane forces which, with suitable programming, could be used as a basis for an interactive graphical determination of reinforcement. This procedure allows the variable K concept to be more fully utilised since, at present, there are no computer programs utilising this approach.

A sandwich method for determination of reinforcement parameters to resist a combination of membrane forces and moments is presented which is suitable for manual or desk top calculation.

A procedure has been outlined to determine the spacing of reinforcement bars to meet the serviceability criteria as specified in the current codes of practice.

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The views expressed in this paper are not necessarily those of the Departments of Transport and the Environment.

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APPENDIX 1 NIELSEN'S EQUATIONS

for $N_x \leqslant N_y$

Case 1. $N_x \geqslant - |N_{xy}|$

$$A_x\sigma_Y = N_x + |N_{xy}|; \quad A_y\sigma_Y = N_y + |N_{xy}|; \quad \sigma D = -2\,|N_{xy}|$$

Case 2. $N_x < - |N_{xy}|$, if $N_y < 0$ then reinforcement required for

$N_xN_y \leqslant N_{xy}^2$ given by

$$A_x = 0; \quad A_y\sigma_Y = N_y + \left|\frac{N_{xy}^2}{N_x}\right|; \quad \sigma D = N_x\,(1 + \left[\frac{N_{xy}}{N_x}\right]^2)$$

Nielsen's equations allowing for variable K

$$A_x\sigma_Y = N_x + K\,|N_{xy}|; \quad A_y\sigma_Y = N_y + \frac{1}{K}\,|N_{xy}|; \quad \sigma D = |N_{xy}|\,(K + \frac{1}{K})$$

APPENDIX 2 ANALYTICAL EXPRESSIONS AND ILLUSTRATIVE EXAMPLE OF GRAPHICAL PROCEDURE

Zone	Analytical Expression
1	$\rho = 1$, i.e. no compression reinforcement
2	for $\sqrt{\{-n_x\,(1 + n_x)\}} \leqslant |n_{xy}| \leqslant \frac{1}{2}$ $\quad \rho \geqslant -[n_{xy}^2 + n_x^2]/n_x$
3	$\rho \geqslant -[n_{xy}^2 + n_x^2]/n_x$
4, 5, 6	$\rho \geqslant -\frac{1}{2}\,[n_x + n_y \pm \sqrt{\{(n_x - n_y)^2 + 4n_{xy}^2\}}]$
7	$\rho \geqslant -[n_{xy}^2 + n_y^2]/n_y$
8	for $\sqrt{\{-n_y\,(1 - n_y)\}} \leqslant |n_{xy}| \leqslant \frac{1}{2}$ $\quad \rho \geqslant -[n_{xy}^2 + n_y^2]/n_y$

Step 1. Assume f_c chosen such that $(n_x, n_y, n_{xy}) = (-0.9, 0.2, 0.4)$.
Step 2. From Figure 4, $(n_x, n_y) = (-0.9, 0.2)$ is in Zone 2 and the co-ordinates are: Cylindrical $a_x = -0.1$; $a_y = 0.2$; Cone $a_x = -0.9$; $a_y = 0.2$.
Step 3. Position surfaces, cylindrical followed by cone, as in Figures A1 and A2 respectively.

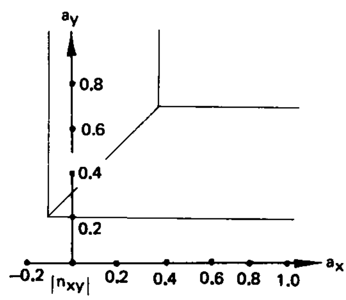

Figure A1

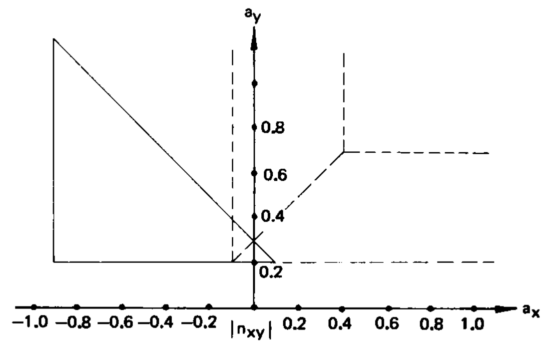

Figure A2

The resulting surface is shown in Figure A3.

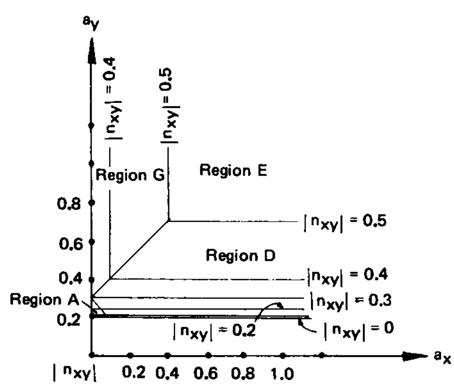

Figure A3

Step 4. Locate the $|n_{xy}| = 0.4$ contour from which $a_x = 0.1$, $a_y = 0.4$ is a solution

Step 5. From Figure 4 the x-steel is yielding in compression and the y-steel yielding in tension

Note: Had $|n_{xy}| < 0.3$ then from Figure A3 a solution lying in the cone surface would have been possible.

If a solution without compression reinforcement is required, then if $n_x = -0.9$ and $n_{xy} - 0.4$, $\rho = 1.067$. Thus, the required concrete strength would be 1.067 times the original value. This can be checked by repeating the above steps with the new values of $(n_x, n_y, n_{xy}) = (-0.843, 0.187, 0.375)$.

STRUCTURAL BEHAVIOUR OF FLOOR SLABS IN SHEAR WALL BUILDINGS

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ABSTRACT The effective stiffness of a floor slab connecting shear walls in laterally loaded tall buildings is determined by the finite element method. The influence of the shape, dimensions and spacing of the walls on the effective width is examined, and a series of design curves is presented to enable the slab stiffness to be assessed rapidly for different wall configurations.

INTRODUCTION

A common form of construction for multi-storey residential buildings consists of assemblies of shear walls and floor slabs, in which the coupling of the cross-walls by the floor slabs results in a more efficient structural system for resisting lateral forces. Figure 1(a) shows a typical (idealised) floor plan of a slab block in which self-contained apartment units are arranged side by side along the length of the building. This arrangement naturally results in parallel assemblies of division walls running perpendicular to the face of the building, with intersecting longitudinal walls along the corridor and facade enclosing the living spaces. The cross-walls are employed as load bearing walls, in addition to serving architectural requirements, since their disposition favours an efficient distribution of both gravity and lateral loads to the structural elements. The longitudinal corridor and facade walls are provided with openings for access to the living areas and balconies, and for window framing. If they are also designed to be load-bearing, these longitudinal walls act effectively as flanges for the primary cross-walls. In addition to the structural partition walls, shear walls are used to enclose lift shafts and stair wells to form the open section box structures which act as strong points in the building. Thus, in practice, shear walls of various shapes, planar, flanged or box-shaped, may be coupled together in cross-wall structures, see Figure 2.

The shear walls resist the lateral loads due to wind or earthquake effects on the structure by cantilever bending action, which results in rotations of the wall cross-sections. The free bending of a pair of shear walls is resisted by the floor slab, which is forced to rotate and bend out of plane where it connects rigidly to the walls, Figure 1(b). Due to the large depth of the wall, considerable differential shearing action is imposed on the connecting slab, which develops transverse reactions to resist the wall deformations, Figure 1(c), and induces tensile and compressive axial forces into the walls. As a result of the large lever arm involved, relatively small axial forces can give rise to

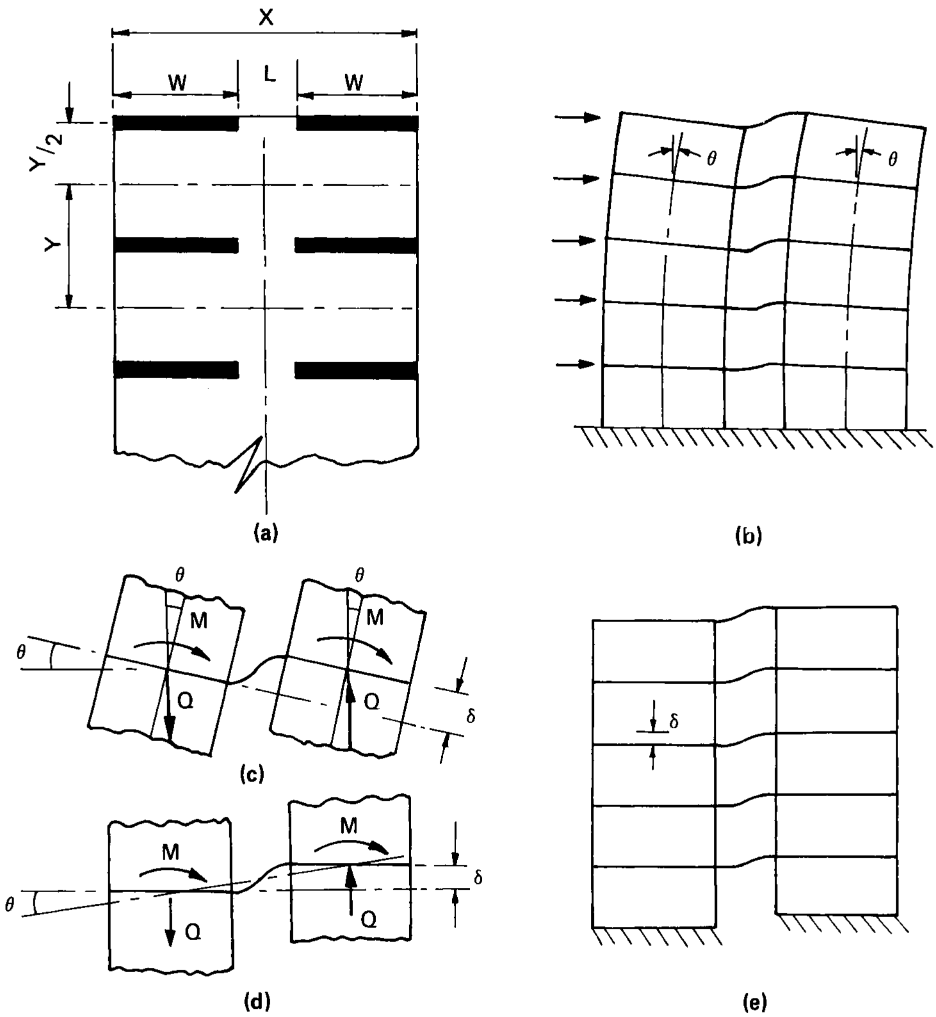

Figure 1 Structural Actions of Coupled Shear Wall-Slab Structure Subjected to Lateral Loads or Differential Movement

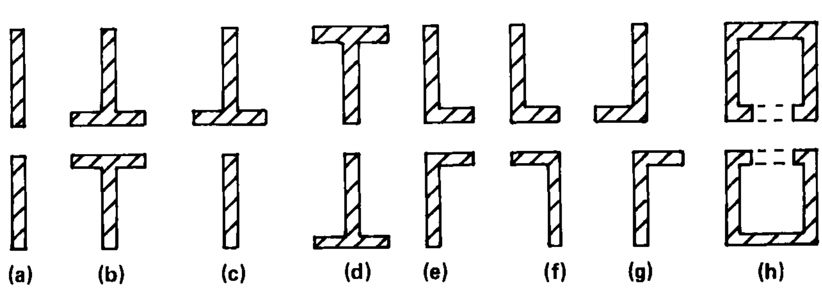

Figure 2 Plan-forms of Coupled Shear Wall Structures

substantial moments of resistance, thereby greatly reducing the wind moments in the walls and the resultant tensile stresses at the windward edges; the lateral stiffness of the structure is also considerably increased. A similar situation arises if relative vertical deformation of the walls occurs, due to unequal vertical loading on the walls or to differential foundation settlement. The effect on the slab is similar to that produced by parallel wall rotation caused by bending, see Figure 1(d) and (e).

The structural analysis and design of a slab-coupled shear wall system may readily be performed using the techniques developed over the past two decades for beam-coupled wall structures, provided that the equivalent width of the slab which acts effectively as a wide coupling beam, or its corresponding structural stiffness, can be assessed. Unlike a beam, however, the coupling stresses are not uniform across the width of a slab, and in order to design the slab safely, the magnitude and distribution of the stresses developed by the coupling action must be known. It is also necessary to determine accurately the interactive shearing forces at the slab-wall junction, particularly in the highly stressed regions near the inner extremities of the walls.

It is only relatively recently that systematic studies have been made of the nature of the interaction between laterally loaded walls and floor slabs, and of the relative importance of the various parameters affecting stresses and deformations. The first investigation of plane walls was carried out by Qadeer and Stafford Smith (1), who used the finite difference technique to solve the plate equation. Similar studies were subsequently made by the same investigators (2) and by Chang (3), whilst the same problem was investigated using the finite element method by Petersson (4) and by Black, Pulmano and Kabaila (5). More recently, the influence of orthogonal walls acting as flanges has been examined experimentally by Coull and El Hag (6) and El Buluk (7) and theoretically using the finite element method by Tso and Mahmoud (8).

The present paper employs the well-established finite element method to determine the effective width and stiffness of a floor slab coupling a range of shear wall structures, with combinations of plane, T-shaped, L-shaped and open box core elements, see Figure 2. The influence of a wide range of structural configurations has been examined, and a series of design curves presented for use in a practical design situation.

EFFECTIVE WIDTH OF FLOOR SLAB

The resistance of the floor slab against the displacements imposed by the shear walls is a measure of its coupling stiffness, which can be defined in terms of the displacements at its ends and the forces producing them. Thus, referring to Figures 1(c) and (d), the stiffness of the slab may be defined either as a rotational stiffness, M/θ, or as a translational stiffness, Q/δ, since the two are related. Due to the non-uniform bending across the width, the force-displacement relationship can only be evaluated from a two-dimensional plate-bending analysis. For convenience, the rotational and translational slab stiffnesses are expressed in the form of non-dimensional stiffness factors K and K_δ given by:

$$K = \frac{M}{\theta}\frac{1}{D} \quad \text{and} \quad K_\delta = \frac{Q}{\delta}\frac{L^2}{D} \tag{1}$$

where D is the flexural rigidity of the plate and L is the clear opening between walls.

For the purpose of the overall analysis, it is convenient to assume that a strip of slab acts effectively as a beam in coupling a pair of walls. The effective stiffness of the slab may then be defined simply in terms of the geometric and material characteristics of the equivalent beam. The effective width of slab can be established by equating the rotational and translational stiffnesses of the slab to those of the equivalent beam, which may be written as:

$$\frac{M}{\theta} = \frac{6EI}{L^3}(L + W)^2 \tag{2}$$

and
$$\frac{Q}{\delta} = \frac{12EI}{L^3} \tag{3}$$

where W is the width of the wall, and I (= $Y_e t^3/12$) is the second moment of area of the beam, of effective width Y_e and thickness t.

The effective width can then be expressed in terms of the rotational and translational stiffness factors, in non-dimensional form, as,

$$\frac{Y_e}{Y} = \frac{K}{6(1 - V^2)}\left(\frac{L}{Y}\right)\left(\frac{L}{L + W}\right)^2 \tag{4}$$

or
$$\frac{Y_e}{Y} = \frac{K_\delta}{12(1 - V^2)}\left(\frac{L}{Y}\right) \tag{5}$$

where Y is the bay width or longitudinal wall spacing, Figure 1(a), and V is Poisson's ratio for the slab material.

FINITE ELEMENT ANALYSIS OF SLAB

The study of the wall-slab interaction involves basically the analysis of the slab actions in resisting the deformations imposed by a pair of shear walls undergoing parallel wall rotation or differential vertical movement. In this study, the finite element method, which is now well documented and widely used, is employed to analyse the slab. For analysis, the slab is assumed to be homogeneous, isotropic and linearly elastic, and plane sections of the wall are assumed to remain plane where the slab interacts with the walls.

In the finite element analysis, the slab panel is discretised into an assembly of plate bending elements using a suitable mesh pattern determined by a convergence study. The mesh is generally graded so that the mesh used in the region close to the wall, where stress gradients are expected to be high, is finer than the mesh for other regions. When conditions of symmetry or anti-symmetry exist with respect to wall configuration or deflection pattern, only the portion of the panel defined by the lines of symmetry or anti-symmetry is analysed in order to reduce the computational effort. The boundary conditions necessary for the solution of the problem are prescribed in terms of known displacements (deflections, slopes or curvatures) at the slab edges, or lines of symmetry and skew-symmetry, and at the wall-slab junction. The displacements prescribed for the wall nodes are due either to a unit wall rotation or to a unit relative vertical wall movement, the slab being subjected to the same form of deformation, relative to the wall, in each case.

The finite element solution furnishes the displacements and stress values at all nodes, and also the slab reactions at the restrained nodes. The reactions at a set of wall nodes provide the static equivalent wall moment, M, and the shear force, Q, transferred from the wall to the slab when the wall undergoes the unit relative displacements assumed. Evaluation of the appropriate force-displacement relationship gives the coupling stiffness of the slab, from which the effective width may be calculated from equation (4) or (5).

A preliminary study was performed to determine the best form of element to use in the computations. A very wide range of simple and higher order elements, of triangular and rectangular shapes, were examined from the point of view of convergence, accuracy, and computing time required. As a result of these studies (9), it was concluded that the most suitable element was the simple rectangular Adini-Clough-Melosh element (10), with three degrees of freedom at each node. The finite element mesh adopted for the analysis was chosen to suit the particular geometrical configuration concerned and used between 88 and 104 elements, requiring the solution of approximately 300 to 360 equations for the structures described.

RESULTS AND DESIGN CURVES

The main results obtained from the numerical investigations are summarised in Figures 3 to 7, which show the influence of the most important parameters on the effective slab width for an interior bay with a slab continuous in each direction. The results are shown in a form which can be used directly, by interpolation if required, in a design situation.

Figure 3 shows the variation of effective slab width with length of opening, L, and bay width, Y. Similar results have been presented earlier (4, 5, 8), and the results have been used to confirm the accuracy of the method. A detailed investigation has revealed that the effect of variations in the absolute wall lengths can be disregarded if the results are presented in non-dimensional form as shown. In addition, the effect of dissimilar wall lengths in a pair of coupled walls can also be neglected if the ratio of the length of the shorter wall to that of the opening is greater than about 0.5.

Corresponding curves for an end bay, in which one edge is assumed continuous and the other free, are indicated by broken lines in Figure 3. The effective width is then found to be between 44 and 47 per cent of that for a doubly-continuous slab. On the basis of these results it may be anticipated that a reasonably accurate value for an end bay would be obtained by taking a value of 45 per cent of that for a corresponding interior bay. A subsidiary study was made of the influence of a slab overhang beyond the outer edge of each wall; it was found that this had a negligible effect on the effective width and could be disregarded.

The design curves of Figure 3 have been established for walls of zero (line) thickness. In certain circumstances, particularly when the opening is small, the influence of the finite wall thickness may become significant and studies have been made of this effect. Figure 4 shows the variation of effective width with wall thickness and size of opening. The wall thickness has a considerable stiffening effect on the slab and is relatively more significant with small wall opening ratios L/X; for the range of values of Z/Y less than about 0.4, the effective width increases almost linearly with values of Z/Y. The results obtained are also applicable to the case of coupled box cores with no openings on the inner edge since the displacements imposed on the slab by a thick solid wall and a box core of the same external dimensions are identical.

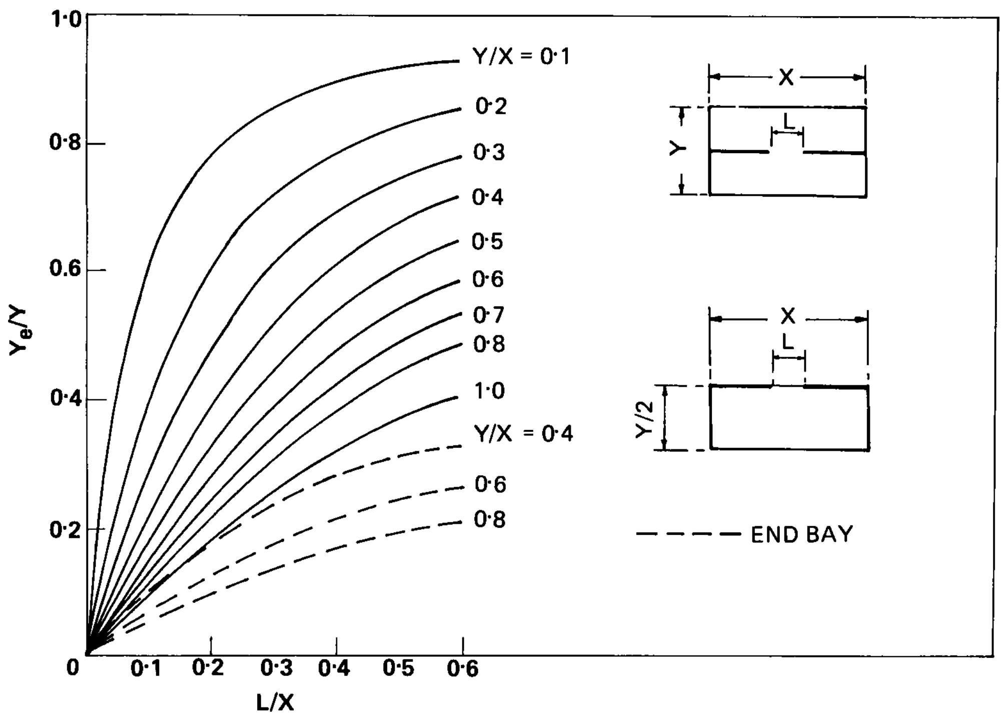

Figure 3 Influence of Bay Width and Opening Size on Effective Width for Plane Walls

The variation of slab effective width with flange width and wall spacing for coupled walls with interior flanges of equal width is shown in Figure 5(a). These design curves are a comprehensive set derived from a series of individual curves used to investigate the influence of the ratios L/X, Z/Y, and Y/X. When using these non-dimensional shape ratios, it was found that the absolute wall (web) length had a negligibly small influence on the effective width. This must be anticipated since the regions of the slab behind the wall flanges are relatively lightly stressed and play little part in the structural actions. The generalised design curves are accurate, compared to more detailed curves showing the influence of wall length, when the wall and slab proportions are such that $[(L/X) + (Y - Z)/X] \leqslant 1$, which is true for most practical situations. In unusual cases where $[(L/X) + (Y - Z)/X] > 1$, and $L/X > 0.4$, the effective widths tend to be overestimated, although the error is always less than 10 per cent.

Corresponding design curves are shown in Figure 5(b) for the case of a plane wall coupled to a T-shaped wall. The generalised curves are accurate for walls of normal proportions such that $[(L/X) + (Y/X)] \leqslant 1$. When the sum of L/X and Y/X exceeds unity, and L/X is also greater than 0.4, the values obtained from the generalised curves are generally overestimated, although the errors are again less than 10 per cent. It is seen that the omission of one flange from the wall configuration results in a disproportionately large reduction in the effective width when the ratio L/Y is small.

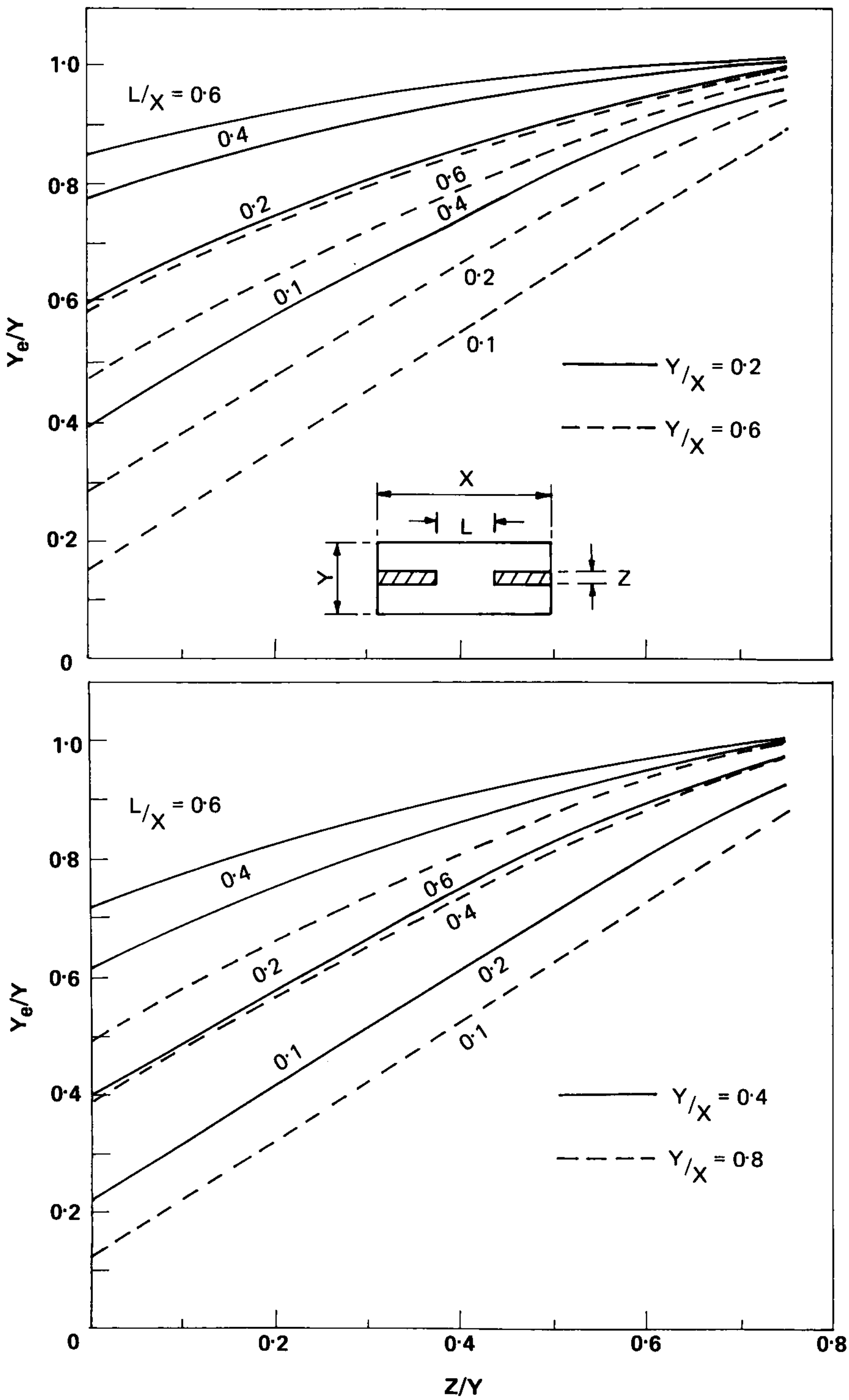

Figure 4 Influence of Wall Thickness on Effective Slab Width for Plane Coupled Wall Structures

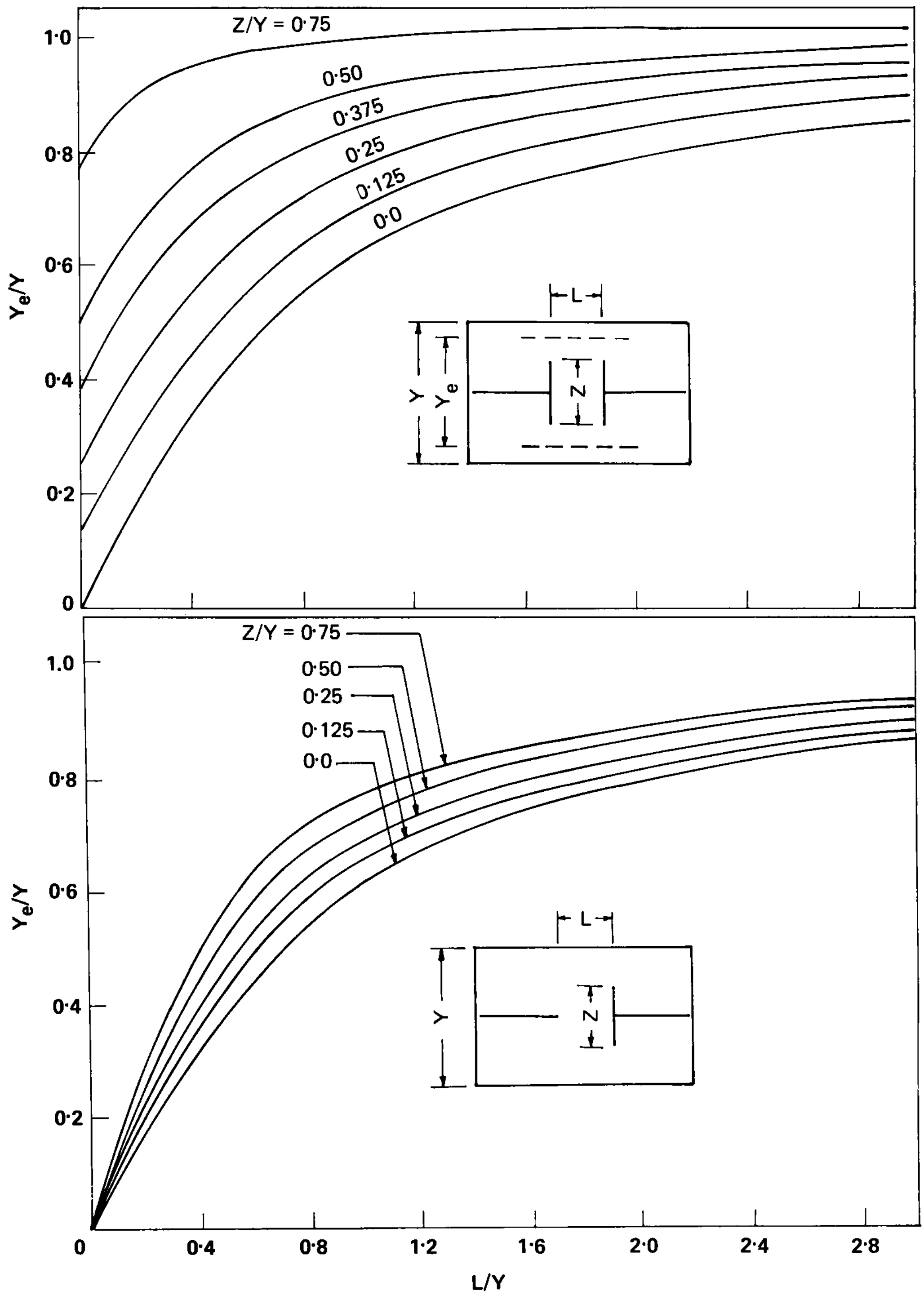

Figure 5 Variation of Wall Opening on Effective Width for (a) Coupled Flanged Walls and (b) Coupled Planar-Flanged Walls

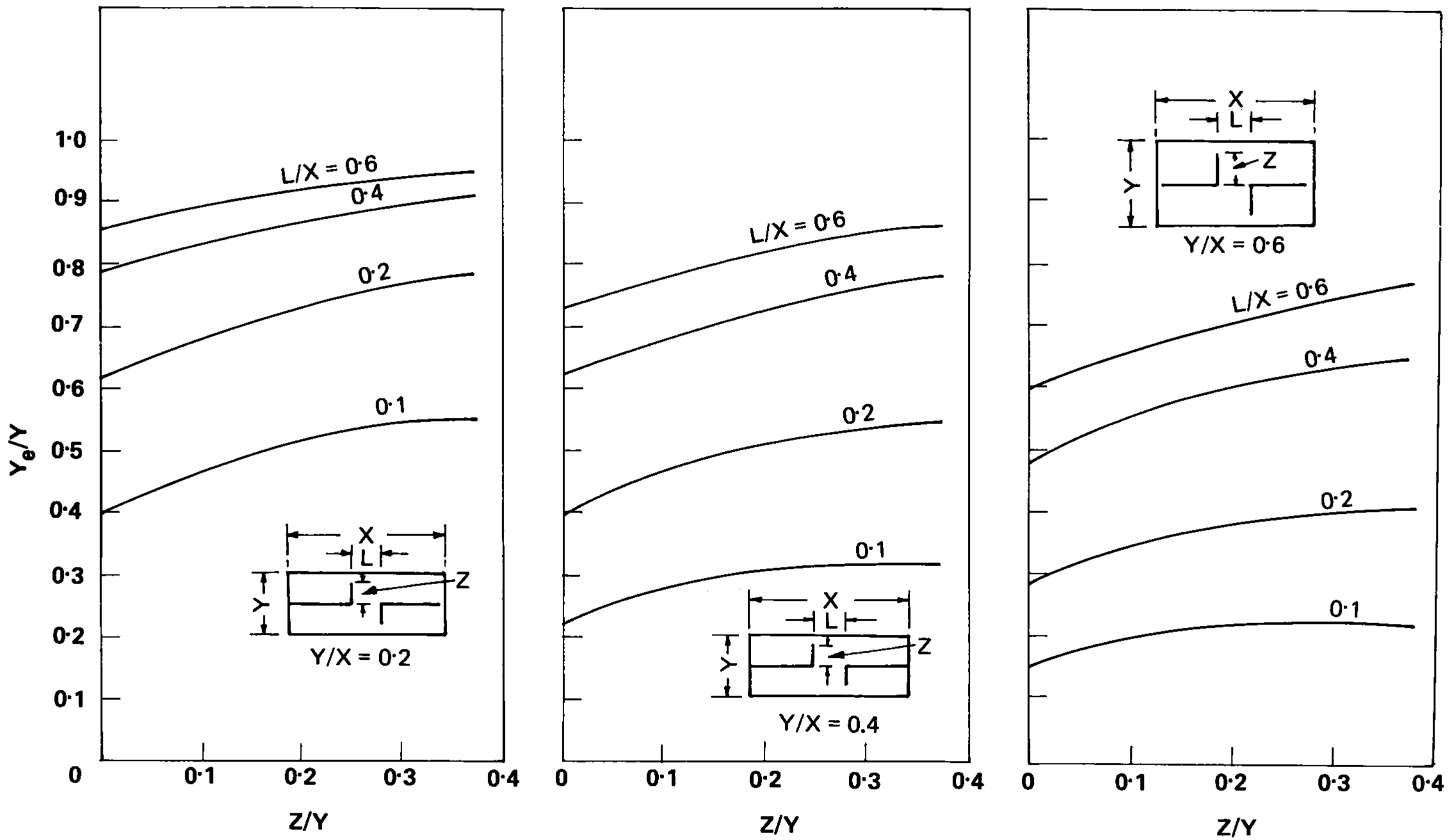

Figure 6 Influence of Flange Width and Wall Opening on Effective Width for Offset L-shaped Walls

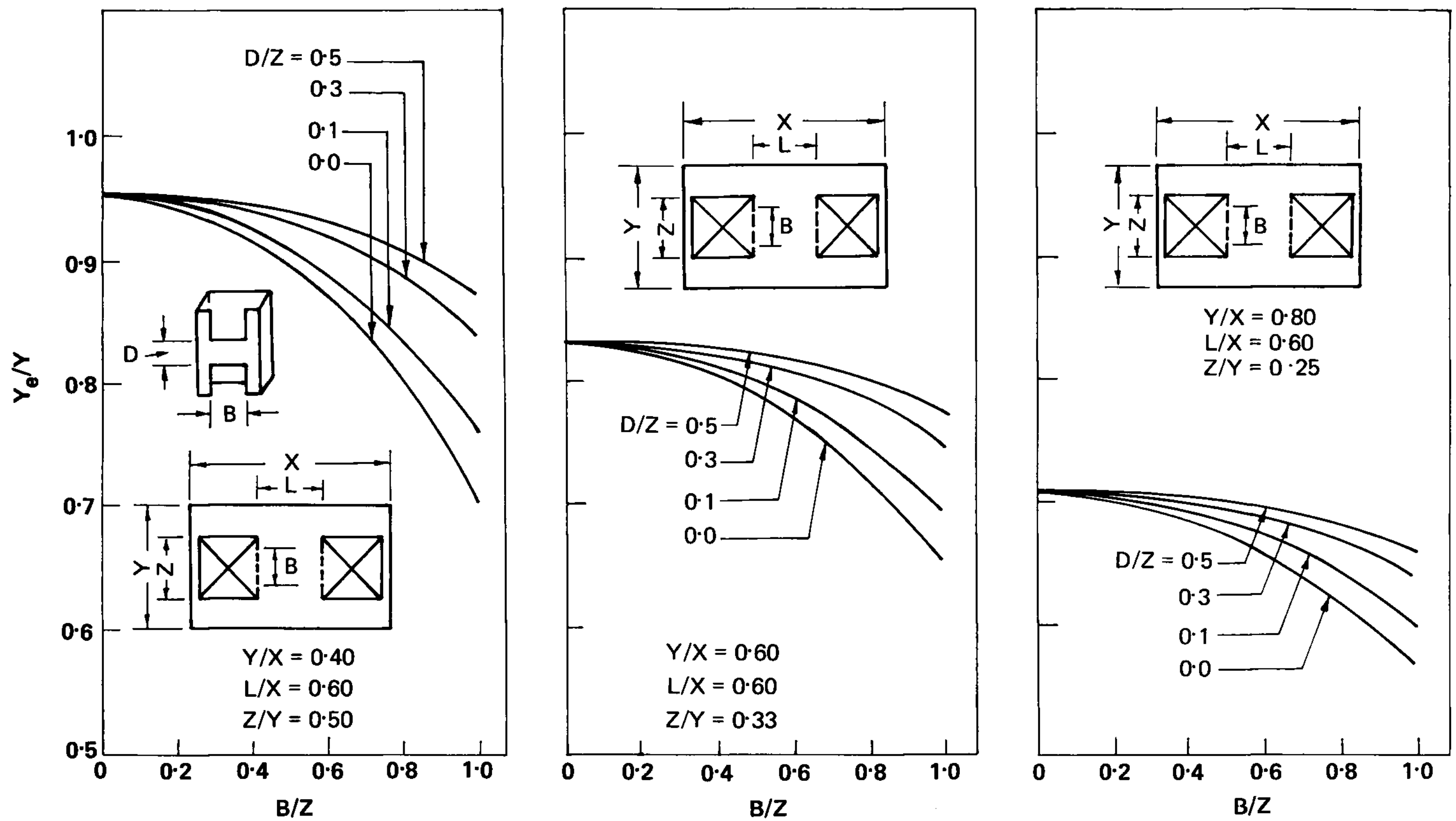

Figure 7 Influence of Opening Size and Lintel Depth on Effective Width for Slab Coupling Open Box Core Structures

The two sets of generalised curves of Figure 5 have been presented to avoid the alternative of a large set of at least eight curves showing the complete range of parameters involved. A corresponding series of calculations for walls with external wall flanges revealed that the flange then has a negligible effect, since all the coupling actions between wall and slab are concentrated over a short length at the inner edges. Exterior facade flanges may thus be neglected, and the walls treated as plane walls.

Coupled L-shaped walls may occur in different configurations. In the configurations shown in Figure 2(e), the cross-walls or web walls are connected in line with opposing flanges, whilst for the configuration of Figure 2(f), the webs are coupled off-line with opposing flanges. The numerical studies performed indicate that the effective widths in these two cases may be obtained directly from the curves presented for coupled T-shaped walls, see Figure 5(a). In the third possible configuration shown in Figure 2(g), the webs are coupled in-line but the flanges are off-set. For this configuration, the results obtained are shown in Figure 6. It can be seen that although the flanges are not directly cross-coupled, they still have a considerable influence on the slab stiffness as a result of their restraining action. It is found that for the same total flange width, the effective slab width in the coupled L-shaped walls and in the planar-flanged wall configuration are practically identical for any corresponding set of wall slab ratios L/X, Y/X and Z/Y. The effective contribution of off-set flanges has been confirmed experimentally by El-Buluk (7).

Although the curves presented for thick walls, Figure 4, apply also to box cores, situations occur in which opposing access openings occur in the cores. The stiffness of the connecting slab will be influenced both by the size of opening and the flexibility of the lintel beams provided at each storey level. In the analysis, the lintel beam is considered as an integral part of the floor slab connected to the core, and is represented by prismatic beam elements along the centroidal axis of the slab. Although the actual eccentric connection between lintel and slab is not being properly represented, this should not affect the results greatly as the action of the slab on the beam is mainly a torsional action in this case. The effective widths Y_e/Y have been evaluated for slabs of various aspect ratios, Y/X, coupling a pair of square cores with various access opening ratios, B/Z, and lintel beams depth ratios, D/Z, for fixed L/X and Z/X ratios. The results are shown in Figure 7. It can be seen that the influence of a core opening is relatively more significant with larger core width ratios, Z/Y, and is not very significant when the core opening width, B/Z, is less than 0.5. With a full core opening (B/Z = 1, D/Z = 0), the effective width of the worst affected slab is reduced by about 25 per cent due to the presence of the opening.

The results throughout have been derived on the assumption of a Poisson's ratio, V, of 0.15 for concrete. The slab stiffness factors, K, are not sensitive to small changes in Poisson's ratio, and, if desired, the value of Y_e/Y obtained from the various design curves may be corrected approximately for any other value of V by multiplying by a factor of $(1 - 0.15^2)/(1 - V^2)$, c.f. equations (4) and (5).

CONCLUSIONS

A series of non-dimensional design curves has been presented for the evaluation of the effective width, and hence stiffness, of a floor slab coupling a pair of shear walls, of plane, T, or L-shape or of open or closed box form. The curves have been presented for a wide range of the structural parameters involved. Once the effective stiffness is obtained, the overall analysis of the coupled wall structure may be carried out using an equivalent frame or continuum solution.

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OPTIMUM DESIGN OF REINFORCED CONCRETE AND PRESTRESSED CONCRETE SLABS

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ABSTRACT Procedures are presented for the practical design and cost minimization of reinforced and prestressed concrete slabs of constant thickness designed to satisfy the requirements of A.C.I. Standard 318-77. The costs considered include costs of material, falsework, formwork and labour. The effect of changes in the slab thickness and weight on the cost of other building elements is also included in order to obtain the slab design required to minimize the total building cost. The solution procedures combine re-solution, to obtain the optimum values of variables considered to be discrete, with direct differentiation, to obtain the optimum values of variables considered to be continuous.

INTRODUCTION

The design of slabs of minimum cost is difficult using standard mathematical optimization techniques because of the large number of variables and constraints. The solution procedures which are presented avoid this difficulty by introducing approximations which reduce the number of variables and constraints. The solutions obtained using these procedures have been found to be virtually identical to those determined using more rigorous methods. The design procedure has been developed to follow the requirements of A.C.I. 318-77 (1) but could be readily modified to follow the provisions specified in other codes of practice.

The procedures which are presented are applicable to solid reinforced or prestressed one-way or two-way slabs of constant thickness and flat plates without drop panels or shear reinforcing.

REINFORCED CONCRETE SLABS

Design Variables

For a solid reinforced concrete slab with N design sections, the design variables are: concrete strength, f'_c, and density, γ_c; steel yield strength, f_y; slab depth, d; steel ratios for all sections, ρ_i, $(i = 1, 2, \ldots, N)$; the cover to the centroid of the tension steel, d_s; and the ratio between the volume of temperature reinforcement and the total volume of the slab, ρ^t.

In practice, only a limited number of choices of f_c', γ_c, and f_y are commercially available. It is, therefore, appropriate to assume certain discrete values for these variables and re-solve the problem to determine the cheapest combination. The optimum values of the cover, d_s, and the temperature steel ratio, ρ^t, are the minimum allowable. On the other hand, the depth and the steel ratios may be assumed to be continuous variables since they may be rounded to values which are only marginally different from the desired optimum values; in practice, for example, the depth would normally be varied in increments of 10 mm.

Cost Equation

The cost function includes the costs of materials, formwork, falsework, labour, fixed costs and the direct effect of the slab thickness on the cost of other elements of the building. By including this last cost, the slab which is obtained is the one which will minimize the total building cost.

The in-place costs of concrete and main reinforcing and temperature steel are assumed to be directly proportional to their volumes:

$$\text{material cost} = C_c A(d + d_s) + \sum_{i=1}^{N} C_s b_i \ell_{si} d\rho_i + C_s \rho^t A d$$

where C_c and C_s are the costs of concrete and steel per unit volume, b_i and ℓ_{si} are the width and average steel length of the i^{th} section and A is the area of the slab.

The formwork, falsework and labour costs per unit area are assumed to be the sum of fixed costs and costs which are proportional to the thickness of the slab. As the thickness of the slab is increased, its weight and, consequently, the cost of the supporting elements will increase, and, in addition, the cost of exterior walls and shear walls will also increase.

The total costs which affect the optimization are:

$$C = C_c A(d + d_s) + \sum_{i=1}^{N} C_s b_i \ell_{si} d\rho_i + C_s \rho^t A(d + d_s) + C_{ad} A d + C_{ao} A \qquad (1)$$

where C_{ad} is the sum of the thickness related costs including the part of the cost of formwork that is proportional to the thickness, and C_{ao} is the sum of the costs which are not affected by d and ρ_i but change for different combinations of f_c', γ_c and f_y.

Additional fixed costs such as finishing costs could be considered, but they do not affect the optimization because they are independent of the design variables.

Design Constraints

The flexural strength of each of the N sections must equal or exceed the design moment:

$$\phi f_y \rho_i d^2 (1 - 0.59\rho_i f_y / f_c') \geqslant [1.4W_D + 1.7W_L + 1.4\gamma(d + d_s)]\, K_i \qquad (2)$$

where W_D and W_L are the dead and the live loads per unit area, ϕ is the capacity reduction factor and K_i is a moment coefficient which equals the moment at the i^{th} section due to a unit uniform load distributed over the slab. These moment coefficients have to be assumed initially and then adjusted if necessary to be consistent with the optimum design produced.

The depth is the only variable affecting shear strength; therefore, the required shear strength of the slab can be provided if the depth is equal to or larger than a minimum depth d_v.

Deflections are affected by the depth and the steel ratios since both affect the moment of inertia of the slab. However, the effect of the steel ratios is relatively minor and the deflection control requirement may, as an approximation, be replaced by a requirement that the depth be equal to or larger than a minimum depth d_Δ. The depth d_Δ may be obtained iteratively by assuming a steel ratio and trying different depths until one is found which is just sufficient to limit deflections. Simplified expressions for d_Δ have been suggested (2,4).

To obtain ductile behaviour the maximum steel ratio is limited to 0.75 of the balanced steel ratio, ρ_b, corresponding to f_y and f'_c. This requirement will be satisfied for all sections if the depth is not less than the depth d_ρ which results when a ρ equal to $0.75\rho_b$ is substituted into equation (2) for the section of maximum moment.

Greater minimum depths may be required to satisfy other constraints such as fire resistance. Thus:

$$d \geqslant d_{min} \tag{3}$$

where d_{min} is the largest of d_v, d_Δ, d_ρ and other minimum depths.

To avoid brittle behaviour, the minimum amount of reinforcement is specified to be equal to the required amount of temperature steel:

$$\rho_i \geqslant \rho_{min} = A_{s_{min}}/d = \rho^t(d + d_s)/d \tag{4}$$

Solution Procedure

A satisfactory approximate relation between steel ratios for different sections is:

$$\rho_i/K_i = \text{Constant} = \rho_o/K_o \tag{5}$$

where K_o is an arbitrary reference moment coefficient and ρ_o is a corresponding reference steel ratio. An appropriate value of K_o is the weighted average: $K_o = [\sum_{i=1}^{N} b_i \ell_{si} K_i]/[\sum_{i=1}^{N} b_i \ell_{si}]$, but any value of K_o near the average will be satisfactory. The N flexural strength constraints may then be replaced by one equation:

$$\phi f_y \rho_o d^2(1 - 0.59\rho_o f_y/f'_c) \geqslant [1.4W_D + 1.7W_L + 1.4\gamma(d + d_s)]\,K_o \tag{6}$$

plus the requirements at each of the N sections that:

$$\rho_i \geqslant \rho_o K_i/K_o \tag{7}$$

Assume that the sections are numbered so that the moment coefficients are in ascending order. At the optimum depth the minimum steel ratio, ρ_{min}, will govern for the first few, say L, sections with lowest moment while the strength requirement as given by equations 6 and 7 will govern for the remaining N-L sections. Initially L is unknown but can be found using the iterative procedure outlined in Figure 1.

At each step of the proposed procedure a value for L is assumed. Substituting

$\rho_i = \rho_{min} = \rho^t(d + d_s)/d$ for $i = 1$ to L and $\rho_i = \rho_o K_i/K_o$ for $i = L + 1$ to N in the cost function and collecting terms, the cost can be written as:

$$C = C_o + C_1^L d + C_2^L d\rho_o \tag{8}$$

where $C_1^L = (C_c + C_{ad} + C_s\rho^t)A + \sum_{i=1}^{L} C_s b_i \ell_{si}\rho^t$,

$C_2^L = \sum_{i=L+1}^{N} C_s b_i \ell_{si} K_i/K_o$,

and C_o = a constant.

Corresponding to the assumed L, the optimum depth, d^{*L}, may be obtained by substituting for ρ_o in terms of d from equation (6) into the cost equation (8) and setting the derivative of the cost with respect to the depth equal to zero. Thus:

$$d^{*L} = q_1\{1 + \sqrt{[1 + (q_2/q_1)]/[1 - q_3^2]}\ \} \tag{9}$$

where $q_1 = 2 \times 0.59 \times 1.4\gamma K_o/(\phi f_c')$,

$q_2 = 2[1.4(W_D + \gamma d_s) + 1.7W_L]/(1.4\gamma)$,

and $q_3 = 1/[1 + 1.18C_1^L f_y/C_2^L f_c')]$

If at any step of the iteration the strength of the L^{th} section is sufficient with $\rho = \rho_{min}$ and section L + 1 requires $\rho < \rho_{min}$, then L is incremented and the procedure repeated; however, the iteration is terminated if section L + 1 requires $\rho \geqslant \rho_{min}$. If the strength of the L^{th} section is not sufficient, then at the optimum depth both the strength and minimum steel constraints will govern for section L and the optimum depth is:

$$d = a_2/(2a_1) + \sqrt{[a_2/2a_1)]^2 + a_3/a_1} \tag{10}$$

where $a_1 = \phi f_y\rho^t(1 - 0.59\rho^t f_y/f_c')$,

$a_2 = 1.4\gamma K_L - d_s\phi f_y\rho^t(1 - 1.18\rho^t f_y/f_c')$,

and $a_3 = [1.4(W_D + \gamma d_s) + 1.7W_L]K_L + d_s^2\phi f_y\rho^t(0.59\rho^t f_y/f_c')$

This procedure results in progressively decreasing values of the depth. If at any stage the depth becomes less than d_{min}, then d_{min} is the optimum feasible depth. Through the solution of a large number of problems we have observed that the optimum reference steel ratio corresponding to an assumed L is approximately:

$$\rho_o^{*L} \simeq C_1^L/C_2^L$$

An approximate value for d^{*L} (instead of using equation 9) may then be obtained by substituting this steel ratio into equation (6).

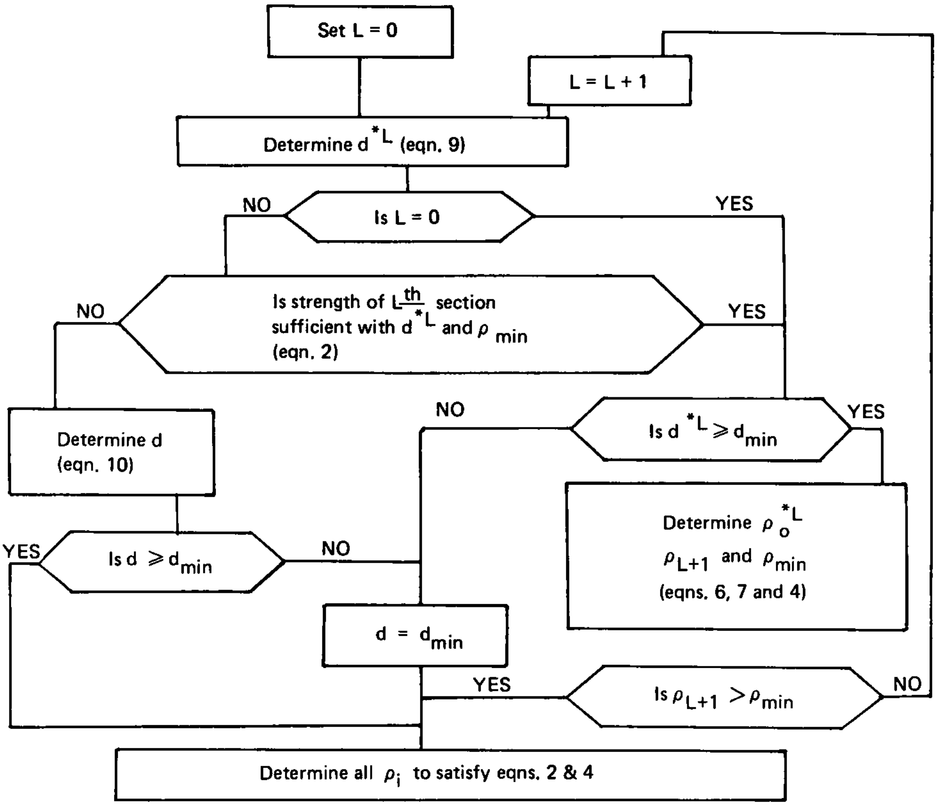

Figure 1 Flow Chart of Iterative Procedure

PRESTRESSED CONCRETE SLABS

Design Variables

As with reinforced concrete slabs, only limited choices of material properties are available so the optimum combination is most conveniently determined by re-solution A slab can be designed as a number of frames, all of equal slab thickness, h.

For prestressed slabs, the solution procedure is simplified by re-solving the problem with discrete values of h in increments of 10 mm. Prestressing tendons are assumed to be parabolic in each span, with profiles as shown in Figure 2 and with a constant prestressing force for the full length of the tendon.

The prestressing force can be different in each frame. To simplify this presentation a one-way slab consisting of only one frame will be considered. For the purpose of analysis, the prestressing may be replaced by a concentric compression force equal, at service conditions, to P per unit width and an upward load W_p per unit area which is assumed to be constant over all spans. The prestress required for any span of length, ℓ, and sag, s, is:

$$P = W_p \ell^2/(8s) \tag{12}$$

The required prestress in each frame will be governed by the span with the largest ratio of ℓ^2/s. The maximum possible sag should be used for this span and the sags in other spans should be chosen so that ℓ^2/s is constant. Each design section requires bonded reinforcing steel with an area equal to $\rho_j(h - d_s)$ where d_s is the cover to the centre of this steel. Design variables then are the discrete variables f'_c, γ, f_y and h and the continuous variables W_p and the steel ratios ρ_j.

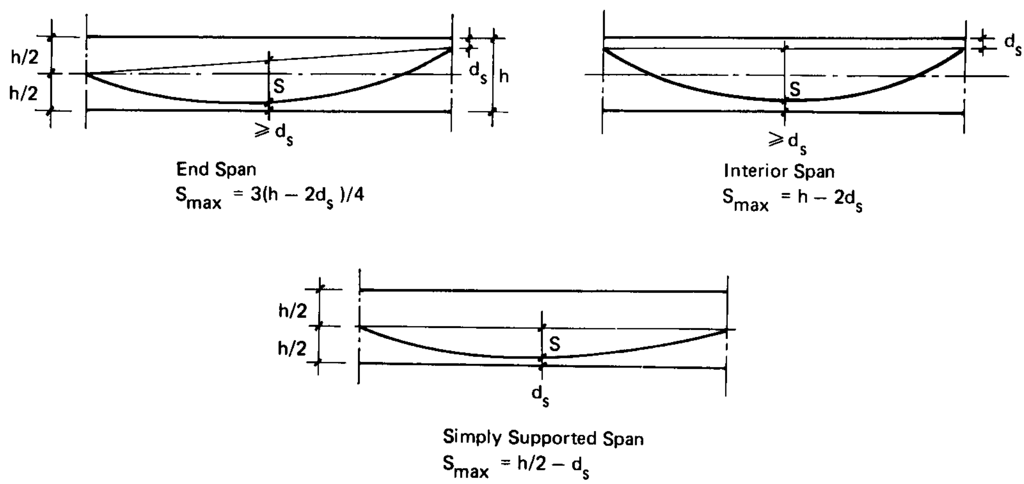

Figure 2 Assumed Cable Profile

Cost

The cost function includes the costs considered for reinforced concrete slabs plus the cost of prestressing steel which is assumed to be equal to C_p per unit of the tendon length, ℓ_t, per unit prestressing force.

$$\text{Total Cost} = C = C_c Ah + \sum_{i=1}^{N} C_s b\ell_{si} d\rho_i + C_s\rho^t Ah + C_p\ell_t Pb + C_{ad}Ah + C_{ao}A \tag{13}$$

where $d = h - d_s$,
b = slab width,
A = total slab area,
and N = number of sections

Design Constraints

For formulating design constraints at transfer and at ultimate, it is assumed that the magnitudes of the prestressing force are equal to q_tP and q_uP, respectively. Values of q_t and q_u are typically about 1.1 and 1.2, respectively. These values may be modified slightly if a more detailed analysis is made.

The design moment for each section is equal to $1.4(W_D + \gamma h)K_{dj} + 1.7W_LK_{\ell j}$ where K_{dj} and $K_{\ell j}$ are the moments at the j^{th} section due to a unit dead load and live load, respectively. A portion of this moment is counteracted by the moment due to the upward load from prestressing. At ultimate this load is $q_u W_p$ and the resulting moment at each section is $q_u W_p K_{dj}$. The concentric compression force ($q_u P$ at ultimate) counteracts another portion of the moment since it has a moment about the centre of compression in the concrete. This moment is equal to $q_u P(h/2 - a_j/2)$ where a_j are the depths of the equivalent rectangular compression blocks, which may be estimated quite closely but may be adjusted if necessary. The remainder of the design moment must be resisted by bonded steel. Thus, for each section:

$$\phi f_y \rho_j (h - d_s)(h - d_s - a_j/2) \geqslant 1.4(W_D + \gamma h)K_{dj} + 1.7W_L K_{\ell j} - \phi q_u W_p K_{dj}$$
$$- \phi q_u \{W_p \ell^2/(8s)\}(h/2 - a_j/2) \tag{14}$$

where ϕ is the capacity reduction factor. For given values of h and W_p, steel ratios which provide the required strength for each section can be determined from equation (14).

The flexural strength of each design section is also required to be at least equal to 1.2 times the cracking moment. Thus:

$$\phi f_y \rho_j (h - d_s)(h - d_s - a_j/2) + \phi q_u W_p K_{dj} + \phi q_u \{W_p(\ell^2/8s)\}(h/2 - a_j/2) \geqslant 1.2M_{crj} \tag{15}$$

where $M_{crj} = W_p K_{dj} + W_p(\ell^2/8s)(h/6) + f_r h^2/6$,
and f_r is the modulus of rupture of concrete.

For a given h and W_p, equation (15) can be used to determine steel ratios for which $M_{uj} = 1.2M_{crj}$.

The maximum tensile and compressive stresses, f_{tj} and f_{cj}, at any section are:

$$f_{tj} = -\frac{W_p \ell^2}{8sh} + \frac{6}{h^2}[(\gamma h + W_D - W_p)K_{dj} + W_L K_{\ell j}] \tag{16a}$$

$$f_{cj} = -\frac{W_p \ell^2}{8sh} - \frac{6}{h^2}[(\gamma h + W_D - W_p)K_{dj} + W_L K_{\ell j}] \tag{16b}$$

By limiting these stresses to their allowable values at service, a minimum limit for W_p can be obtained as the largest of the values obtained from equations (16a) and (16b) for all sections. Generally the section with the maximum moment will govern.

At transfer, the maximum tensile and compressive stresses are:

$$f_{tj} = -\frac{q_t W_p \ell^2}{8sh} - \frac{6}{h^2}[(\gamma h - q_t W_p)K_{dj}] \tag{17a}$$

$$f_{cj} = -\frac{q_t W_p \ell^2}{8sh} + \frac{6}{h^2}[(\gamma h - q_t W_p)K_{dj}] \tag{17b}$$

These stresses will be satisfactory if W_p is smaller than the smallest of the two values obtained from equations (17a) and (17b), for the section with maximum K_{dj}, with f_{tj} and f_{cj} set equal to their limiting values.

The code (1) specifies that, for each section:

$$\rho_j \geqslant N_{cj}/[0.5f_y(h - d_s)] \tag{18}$$

where N_{cj} is the tensile force in the concrete when subjected to the dead load plus 1.2 times the live load, i.e.

$$N_{cj} = \frac{1}{2}[f_{tj}h/(f_{tj} - f_{cj})]f_{tj} \tag{18a}$$

where f_{tj} and f_{cj} are the maximum tensile and compressive stresses obtained from equations (16a) and (16b) with $1.2W_L$ instead of W_L. This constraint needs to be considered only when f_{tj} is positive, i.e. in the case of tension.

For each section, the area of bonded reinforcing steel must also be at least equal to 0.002 times the gross area of the section. Thus:

$$\rho_j \geqslant 0.002h/(h - d_s) \tag{19}$$

To obtain ductile behaviour, it is specified that

$$\rho_j \frac{f_y}{f'_c} + \frac{P}{f_{se}(h - d_s)} \frac{f_{ps}}{f'_c} \leqslant 0.3$$

where f_{se} and f_{ps} are the stresses in the prestressing steel at service and at ultimate, respectively. The ratio f_{ps}/f_{se} is q_u, and therefore:

$$\rho_j \leqslant \rho_{max} = 0.3\,\frac{f'_c}{f_y} - \frac{q_u W_p \ell^2}{8s(h - d_s)f_y} \tag{20}$$

Deflections are a function of h and W_p but not ρ_j. Therefore, limits on h and W_p are sufficient for deflection control. Immediate deflections due to live loads, initial camber due to prestressing and downward deflections at service will be satisfactory if h is larger than a minimum h and W_p is between its minimum and its maximum limit. These limits can be obtained by equating the relevant deflections to their limiting values.

Solution Procedure

For each combination of material properties, the optimum feasible h and the corresponding optimum W_p and ρ_j are determined by the following steps.

A - Feasible Region

Step 1. Start with the smallest value of h for which the immediate deflection due to live load is satisfactory and which satisfies any other minimum thickness requirements, such as that for fire resistance.

Step 2. Determine $W_{p\,min}$ as the largest of the minimum values of W_p required to limit downward deflections and stresses at service. Also determine $W_{p\,max}$ as the smallest of the limiting maximum values of W_p for allowable stresses at transfer and for control of camber.

Step 3. If $W_{p\,min} > W_{p\,max}$ then the current h is not feasible; increment h and start Step 2 again.

Step 4. Using $W_{p\,min}$ determine ρ_{min} as the largest of the steel ratios required to satisfy equations (14), (15), (18) and (19). If ρ_{max} from equation (20) is less than ρ_{min}, increase W_p to the value which makes ρ_{min} from the limiting equation equal to ρ_{max} from equation (20) and return to Step 3. If no real solution exists then increment h and return to Step 2.

Step 5. Using $W_{p\ max}$ determine ρ_{min} as the largest of the steel ratios required to satisfy equations (14), (15), (18) and (19). If ρ_{max} from equation (20) is less than ρ_{min}, decrease W_p to the value which makes ρ_{min} from the limiting equation equal to ρ_{max} from equation (20) and begin Step 5 again.

B - Optimum Prestress for a Given h

For a given slab thickness the cost can be expressed as a function of the single variable W_p since the optimum steel ratios can be expressed in terms of W_p as they will be the minimum necessary to satisfy the constraint equations. For any continuous slab there will be so many of these constraint equations that an exact solution for the optimum W_p is difficult. The authors have found that the cost can be satisfactorily approximated by a quadratic equation fitted using the costs at three values of W_p. The three values that can be expected to give the best fit are:

$$W_{p_1} = W_{p\ min} + 0.11(W_{p\ max} - W_{p\ min}),$$

$$W_{p_2} = W_{p\ min} + 0.5(W_{p\ max} - W_{p\ min}),$$

and $$W_{p_3} = W_{p\ min} + 0.89(W_{p\ max} - W_{p\ min}).$$

The optimum W_p for a given h can be determined, therefore, as follows.

Step a. Determine W_{p_1}, W_{p_2} and W_{p_3}.

Step b. For each W_p, determine the steel ratios, ρj, for each section, the largest ρ from equations (14), (15), (18) and (19), and the corresponding costs from equation (13). Let these costs be C_1, C_2 and C_3.

Step c. Determine the value of $W_p(W_p^*)$ for which the cost given by the quadratic expression is minimum. It has been shown (3) that

$$W_p^* = \frac{1}{2}[B_{23}C_1 + B_{31}C_2 + B_{12}C_3]/[D_{23}C_1 + D_{31}C_2 + D_{12}C_3]$$

where $B_{ij} = W_{pi}^2 - W_{pj}^2$

and $D_{ij} = W_{pi} - W_{pj}$

Step d. If $W_p^* < W_{p\ min}$ set $W_p^* = W_{p\ min}$ and if $W_p^* > W_{p\ max}$ set W_p^* equal to $W_{p\ max}$.

Step e. Using h and W_p^* determine ρj, from equations (14), (15), (18) and (19), and the cost, from equation (13). This design is the optimum feasible with the current h.

C - Optimum Slab Thickness

Increment h and repeat the procedure starting at Step 2 until the cost begins to increase.

CONCLUSIONS

Application of the suggested procedures to representative slab design problems shows that these procedures are simple and result in designs which are virtually identical to the optimum determined using geometric programming (2) and sequential unconstrained minimization technique (SUMT), which are more difficult to use.

ACKNOWLEDGEMENT The financial assistance provided by the National Sciences and Engineering Research Council Canada and the University of Calgary, Alberta, is gratefully acknowledged.

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SOME ECONOMIC IMPLICATIONS IN REINFORCED CONCRETE SLAB DESIGN

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ABSTRACT The effect of materials cost and strength is examined in relation to the cost of reinforced concrete one-way slabs. Cost sensitivities of building code performance requirements are compared with those of basic design parameters and several trends are recognized. Assuming that formwork costs are independent of slab thickness, the cost of a slab can be expressed as a function of main and temperature reinforcement and concrete quantities. This function is then minimized subject to several constraints, the most important being the strength requirement. A computer programme using geometric programming was used to solve a large number of design problems for a wide range of loads and material strengths. It was found that the main reinforcement contributes between 20 to 28 per cent of the total cost while temperature steel contributes less than 3 per cent, concrete contributes between 20 and 30 per cent and formwork between 45 and 62 per cent. The use of high strength steel and low strength concrete resulted in lower costs. The optimum reinforcement ratio ranged between 53 and 60 per cent of that corresponding to balanced conditions, although the minimum cost was found to be not very sensitive to reinforcement ratio. For loads commonly found in practice, the slab thickness was found to be largely dictated by deflection considerations. A set of guidelines is proposed for the minimum cost design of one way slabs.

INTRODUCTION

The problem of designing a reinforced concrete flexural member can be defined by a few design parameters, which in turn can be used to establish a cost function in terms of strength, material properties, and building code requirements. The author (1) has undertaken an extensive study of the implications of these requirements on the cost of a structural element by establishing an analytical cost function and minimizing it with respect to a set of design parameters. The cost function is subject to several constraints which incorporate the various requirements. The technique of geometric programming (2,3) has been extensively and advantageously used to solve a large number of design problems including simply supported and continuous one-way slabs and beams with rectangular and T cross-sections. This paper reports only that part of the results obtained which pertains to one-way solid slabs (4).

NOTATION

A_s^t Area of temperature and shrinkage reinforcement
A_s Area of main steel reinforcement
C Cost per unit area of slab
C_c Cost per unit volume of concrete
For $f'_c = 20\ N/mm^2$, $C_c = 4.99 \times 10^{-8}\ \$/mm^3$
$f'_c = 30\ N/mm^2$, $C_c = 5.26 \times 10^{-8}\ \$/mm^3$
$f'_c = 40\ N/mm^2$, $C_c = 5.81 \times 10^{-8}\ \$/mm^3$
C_F Cost per unit area of formwork, $C_F = 2.42 \times 10^{-5}\ \$/mm^2$
C_s Cost per unit volume of main steel reinforcement
For $f_y = 300\ N/mm^2$ $C_s = 8.97 \times 10^{-3}\ \$/mm^3$
$f_y = 400\ N/mm^2$ $C_s = 9.35 \times 10^{-3}\ \$/mm^3$
C_s^t Cost per unit volume of temperature reinforcement
d Effective slab depth. Distance from extreme compressive fibre to centroid of tension reinforcement
f'_c Compressive strength of concrete
f_y Yield strength of reinforcing steel
h Overall slab depth, $h = kd$ for $k = 1.15$ to 1.40
ℓ Span length of one-way solid slab
M_u Ultimate moment capacity
W_D Self weight of slab plus superimposed dead load per unit width (kN/m^2)
$W_D = \gamma_c\ (1.0)h = \gamma_c\ kd$
W_L Service live load (kN/m^2)
W_u Design uniformly distributed load, $W_u = 1.7\ W_L + 1.4\ W_D$
γ_c^x Density of concrete
ϕ Capacity reduction factor, $\phi = 0.90$ for bending
ρ Reinforcement ratio, $\rho = A_s/bd$
ρ^t Temperature reinforcement ratio

PROBLEM FORMULATION

The design of a one-way solid slab is defined by two variables, namely the reinforcement ratio, ρ, and the slab depth, d, provided the concrete and steel strengths are specified. Building code requirements impose limits on the reinforcement ratio and slab depth to control ductility and deflections. For the range of concrete and steel strengths available and within the limits imposed on design parameters there are still a very large number of acceptable solutions to the design problem. The object then is to find that solution which results in a minimum cost and still satisfies all strength and safety requirements. Having found the values of the design parameters which give a minimum cost design, a sensitivity analysis is then performed to examine the implications of the assumptions made.

Objective Function

The cost per square metre of a one-way solid slab, Figure 1, is given by the following equation:

$$C = C_s\ \rho d + C_s^t\ \rho^t h + C_c h \tag{1}$$

The cost of the formwork is assumed to be independent of the slab depth and is not included in equation (1) but is added to the cost obtained from equation (1) after the optimization is carried out.

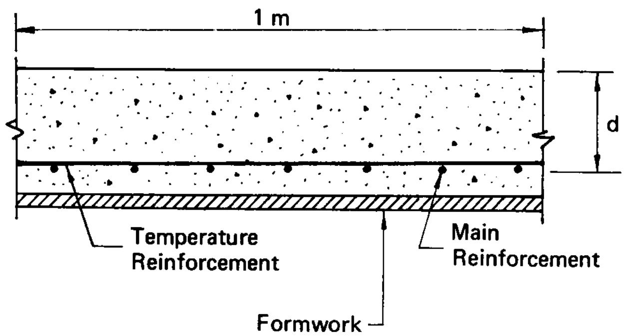

Figure 1 One-Way Solid Slab

Strength Constraint

The ultimate moment capacity of a 1m wide one-way slab is given by:

$$M_u = \phi \rho d^2 f_y (1 - 0.59 \rho \frac{f_y}{f'_c}) \tag{2}$$

The National Building Code of Canada (5) stipulates that reinforcement for shrinkage and temperature stresses, normal to the main reinforcement, shall be provided in structural floor and roof slabs where the main reinforcement extends in one direction only. For a specified steel yield strength, ρ^t is a known value. The over-all slab depth, h, is assumed to be equal to kd, where k may vary between 1.15 and 1.40.

The problem can now be formulated in a format suitable to solution using geometric programming.

Minimize

$$C = A_1 \rho d + A_2 d \tag{3}$$

Subject to

$$A_3 \rho^{-1} d^{-2} + A_4 \rho^{-1} d^{-1} + A_5 \rho \leqslant 1 \tag{4}$$

where

$$A_1 = C_s$$

$$A_2 = C_s^t \rho^t k + C_c k$$

$$A_3 = \frac{1.7 W_L^2}{8(0.9) f_y}$$

$$A_4 = \frac{1.4 \ k \ \gamma_c^2}{8(0.9) f_y}$$

$$A_5 = 0.59 \frac{f_y}{f'_c}$$

A computer programme using geometric programming was used to solve equations (3) and (4). A large number of design problems were formulated to find the optimum values of ρ and d under a wide range of loadings using concrete strengths ranging from 20 to 40 N/mm^2 and steel strengths from 300 to 400 N/mm^2.

RESULTS

Effect of Building Code Requirements

To limit deflection, the building code (5) requires that the thickness of a one-way solid slab does not exceed 1/20 of the span for simply supported slabs or 1/28 of the span for continuous construction. When this limitation is incorporated as a constraint in the minimization problem, it becomes an active constraint forcing d to a value larger than that required for strength alone and increasing the cost by as much as 30 per cent.

For ductility considerations, the building code limits the reinforcement ratio to 75 per cent of that which produces balanced conditions (simultaneous yield of reinforcement and crushing of concrete in compression). This limitation is always satisfied at optimum and has no effect on the cost, i.e. it becomes an inactive constraint.

Effect of Materials Strength

The ready-mixed concrete and reinforcing steel industries follow a pricing policy which establishes a basic cost plus a premium for added strength. Thus the price of concrete and steel increase as their strength increases. The cost of concrete and steel can then be formulated as an analytical function of their strength and be incorporated in the optimization. For the problems solved in this study, high strength steels consistently resulted in lower optimum costs. For a given steel strength, the use of low concrete strength resulted in lower costs and a thicker slab. If the cost of forming is truly independent of slab thickness, a thicker slab will satisfy code requirements better.

Effects of Materials Cost

Rapidly changing economic conditions affect the basic price of concrete and steel although the premium paid for strength remains practically unchanged. This results in a "shift" of the analytical cost-strength relationships. To study this effect, a price increase of 50 per cent was considered for concrete, steel and formwork and its effect was evaluated for slabs subject to loading producing bending moments ranging from 100 to 1000 kNm.

As shown in Figure 2, a 50 per cent increase in the cost of concrete will result in a total cost increase of less than 10 per cent, while a 50 per cent increase in the cost of reinforcing steel will result in a 10 to 15 per cent increase in the total cost. A 50 per cent increase in the cost of formwork will result in a change in total cost of 25 to 35 per cent depending on the concrete strength and the load applied.

Based on 1976 costs, the main reinforcement and temperature steel contribute between 20 and 28 per cent towards the optimum cost of one-way solid slabs, while concrete contributes between 20 and 30 per cent and formwork costs, assumed to be independent of slab thickness, account for 45 to 62 per cent of total costs.

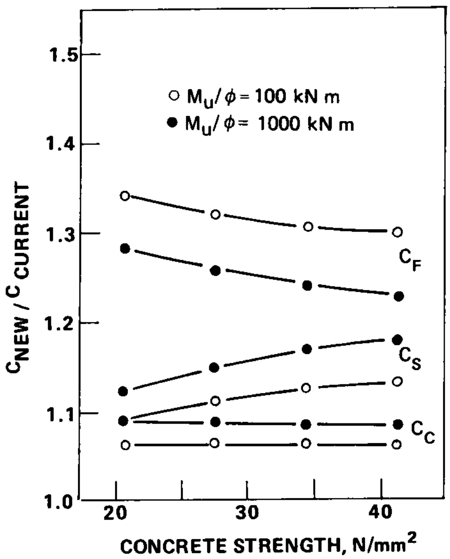

Figure 2 Cost Sensitivity with respect to a 50 per cent Increase in Cost of Materials (f_y = 400 N/mm^2)

Effect of Design Parameters

Although the optimum reinforcement ratio and slab depth may be determined, it may not be possible to use their exact values, the designer having to round them off in order to meet construction requirements. It is then necessary to determine the sensitivity of the total optimum cost to changes in the optimized parameters. Figure 3 shows the variation in the slab depth, d, required to support a superimposed live load for a changing reinforcement ratio (expressed as a ratio of the reinforcement ratio for balanced conditions). It can be seen that for the range of ρ required for minimum cost (50 to 60 per cent of ρ_b), the slab depth does not change more than 15 mm. Table 1 contains values of ρ_b for commonly used values of f_c' and f_y.

Table 1 Typical Values of ρ_b

f_y, N/mm^2	f_c', N/mm^2	ρ_b
300	20	0.0321
	30	0.0482
	40	0.0582
350	20	0.0261
	30	0.0391
	40	0.0472
400	20	0.0217
	30	0.0325
	40	0.0393

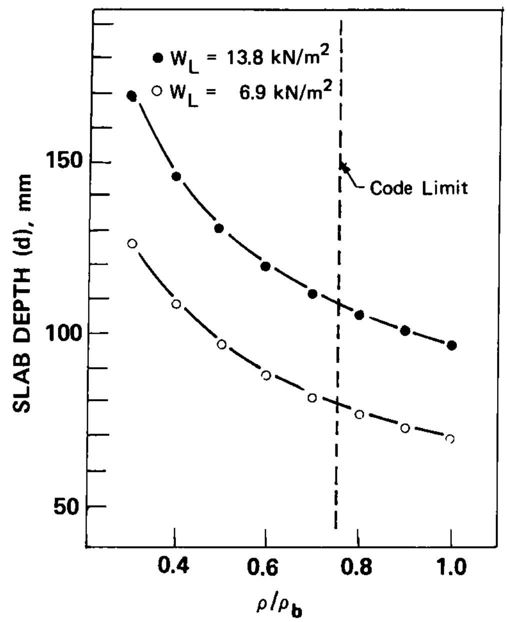

Figure 3 Study of Slab Depth versus Reinforcement Ratio

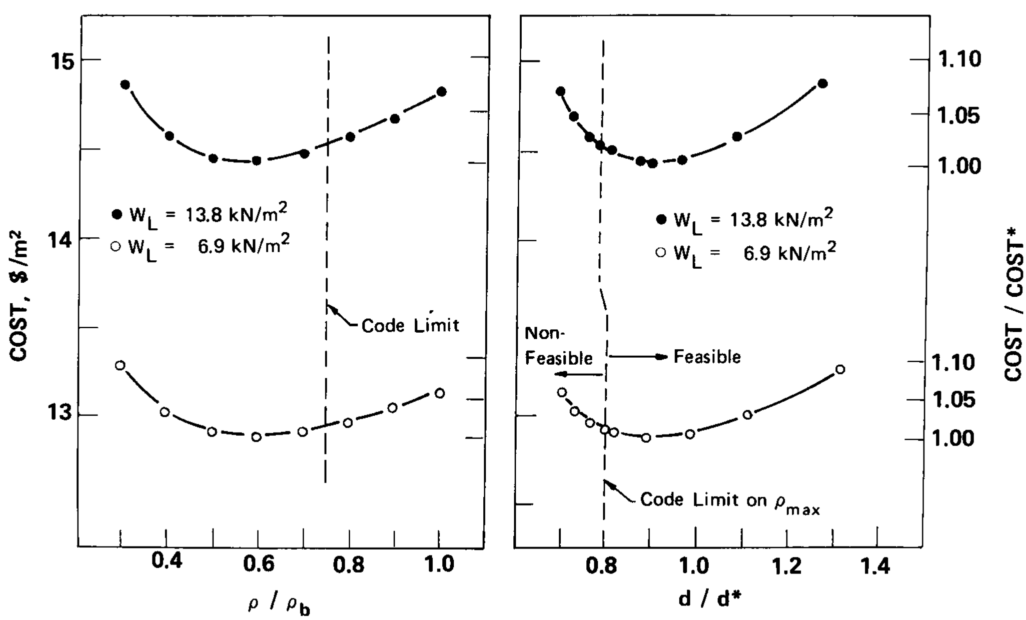

Figure 4 Cost Sensitivity with respect to (a) Reinforcement Ratio and (b) Slab Depth

Cost Sensitivity with respect to Reinforcement Ratio

In the vicinity of the optimum reinforcement ratio, the minimum cost is insensitive to variations in ρ. Thus a minimum cost can be obtained for a fairly wide range of values of ρ, allowing the designer both flexibility and economy. Figure 4(a) shows that optimum cost is achieved for values of ρ corresponding to 53 to 60 per cent of the balanced reinforcement ratio. Figure 4(b) shows the effect of slab depth, d, on cost. It can be seen that when rounding-off optimum depth d to a practical value, it is safer to round upwards since this takes the design away from the code limit on maximum ρ. It is also more economical since the slope of the curve is smaller in this direction, resulting in a reduced rate of change of costs with respect to slab depth.

SUMMARY OF RESULTS

The use of higher strength steel results in lower costs.
The use of lower concrete strength results in lower costs.
The optimum reinforcement ratio ranges from 0.53 ρ_b to 0.60 ρ_b, depending on the live load.
The optimum cost is insensitive to changes in ρ and d when these parameters are near their optimum values.
For loads commonly found in practice, the slab thickness is largely controlled by deflection considerations.

CONCLUSIONS

On the basis of the results summarized above, it is possible to adopt some guidelines for the design of reinforced concrete one-way solid slabs. It is believed that these guidelines will result in a design of minimum cost. Furthermore, the sensitivity analyses performed indicate that the results will remain valid for a wide range of changes in the cost of materials, in the light of changing economic conditions.

Design Guidelines

Choose a steel reinforcement of high strength.
Choose a concrete strength in the medium to low range.
Choose a reinforcement ratio between 0.53 ρ_b and 0.60 ρ_b. Upper bound for small live loads, lower bound for large live loads.
Determine effective slab depth, d, by solving equation (2).
Check for deflections.

After determining an optimum value for d, the designer must verify that it does not exceed the minimum thickness set by the appropriate code. If this limit is exceeded, the designer has the choice of computing deflections under actual live load conditions and satisfying the code's limits on deflections, or choosing a different value for ρ which will result in an acceptable thickness.

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FUTURE FRENCH RECOMMENDATIONS CONCERNING CONCRETE PAVEMENTS DESIGN

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ABSTRACT The paper is concerned with the analysis of concrete pavements intended for houses or to support industrial moving loads such as those found in factories and warehouses (e.g. lorries, lift trucks, etc). Minimal reinforcement is proposed for shrinkage effects and curling. Numerical Tables give the maximum stress for edge, side and central loads. Dispositions of reinforcement and joints are presented.

INTRODUCTION

Professional recommendations have been proposed in France (U.T.I.) in which two categories of concrete pavement are defined, (a) unreinforced pavements, and (b) reinforced pavements. The purpose of these recommendations is to facilitate the design of the rigid concrete pavement, in particular the thickness required and, if necessary, the amount of reinforcement. They offer comprehensive directions to enable decisions to be made on the following problems of reinforced pavement design: i) at which level should the reinforcement be placed and ii) must the reinforcement be stopped at straight joints?

Only very few chapters can be presented in this paper. The French recommendations, in fact, are divided into 3 parts:

> Part A: Conception (choice of pavement). This part is intended for the Architect.
>
> Part B: Construction of Pavement. This part is intended for the contractor. It deals with soil and sub-base foundation preparation, forms, concrete properties, installation of joints and reinforcement and finishing of concrete.
>
> Part C: Design of Pavement. This part is intended for the structural engineer.

GENERAL NOTATION

- σ_a allowable tensile stress in steel
- σ allowable tensile stress in concrete
- P wheel load
- h pavement thickness
- r radius of a circular area equal to that of the actual area of contact of a single tyre with pavement
- R radius of relative stiffness of the slab
- k subgrade support reaction
- L length of slab between joints

DEFINITIONS

The recommendations define the subgrade as the prepared and compacted soil below the pavement-system and the sub-base as the layer in a pavement system between the subgrade and the base course.

SOIL INVESTIGATION

A soil study is necessary in order to determine the subgrade strength and the recommendations give suggestions for the frequency of testing, depending on the purpose and extent of the pavements, as follows.

Pavement for Housing

For 1 house 3 soil tests are specified, while for a group of houses 1 test every 25 m is specified. If the pavement surface is over 20000 m^2, the recommendations call for 1 test every 2500 m^2.

Industrial Pavements

The number of tests for industrial pavements is related to the magnitude of the loads. The tests specified are the water content test, tests for the Atterberg limits, WL and WP, and, if the soil is composed of compressible layers, the oedometer test.

GENERAL

Basic Data

The owner should present, in his initial specification, data relating to the traffic area, the wheel loads, the coefficient of dynamic magnification, the loaded area and the permissible deflection in accordance with a satisfactory use of the pavement.

Factor of Safety

The ratio of the ultimate stress under bending and tension to the working stress ($\sigma = 6M/bh^2$) should range between 1.7 and 2.0 depending on the traffic volume and the magnitude of the loads.

Materials

Materials for the pavement system must meet the following requirements:

a) Fill materials should have less than 2 per cent organic elements. If the liquid limit is greater than 40 and the plasticity index greater than 17, the fill should be stabilised with a cementing agent.

b) All other materials should be in accordance with standard French specifications such as, for water: norme NF-P-18-303; for aggregate: norme NF-P-18-301 or NF-P-18-302.

PAVEMENTS FOR HOUSING

Rainfall

Rain has an influence on the stability and strength of the supporting medium because it affects the moisture content of the subgrade and sub-base. Guide lines are given in Figure 1.

Soil Fill Material

All material used as earth fill must meet the following specifications:

i) no gravel or stone shall be larger than 31.5 mm
ii) there shall be less than 20 per cent of material smaller than $80x10^{-6}$ m
iii) sand equivalent test shall be more than 40.

Foundation, Sub-Base and Subgrade

Recommendations concerning suitable foundation, subgrade and sub-base conditions are given, see Figures 2 and 3.

Joints

The bearing walls should be separated from the pavement by suitable expansion/isolation joints, see Figure 4, or a suitably well compacted stone sub-base should be used, see Figure 3, to avoid edge cracking due to differential settlement, see Figure 5.

The area of pavement surface between contraction joints should generally be less than 240 m^2. In the case of a pavement sitting on clay, areas between joints should be very small.

Slab Dimensions

Recommendations for the dimensions and details of the pavements are given as a thickness of concrete of 80 to 120 mm, with a cement content of 300 to 350 kg/m^3, and reinforcing steel as one layer of bar mat equivalent to a minimum of 1.10kg/m^2. Additional reinforcement is needed in the vicinity of the walls, see Figure 6, when the pavement is not isolated from the walls.

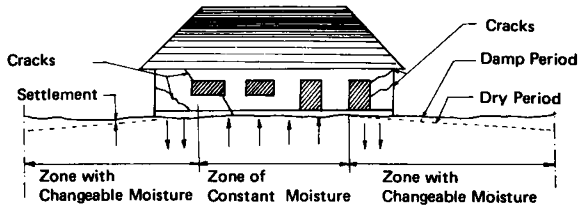


Figure 1 Effect of Variation in Moisture Content of Subgrade

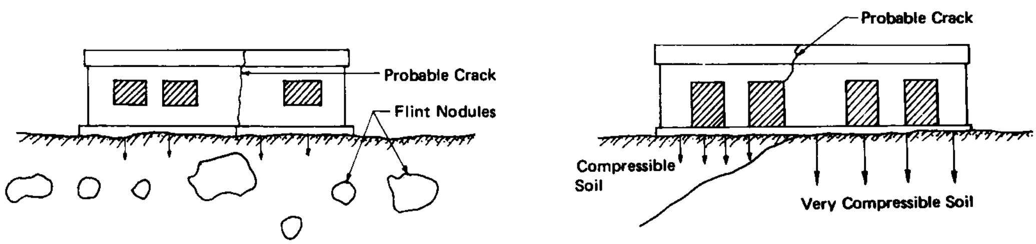


Figure 2 Effect of Variability of Foundation Material

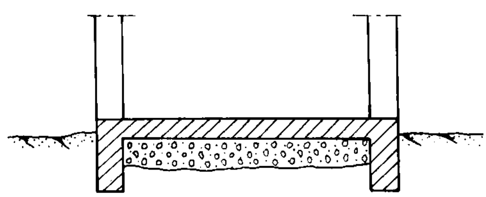


Figure 3 Stone Sub-base

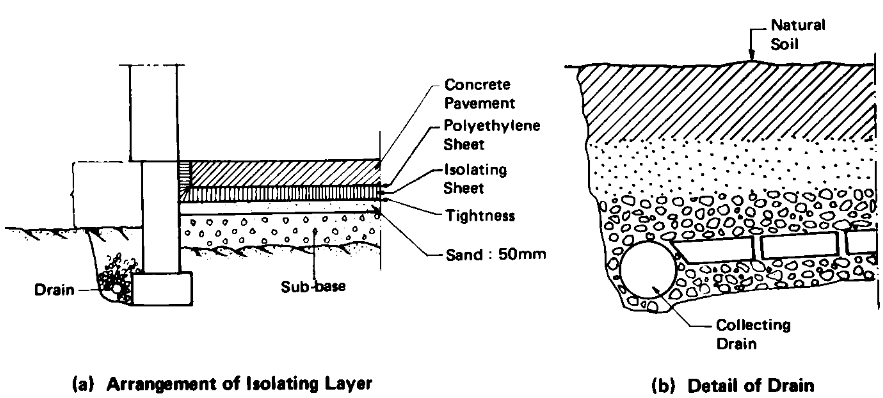
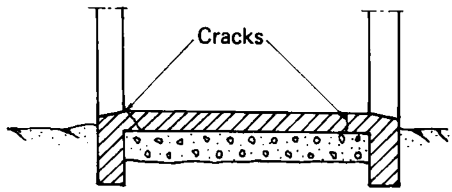


Figure 4 Isolation of Pavement from Bearing Walls



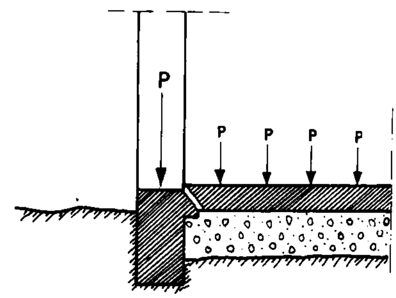


Figure 5 Differential Settlement causing Edge Cracking due to Poorly Compacted Sub-base

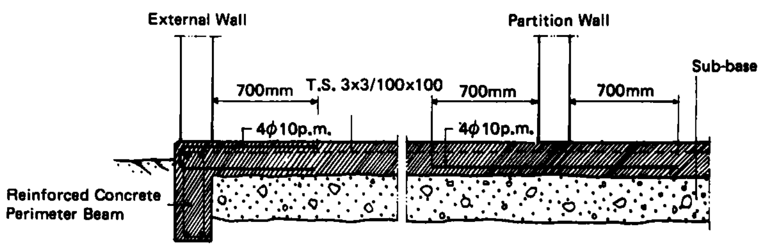


Figure 6 Reinforcement Details near Walls and Partitions

INDUSTRIAL PAVEMENTS

Sub-Base

The decision whether or not to use sub-bases and/or stabilised soils depends upon many factors. The most important of these are the type and quality of the subgrade, the magnitude of the loads, and the allowable tolerance on differential settlements. If the plate bearing test gives a modulus of subgrade reaction less than 30 N/cm^3, the soil should be stabilised. Details on the treatment of subgrade soil are given in the recommendations.

Compaction

All fills, base courses, and sub-bases shall be compacted in layers of about 200 mm depth. Each layer shall be compacted to a minimum density of not less than 95 per cent.

Installation of Joints

Contraction Joints

Contraction joints shall be formed by cutting the surfaces into areas the diagonals of which are less than 7 m for uncovered pavements and 8.50 m for covered pavements. Such joints shall be not less than h/4 thick, see Figure 7.

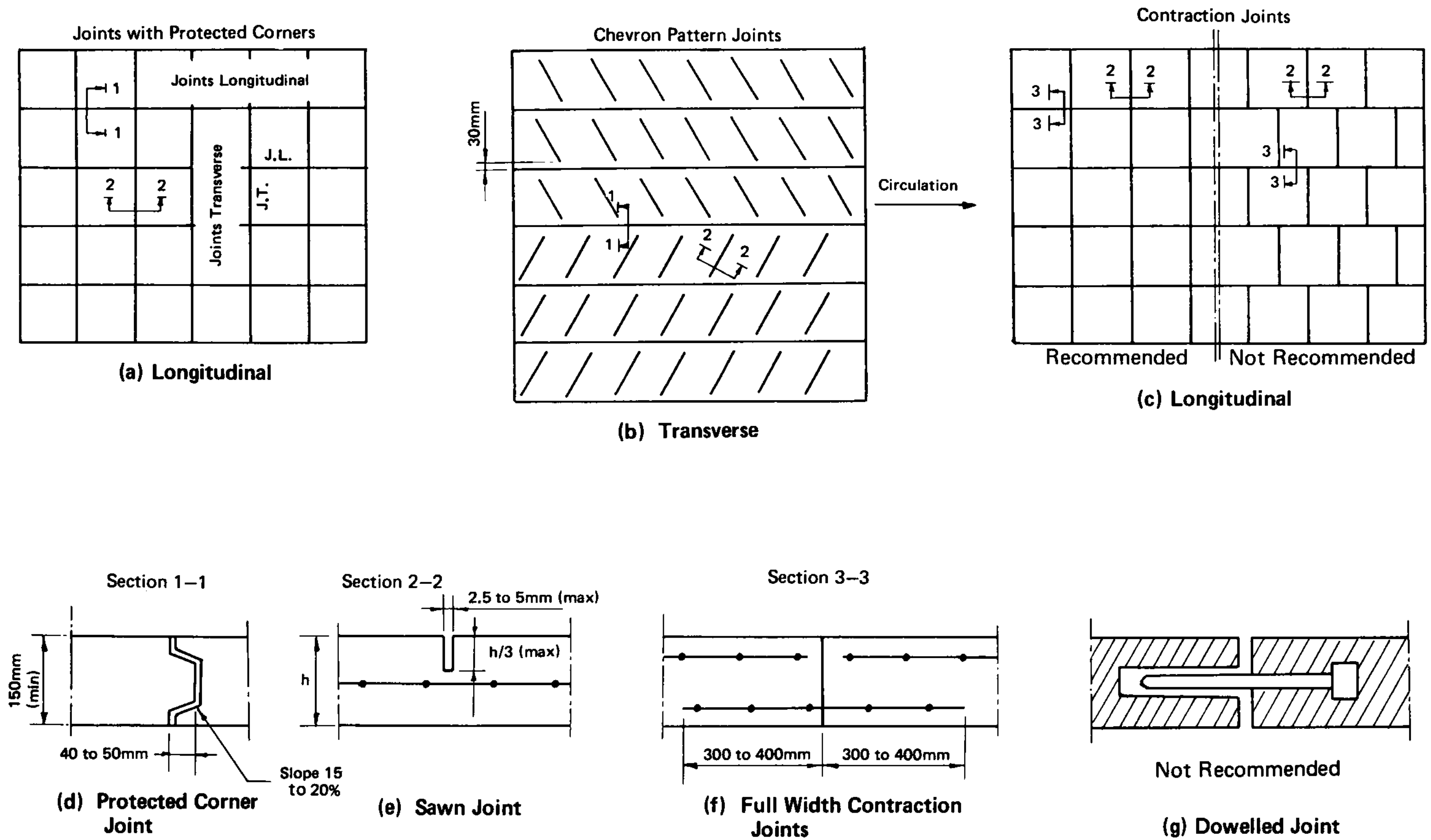


Figure 7 Plans and Sections of Typical Joint Arrangements

Expansion Joints

Expansion joints shall be constructed every 25 m if the pavement is uncovered, see Figure 7.

Construction Joints

Such joints are longitudinal, see Figure 7.

PAVEMENT DESIGN

Non Reinforced Pavements

Allowable Unit Stress in Concrete

The unit stress in the concrete should not exceed 2.4 N/mm^2 for a concrete with 350 kg/m^3 cement content. Warping stresses as well as load stresses must be considered in determining the total unit stress.

Wheel Load

The accepted theory developed by Westergaard leads to the following expressions developed by Pickett for the maximum stress for a corner load:

$$\sigma = \alpha \frac{P}{h^2} \left[1 - \frac{\sqrt{\frac{r}{R}}}{0.925 + 0.22 \frac{r}{R}} \right]$$

with α = 3.36 for protected corners, see Figure 7(d),
α = 4.2 for unprotected corners, see Figure 8 and Figure 9.

$$R = \sqrt[4]{\frac{Eh^3}{12(1 - \nu^2)\ K}}$$

K = modulus of subgrade reaction, see also Table 1 in Appendix.
ν = Poisson's ratio

Uniform Live Load

The maximum stress σ_{max} in daN/cm^2 are given in Appendix Table 2 for B = 5 m, see Figures 10 and 11.

The maximum stress for a variable width B is given in Appendix Table 3.

Settlement

Permissible settlement. The basic assumptions are, see Figure 11, that sections m and n are supposed to be hinged, the magnitude of the live load is 10 kN/m^2 for a width of 1 m in the y direction, the concrete Young's modulus is 12.1 kN/mm^2 and the ultimate tensile strength of the concrete is 2.36 N/mm^2. The allowable settlement in regard to the concrete failure is:

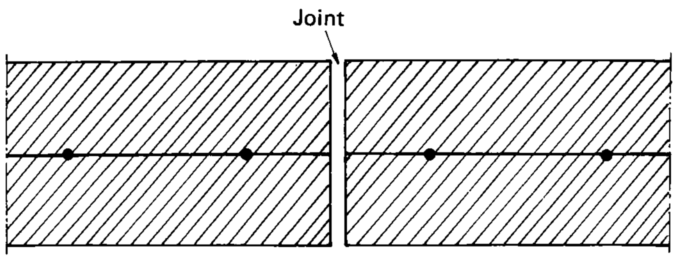


Figure 8 Unprotected Corners — Not Recommended

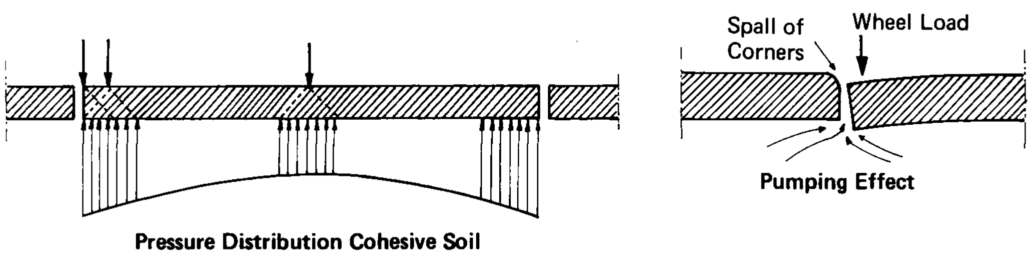


Figure 9 Soil Pressure Distribution, Pumping

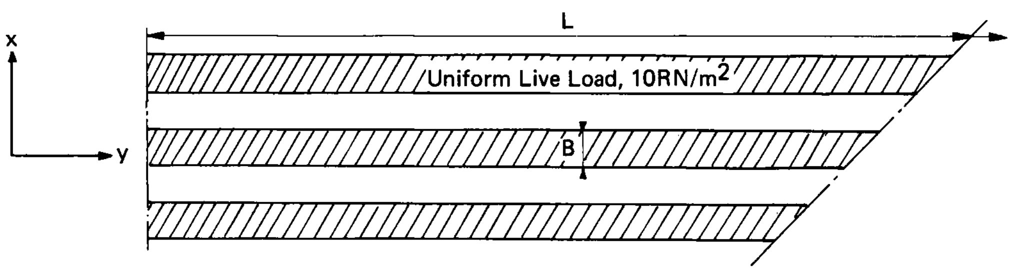


Figure 10 Strip Pattern for Live Load

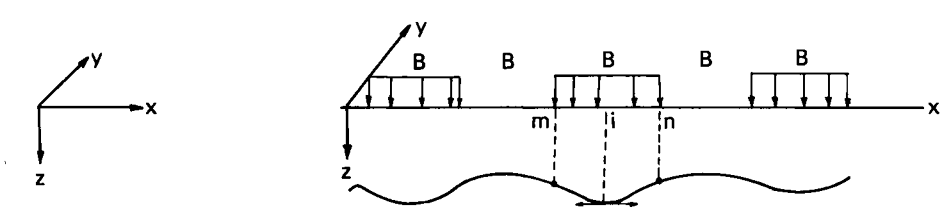


Figure 11 Settlement under Strip Pattern Live Load

$$\bar{\delta}_d = 8.5 \times 10^{-5}\ B^2/h$$

with $\bar{\delta}_d < B/500$ where B is the width of the applied live load, see Figures 10 and 11.

Total settlement. For a 'homogeneous' soil an approximate value of the settlement in the middle of the load strip of width B is:

$$\Delta = 2p/K$$

In the case of a two layer soil, see Figure 12, the settlement in the middle of the loaded strip of width B is:

$$\Delta_H = \frac{H\ \Delta\sigma\ \psi}{E'}$$

where H is the height of the compressible layer,
E' is the Oedometer test coefficient,
$\Delta\sigma$ is the load per unit area,
and ψ is a coefficient given by Appendix Table 4.

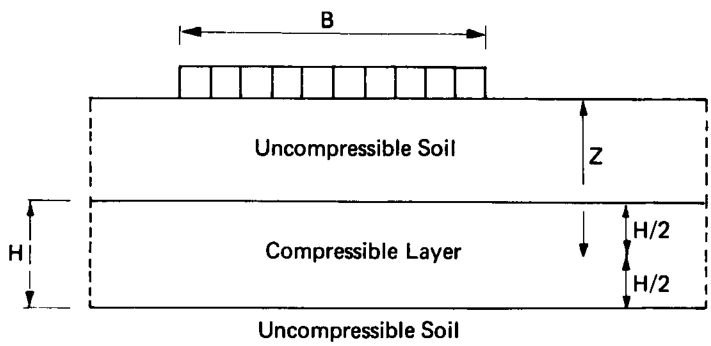


Figure 12 Two Layer Soil

Approximate value of the differential settlement δp. For a 'homogeneous' soil the settlement in section n or m, see Figure 11, is nearly half of that obtained in section i. In the case of a compressible layer the settlement is given by the above expression with the coefficient ψ_d obtained from Appendix Table 4.

Shrinkage reinforcement. The minimum steel reinforcement is given by the expression:

$$A = \frac{0.5\ C_f\ L\ P_o}{\bar{\sigma}_a}$$

where A is the reinforcement section in cm^2/m,
C_f is the coefficient of friction between slab and subgrade, $C_f \simeq 1.5$,
L is the distance between transverse joints
P_o is the weight of the slab,
and $\bar{\sigma}_a$ is the allowable steel stress

Shear force. The allowable load $\bar{P}$ is given by the expression:

$$\bar{P} = \left(\frac{\bar{\sigma}_b\, P_c\, h}{1.25}\right) + \bar{\sigma}_s S$$

where $\bar{\sigma}_b$ is the allowable tensile stress in concrete, = 0.6 N/mm^2,
P_c is the perimeter of the surface situated at h/2 from the periphery of the load surface,
S is the area whose perimeter is P_c,
and $\bar{\sigma}_s$ is the allowable soil pressure.

Reinforced Pavements

If the working stress in the pavement under service loads is more than 2.4 N/mm^2 (for a 350 kg/m^3 cement content), the pavement will be reinforced according to the French Code of Practice CCBA 68.* The minimum reinforcement percentage will be:

$$A_{min} \geqslant 0.15h$$

CONCLUSIONS

Until the full scale test results at St. Remy les Chevreuses (France) (U.T.I.) are available, some professional recommendations have been proposed. These recommendations, which are summarized here, class the pavements in two categories (a) non-reinforced pavement, if the maximum concrete stress is less than 2.4 N/mm^2 and (b) reinforced pavement, if the concrete stress is greater than 2.4 N/mm^2, for a 350 kg/m^3 cement content concrete. Details for joints and steel reinforcement positions are given.

REFERENCES

Guide No. 76-5, Contrôle des dallages en béton sur terre-plein, SOCOTEC, Juin 1976.
Union Nationale de la Maconnerie, Les dallages en béton sur terre-plein, Janvier 1975.
Travaux de Dallages, Regles Professionnelles Provisoires, 1978.

APPENDIX

Table 1, Edge Load Stress

Basic assumptions:

Applied load 10 kN
Unprotected joints
Loaded area : radius of 150 mm
E = 38.0 kN/mm^2
Table 1 gives the maximum stress ($\sigma = \frac{6M}{1.h^2}$) in daN/cm^2 for a unit load of 10 kN.
Enter with K and h and read the stress value.

*CCBA 68 is the equivalent of C.P.110

Reinforced Pavement Design

Basic assumptions:

Applied load 10 kN
Loaded area : radius of 150 mm
E = 38.0 kN/mm^2
Concrete cover : 30 mm
Tables 5 to 10 give the steel reinforcement section, A, in cm^2/m, needed per meter in each direction. Enter with K and h and read A for a unit force of 10 kN. Tables 5 and 6 are for central load, Tables 7 and 8 are for an edge load and Tables 9 and 10 are for uniform live load (E = 12.0 kN/mm^2), see Figures 10 and 11.

Table 1 Maximum Stress for Edge Load

K	MAXIMUM STRESS, daN/cm^2						
kg/cm^3	h,mm = 100	120	140	160	180	200	250
1.5	21.76	15.96	12.22	9.68	7.86	6.52	4.37
5	18.88	14.04	10.88	8.68	7.10	5.92	4.01
10	17.09	12.85	10.03	8.06	6.62	5.54	3.78

Table 2 Maximum Stress for Uniform Live Load on Strip Width of 5 m

K	MAXIMUM STRESS, daN/cm^2			
kg/cm^3	h,mm = 120	150	200	250
1	2.38	2.11	1.98	1.92
5	1.14	1.02	0.83	0.73
10	0.77	0.72	0.62	0.53

Table 3 Maximum Stress for Uniform Load on Variable Width Strip

K	MAXIMUM STRESS, daN/cm^2			
kg/cm^3	h,mm = 120	150	200	250
1	5.34	4.77	4.13	3.69
5	2.40	2.13	1.85	1.65
10	1.69	1.51	1.31	1.17

Table 4 Settlement Coefficients for Two Layer Soil, Total Settlement, ψ, Differential Settlement, ψ_d

$\frac{B}{Z}$	∞	5	4	3	2	1	0.75	0.50	0.25	0.125
ψ	1.00	0.98	0.96	0.92	0.82	0.55	0.44	0.30	0.16	0.08
ψ_d	0.50	0.49	0.47	0.43	0.34	0.14	0.10	0.03	0.01	0.00

Table 5 Required Steel Areas (cm^2/m) for Central Loads, $\bar{\sigma}_a$ = 350 N/mm^2

K	STEEL AREA, cm^2/m						
kg/cm^3	h_o,mm = 100	120	140	160	180	200	250
1.5	0.92	0.77	0.66	0.59	0.52	0.48	0.39
5	0.79	0.67	0.58	0.52	0.46	0.42	0.35
10	0.71	0.61	0.53	0.48	0.43	0.39	0.32

Table 6 Required Steel Areas (cm^2/m) for Central Loads, $\bar{\sigma}_a$ = 280 N/mm^2

K	STEEL AREA, cm^2/m						
kg/cm^3	h,mm = 100	120	140	160	180	200	250
1.5	1.15	0.96	0.83	0.73	0.66	0.60	0.48
5	0.99	0.83	0.73	0.65	0.58	0.53	0.43
10	0.89	0.76	0.67	0.60	0.54	0.49	0.40

Table 7 Required Steel Areas (cm^2/m) for Edge Loads, $\bar{\sigma}_a$ = 350 N/mm^2

K	STEEL AREA, cm^2/m						
kg/cm^3	h,mm = 100	120	140	160	180	200	250
1.5	1.69	1.39	1.18	1.04	0.92	0.83	0.67
5	1.47	1.22	1.05	0.93	0.83	0.76	0.62
10	1.33	1.12	0.97	0.86	0.78	0.71	0.58

Table 8 Required Steel Areas (cm^2/m) for Edge Load, $\bar{\sigma}_a$ = 280 N/mm^2

K	STEEL AREA, cm^2/m						
kg/cm^3	h,mm = 100	120	140	160	180	200	250
1.5	2.11	1.74	1.48	1.30	1.15	1.04	0.84
5	1.83	1.53	1.32	1.16	1.04	0.95	0.77
10	1.66	1.40	1.22	1.08	0.97	0.89	0.73

Table 9 Required Steel Areas (cm^2/m) for Uniform Live Load, $\bar{\sigma}_a$ = 350 N/mm^2

K	STEEL AREA, cm^2/m			
kg/cm^3	h,mm = 120	150	200	250
1	0.424	0.444	0.477	0.514
5	0.188	0.197	0.212	0.229
10	0.132	0.138	0.149	0.161

Table 10 Required Steel Areas (cm^2/m) for Uniform Live Load, $\bar{\sigma}_a$ = 280 N/mm^2

K	STEEL AREA, cm^2/m			
kg/cm^3	h,mm = 120	150	200	250
1	0.532	0.557	0.598	0.644
5	0.236	0.247	0.265	0.286
10	0.165	0.173	0.186	0.201

BEHAVIOUR AND DESIGN OF RESIDENTIAL SLABS ON THE EXPANSIVE CLAYS OF MELBOURNE

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ABSTRACT The principal clay soil types of the Melbourne area of Australia are outlined in the paper. Data from seasonal ground movement stations is used to demonstrate the expansive nature of some of these soils. The behaviour of a series of full size experimental slabs and companion flexible covers, together with a series of actual housing slabs is outlined. A simplified mathematical model is demonstrated to lead to a reasonable correlation between predicted and experimental behaviour. The results of all the above studies are combined to yield a practical design procedure for slabs on expansive clays in Melbourne.

INTRODUCTION

In the eastern states of Australia, the expansive nature of the clay soils and the prevailing climatic conditions lead to the development of significant seasonal soil movements in the top few metres of soil. When light domestic structures are constructed on these clay soils, foundation failures of all but slabs with integral stiffening beams are relatively common. The generally superior performance of slabs over the more conventional strip footing foundation systems for housing was first recognised in the early 1970's.

In about 1972, studies were commenced at C.S.I.R.O. (an Australian Government research organisation) and Swinburne College of Technology, aimed at developing a more rational and economical design method for house foundation slabs on the expansive clay soils of Australia. The studies at both of these institutions have included performance and construction surveys and laboratory and field material testing. Extensive mathematical modelling has been carried out at C.S.I.R.O., whilst a major field study of the seasonal ground movements of Melbourne clay soils and the behaviour of full size experimental and actual housing slabs is being conducted at Swinburne. The results of these studies have been used to develop two simple standard slab designs for the expansive clays of Melbourne. The designs have recently been incorporated into the building regulations of this city of about three million people.

This paper will briefly present some of the major aspects and conclusions of both these parallel studies. It will also show how these studies have been coalesced to yield the standard slab designs for Melbourne's building regulations.

SEASONAL HEAVE OF MELBOURNE CLAY SOILS

The major residual clay soils of the greater Melbourne area can be identified and classified according to their geological origin. These clay soils can be divided into two groups: i) the Quaternary basaltic clays and ii) the Tertiary to Ordovician clays. The distribution of these two clay soils throughout the greater Melbourne area, together with the locations of test sites which will be discussed later in this paper, is shown in Figure 1.

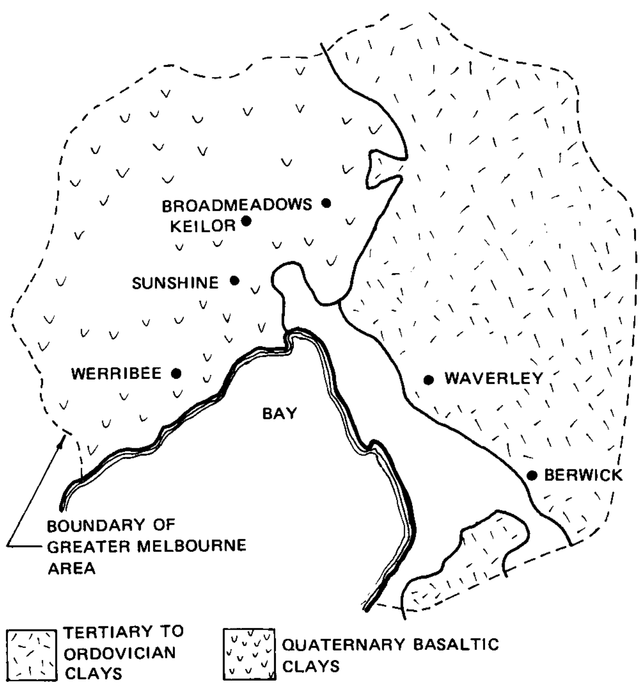


Figure 1 Distribution of Clay Soils Throughout Melbourne

The soil profile and range of linear shrinkage values for each of these two clay soil groups is presented in Figure 2.

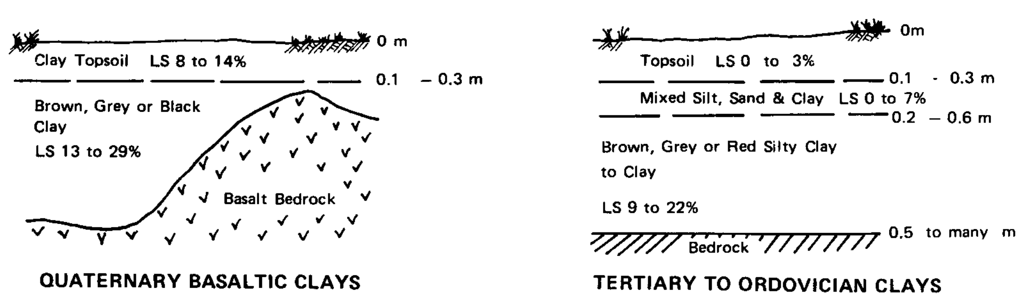


Figure 2 Soil Profile of Melbourne Clay Soils

Due to Melbourne's semi-arid climatic pattern, these clay soils are subjected over their first few metres depth to seasonal soil moisture changes which can lead to cyclic volume changes and vertical movements. The vertical movement from a clay soil's seasonally driest to wettest states may be defined as seasonal heave. The seasonal heave at a particular clay soil site, and the depth to which it occurs, depends mainly on the clay type, the soil profile, the weather pattern and the site drainage. Many engineers, however, consider only the clay type when attempting to estimate the seasonal heave of a particular site, which generally leads to gross over-estimation of movements. They are, in fact, determining only the potential for swell of a particular clay from either simple linear shrinkage or plasticity index soil tests.

From the linear shrinkage values given in Figure 2, it is apparent that high potential volume change is likely in most Melbourne clay soils. This does not, however, mean that high seasonal heaves will occur right across Melbourne since there is a distinct difference in the soil profile for the two general soil groups. The Quaternary basaltic clays occur from the surface down while the other clays are overlaid by two non-expansive soil horizons which tend to insulate the underlying material from gross soil moisture changes and so appreciably reduce the seasonal heave of these soils. In fact, from the results of fourteen ground movement stations that have been established throughout the developing areas of Melbourne, the range of surface seasonal heave in Quaternary basaltic clays is from 24 to 65 mm, while for the other clays the range is only from 6 to 33 mm. On the average, therefore, the Quaternary basaltic clays heave about twice as much as the other clays.

The effect of site drainage on seasonal heave can be readily appreciated by considering the movements observed at test sites at Sunshine and Keilor, see Figure 3, both of which are located in Quaternary basaltic clay areas. Rainfall and evaporation patterns are closely similar and the soil profiles are practically identical. The Keilor site, however, is located in a well drained area of the international airport, while the Sunshine site is flat, open, poorly drained and becomes waterlogged during the wetter winter and spring months. Because of this difference in site drainage the seasonal heave at Keilor is only about one half of that at Sunshine.

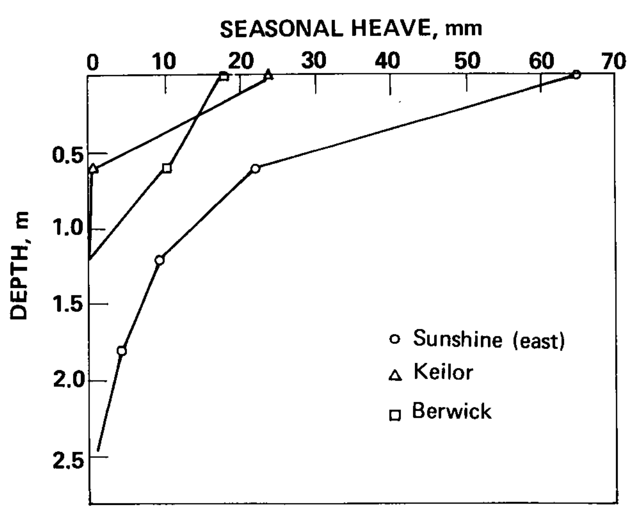


Figure 3 Seasonal Heave Versus Depth Of Melbourne Clays

Most of the seasonal heave in the Quaternary basaltic clays occurs in the top 1.5 metres, whereas in all the other clay soil types most of the movement occurs in the top 1 metre.

MODIFICATION OF SEASONAL HEAVE BY SLABS

When an impermeable surface cover, or slab, is placed on an expansive clay, the seasonal soil moisture change pattern will be altered since surface evaporation from the clay will be terminated, see Figure 4(a). If the site is very dry when a completely flexible slab is placed on it, edge wetting and heave of the underlying clay will lead initially to the development of an edge heave or saucer type of slab distortion mode, Figure 4(b). With time the heave under the slab will slowly progress inwards, Figure 4(c) & (d), until ultimately a centre heave or mound distortion mode will form under the slab, Figure 4(e).

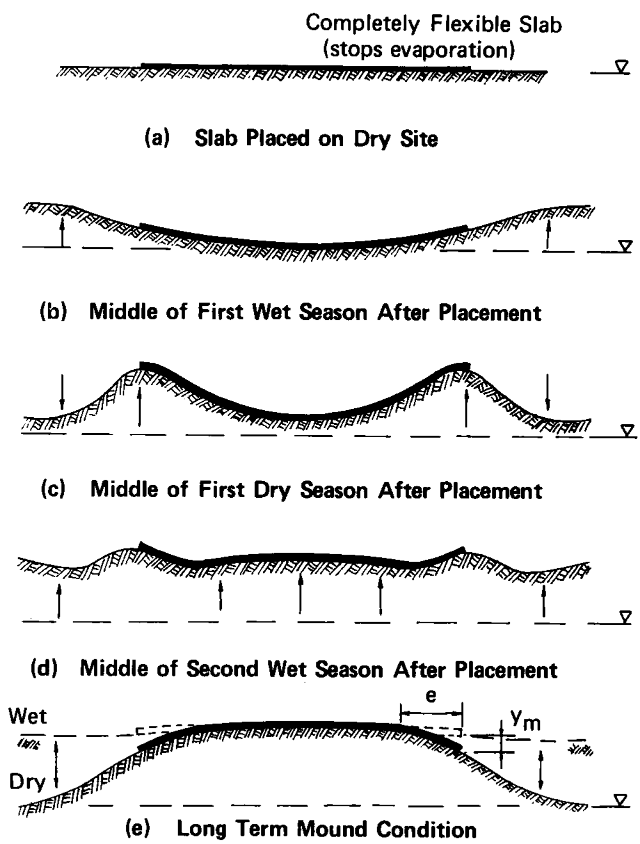


Figure 4 Idealised Mound Development

In the long term, the soil surface adjacent to the edges of the slab will continue to move up and down with the seasons, so leading to flexing of the edges over a length which is commonly referred to as the edge distance, e, see Figure 4(e). However, if the slab is sufficiently rigid it will not flex, and distress of the superstructure will not occur. The economic design of an actual house slab consists of making it sufficiently stiff so that any deflection of the slab, due to mound development or due to seasonal wetting and drying under the slab edges, will not lead to distortion of the house superstructure sufficient to cause cracking of brick walls or jamming of doors or windows.

If a slab is placed on a very wet site, a mound will effectively exist, so only seasonal wetting and drying of the clay under the slab edges will need to be accommodated by the slab.

EXPERIMENTAL SLABS STUDIES

Over the last six years, six experimental slabs, seven actual housing slabs and eight completely flexible surface covers have been monitored at a number of clay soil sites throughout Melbourne. Most of the sites are located in Quaternary basaltic clay areas.

The experimental slabs were all extensively instrumented with precise levelling points, pressure cells and strain gauges. They are located at Sunshine in a Quaternary basaltic clay area and at Waverley in a Tertiary clay area. All the slabs have been loaded with pig-iron to simulate single storey brick veneer house construction. Generally, the design details of these slabs, which were either 14 by 7.4 m or 7.4 m square in plan, were appreciably lighter than those commonly used in Melbourne.

The seven actual housing slabs were mainly located in the Quaternary basaltic clay areas and had slightly stronger slab sections than the experimental slabs. They have been instrumented with a limited number of precise levelling points to enable the shape distortion pattern with time to be observed under real housing conditions.

The completely flexible surface covers consist of black plastic ground sheets protected from sunlight by aluminium sheeting. Some of the covers had their edges embedded into the underlying clay by various amounts to simulate slab edge beams. No loading was applied to these covers. Precise levelling points were established on them and in the surrounding clay.

Since the behaviour of the experimental slabs and covers in the Tertiary to Ordovician clays of Melbourne was found to lead to far less severe design conditions than those observed in the Quaternary basaltic clays, the results from these sites will not be discussed.

At the Sunshine site, where seasonal heaves of up to 65 mm have been observed, four experimental slabs have been constructed. Only the behaviour of the lighter two of these slabs will be outlined. Design details of both slabs are given in Figure 5.

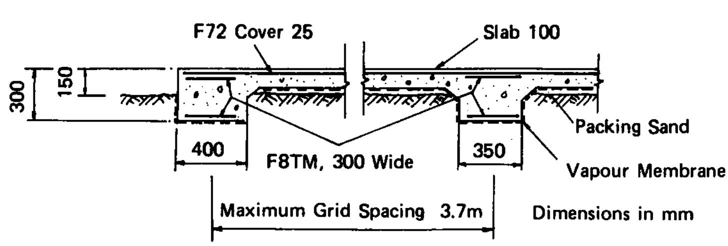


Figure 5 Details of Sunshine S1 and S3 Slabs

One of these slabs (S1) was constructed in October 1974, when the Sunshine site was in its seasonally wettest state, while the other (S3) was placed when the site was very dry, heavily fissured, and in its seasonally driest site.

Plots of the average slab movements and the surface clay movement against time for slabs S1 & S3 are presented in Figure 6. The effect of initial seasonal moisture conditions is strikingly evident in this Figure; the slab placed on a site that was seasonably wet shows very little movement. Figure 7 demonstrates the seasonal flexing of the edges of slab S3. Even with this significant flexing, measurements at the slab clay interface showed that edge lift-off of about 5 mm occurred.

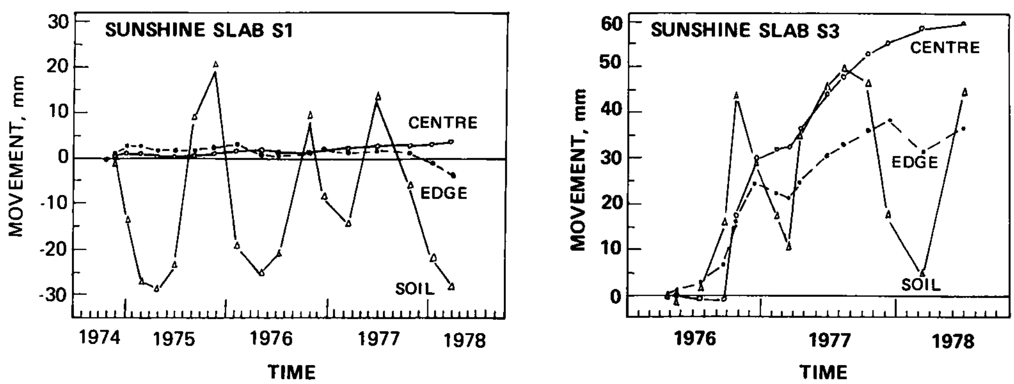


Figure 6 Sunshine Slab and Soil Movement Time Plots

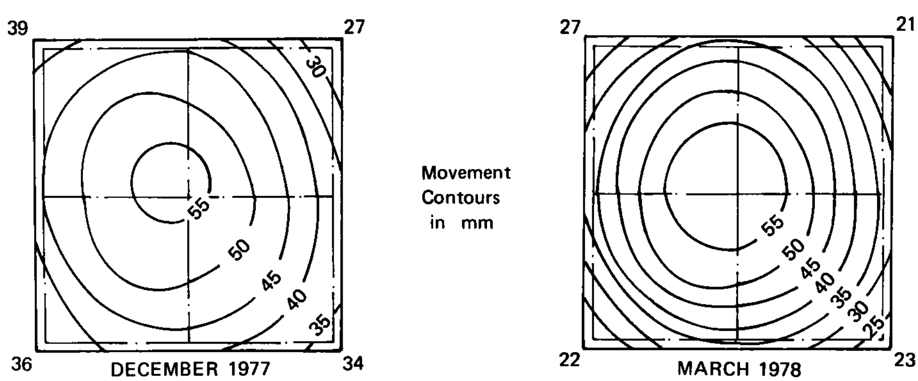


Figure 7 Seasonal Distortion of Slab S3

It should be appreciated that the moisture conditions at this site are quite extreme; regular flooding in winter and extremely dry conditions in summer with surface cracks of 75 mm can be expected. To investigate the influence of more normal site conditions a series of actual house slabs have been monitored. Only the behaviour of two of these housing slabs, in the same general clay area as Sunshine, will be discussed in detail. The slab designs used for both of these houses were very similar to that of Sunshine S3 slab, but were slightly stiffer, with 375 to 400 mm deep beams.

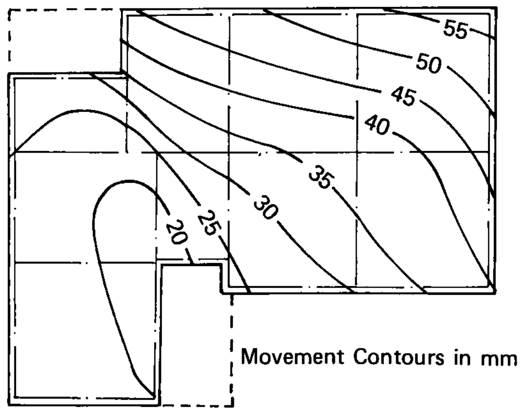


Figure 8 Distortion of Werribee House Slab

The behaviour of the Werribee house slab is very interesting. Over the last two years the slab has slowly developed a pronounced edge heave distortion mode, Figure 8. Those edges of the slab toward which surface water drained, were observed to heave more rapidly than the other edges. No edge drying effects have occurred during the last two summers, probably because of the well-watered lawns which surround the slab. In contrast, the large irregularly-shaped slab at Broadmeadows quickly developed a centre heave distortion pattern, with some minor local edge drying, see Figure 9.

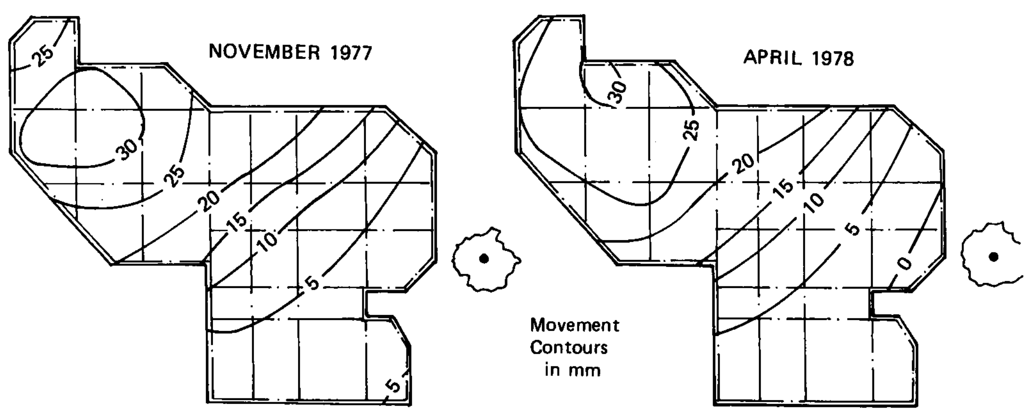


Figure 9 Distortion of Broadmeadows House Slab

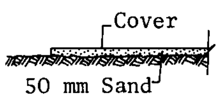
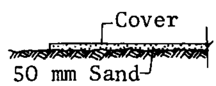
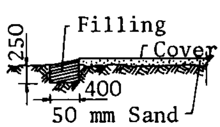
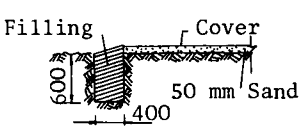
Slab settlements occurred near a large gum tree located only a few metres from one corner and again more rapid heave of the slab occurred where surface water drained towards it. It is also interesting to note the much smaller slab heave in the lower right hand side of the house, see Figure 9. The site fall across the house required the building up of this area under the slab with rock filling. The weight of this filling has restrained the underlying clay, resulting in the much lower slab heaves observed.

Measured deflections of both these slabs indicate that edge distances occurring at these two well drained sites do not exceed one metre. The houses at Werribee and Broadmeadows have not suffered any structural distress as a result of the slab movements.

Three housing slabs located in Quaternary basaltic clays similar to Sunshine, but placed on wet sites, have been subjected to only a minor level of underlying uneven clay heave. This behaviour further demonstrates that slabs placed on drier sites are subjected to more severe design conditions.

As already indicated, a series of completely flexible surface covers have been placed at Sunshine and Waverley to obtain information about the unloaded mound development. Details of the four covers at Sunshine, with its far more severe site conditions, together with the mound parameters e and y_m measured from them, are given in Table 1.

Table 1 Sunshine Cover Details

COVER NUMBER	PLACEMENT DETAILS	COVER DIMENSIONS AND DETAILS	MAXIMUM EDGE DISTANCE 'e', m		DIFFERENTIAL HEAVE 'y_m' mm
			Long.	Trans.	
SC1	October 1974 (wet site)	 14 m x 7 m	2.5	1.5	42
SC2	March 1978 (dry site)	 5.8 m x 5.8 m	-	-	-
SC3	April 1976 (dry site)	 5.8 m x 5.8 m	2.1	2.1	32
SC4	April 1976 (dry site)	 5.8 m x 5.8 m	2.1	2.2	47

Notes: 1. The cover is an impermeable membrane consisting of polythene sheet and a layer of reflective insulation material.

2. Beam filling consists of compacted clay.

SOIL-STRUCTURE INTERACTION MODEL

To improve the understanding of slab behaviour a mathematical model was developed to represent the interaction between the slab and the movements of the underlying expansive soils. One purpose of such a model is to produce a rational design procedure. Since the material properties cannot be found with exactitude, excessive refinement of the model is not warranted. To achieve the aim of a simplified model the following assumptions were made:

i) the slab can be represented as a beam on a foundation subjected to vertical movements only

ii) the foundation can be represented as a coupled Winkler system related not to the elastic behaviour, but to the swell behaviour of the soil

The proposed model is indubitably a development of the work of Lytton (1). However, significant changes to the representation of the ground movement, the clay swell-pressure relationship and importantly, the influence of slab deflections on the interaction, lead to an improved, more realistic model.

For simplicity, the model will be described with reference to centre heave, although edge heave can also be treated. As described previously, a completely flexible slab would eventually deform into a centre heave condition which can be characterised by the parameters y_m and e, see Figure 4. Moreover, the weight and stiffness of the slab interact with the swelling of the soil to moderate this movement. To represent this interaction, the swell-pressure behaviour of the soil needs to be described. It has been shown that the swell-pressure behaviour of the foundation can be represented by the bilinear system shown in Figure 10. This material behaviour is then incorporated into a Coupled Winkler representation of the foundation, see Figure 11.

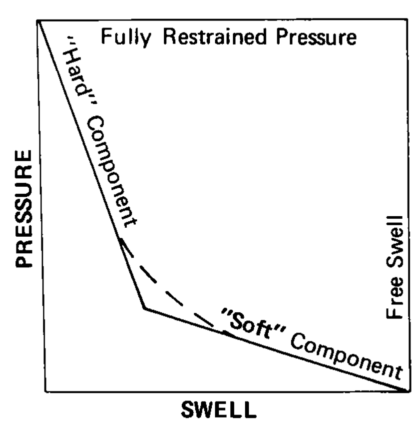


Figure 10 Soil Swell Model

The analysis of a beam-on-mound in accordance with the above assumptions was accomplished with the aid of a computer program. The beam was represented by short segments and finite-element theory was used to determine the segments' stiffness including the foundation stiffness. Iterations, in the analysis, were needed to determine the lift-off points, if any, in the system from both the hard and soft components of the foundation idealisation. A further feature was that the effective moment of inertia, for moments in excess of the cracking capacity, was computed using the approach of Branson (2). The dependence of the effective moment on the bending moment required extra iterations in the solution.

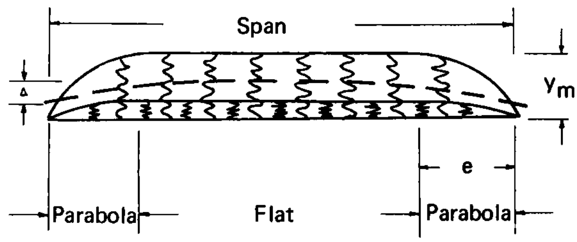


Figure 11 Mound Representation

The accuracy of the proposed model was verified by comparison of its predictions with the experimental results quoted previously. In this comparison, the data adopted are given in Table 2 along with the theoretical and experimental results. As can be seen from this Table, the comparison substantiates the use of the model, particularly for the more critical case where initially a site is in its seasonally driest state (e.g. S3 & S4). This model has been further developed into a design procedure based on non-dimensional soil structure parameters.

Table 2 Comparison of Experimental and Theoretical Maximum Deflection to Span Ratio (Δ/L)

SUNSHINE SLAB NUMBER	MOUND SHAPE		LONG TERM E_c	MAXIMUM CENTRAL Δ/L	
	y_m mm	e mm	kN/mm^2	Field	Predicted
S1	34	2200	16	1/1320	1/690
S2$^+$	38	2100	16	1/3110	1/1200
S3	34	2200	14	1/310	1/390
S4^{++}	30	2500	14	1/340	1/320

$^+$ Similar slab design to S1 but with 500 mm deep beams
$^{++}$ Same slab design as S1 but lifted up on 150 mm of compacted rock filling

PRACTICAL IMPLEMENTATION

The results of this research have had a very real impact on foundation practice in Victoria and to a lesser extent in other Australian states.

The most common form of house construction in Victoria is a timber framing with a single leaf external brick veneer. In the past, conventional footings consisted of a reinforced concrete strip 250 mm deep by 375 mm wide founded at a depth of 450 mm under the external walls, with timber or concrete point footings under the internal framing. The subfloor space is ventilated but the ground is not sealed by a membrane or concrete. The performance of this footing system was often poor. Drying of the subfloor frequently caused shrinkage settlements under the internal areas and moisture changes due to trees and seasonal effects distorted the external walls, particularly near corners. The need for improved footings was given added impetus by the introduction of a six year guarantee of structural adequacy of houses.

On the other hand, the alternative of a concrete slab foundation was regarded with suspicion by the building authorities. In the absence of guidance from the Uniform Building Regulations, many municipalities, of which there are approximately 45 in Melbourne, developed their own approach. Normally, laboratory soil testing and an engineered design were required. In some areas, slabs were virtually prohibited. Frequently very expensive designs were employed. This conservatism occurred despite the virtual absence of any slab failures.

These problems resulted in one of the co-authors being invited to join a committee to review the building regulations dealing with foundations. As a result of this committee's work, the recommendations of the authors were adopted and the current practice can be summarised as follows:

i) Each site is classified on the basis of only its geological origin and soil profile into 'stable' or 'intermediate'. Generally only the Quaternary basaltic clays come into the higher category.

ii) A standard slab system is then selected from those shown in Figure 12. No engineering computations are expected.

A feature of the new system is that lighter slabs are recommended for the majority of clay types and only a lightly stiffened slab for the basaltic clays. Stronger and deeper standard strip systems have also been developed.

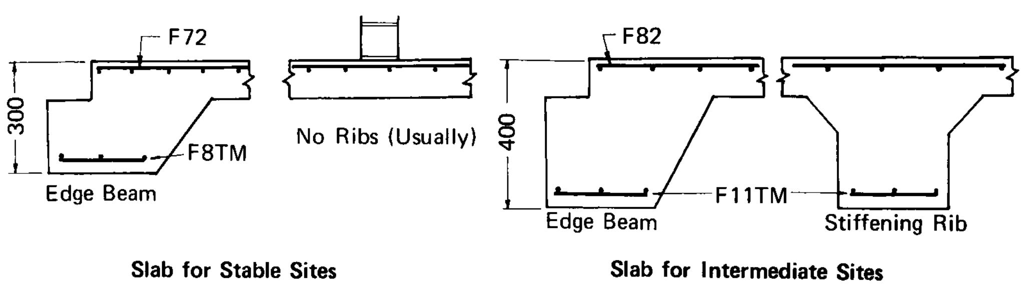


Figure 12 Standard Slab Details

CONCLUSIONS

Slabs with integral stiffening beams have been shown to perform satisfactorily on expansive soils in Melbourne, where seasonal ground movements of up to 65 mm have been recorded.

Slabs placed on highly expansive clay sites, which are in their seasonally driest condition, are subjected to much greater underlying clay heave and therefore need to be significantly stiffer than those placed on wet sites.

For high seasonal heave clays, it is essential to grade the surface surrounding the slabs so as to ensure surface runoff does not pond against the slabs. Ponding is likely to result in high localised slab heave.

A simplified mathematical model is demonstrated to lead to a reasonable correlation between predicted and experimental behaviour.

Finally, the results of studies reported have been coalesced to yield a practical design procedure for slabs on expansive clays in Melbourne. More detailed information relating to these studies is given in references 3, 4 & 5.

ACKNOWLEDGEMENTS This research study at Swinburne would not have been possible without the generous financial support of the Australian Engineering and Building Industries Research Association Ltd. The substantial contributions to the Swinburne studies by the following postgraduate students: Messrs A Crichton, J Washusen, D Cameron, J Jackson, B Pitt, D Cimino and C Lawrance are acknowledged.

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THE DESIGN OF GROUND BEARING SLABS IN WAREHOUSE CONSTRUCTION

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ABSTRACT Consideration is given to the analysis and design of ground bearing warehouse slabs, in particular the analysis of the slab of a new warehouse whose performance has been monitored. Computed results obtained from a truly three-dimensional analysis utilising plate bending finite elements and a layered continuum model for the soil are compared with those given by the conventional beam on an elastic foundation (Winkler Spring) analogy. Details of the soil strata, load distribution, etc., are given and a comparison made between computed and measured deflections of the slab. The importance of the various design parameters is assessed and some tentative recommendations are given with regard to warehouse slabs in general.

INTRODUCTION

Whilst the design and analysis of raft foundations in general has received considerable attention in recent years (1,2), comparatively little attention has been focused on ground bearing warehouse slabs in particular (3). The construction of the latter is characterised by the use of relatively thin nominally reinforced concrete slabs (125 to 275 mm thick) of large extent, resting on compressible alluvial soil deposits of limited thickness. In addition, the method of construction of the slabs, and the avoidance of shrinkage cracking, necessitates the provision of numerous joints which may or may not be capable of transmitting shear forces.

The results obtained from monitoring the settlement performance of the ground slab of a typical single-storey warehouse structure are discussed in relation to independent predictions. The theoretical basis of the two methods based upon continuum and Winkler soil models, are outlined and the differences discussed in the light of the results obtained. Structural details of the warehouse and the soil properties revealed by a site investigation are summarised.

NOTATION

a, b	Dimensions of typical slab strip;
C_v, m_v	Coefficients of consolidation and volume compressibility;
E_c, ν_c	Young's modulus and Poisson's ratio concrete;
E_s, E', E_u	Young's moduli soil, fully drained, undrained;
ν_s, ν', ν_u	Poisson's ratios soil, fully drained, undrained;

k Coefficient of subgrade reaction;
K Relative stiffness factor;
M Moment intensity, sagging +, hogging -;
q Load intensity;
ρ Settlement.

THE WAREHOUSE

The warehouses, located in South East London, were constructed in 1977/78 and consist of two blocks, A and B. Block B, which is the subject of this paper, consists of four identical units, each approximately 37 x 24 m with a clear height to underside of the roof truss of 6.4 m. The warehouses, built as a speculative venture, were intended for general purpose storage without prior knowledge of the precise nature of the floor loading.

Structural Details

The structure consists of precast concrete framing with open web steel lattice girder roof beams and in-situ reinforced concrete ground beams supporting brick and blockwork internal and external walls. The mass concrete bases are founded in gravel at a depth of 4.5 m. The 178 mm thick concrete (characteristic strength 30 N/mm^2) ground slab, reinforced with an A252 top mesh, was cast in strips and separated from the structure by flexible filler. In the analysis the isotropic elastic moduli of the concrete have been taken as E_c = 15 kN/mm^2 and ν_c = 0.15.

Soil Data

The soil succession, as revealed by the site investigation, is shown in Figure 1.

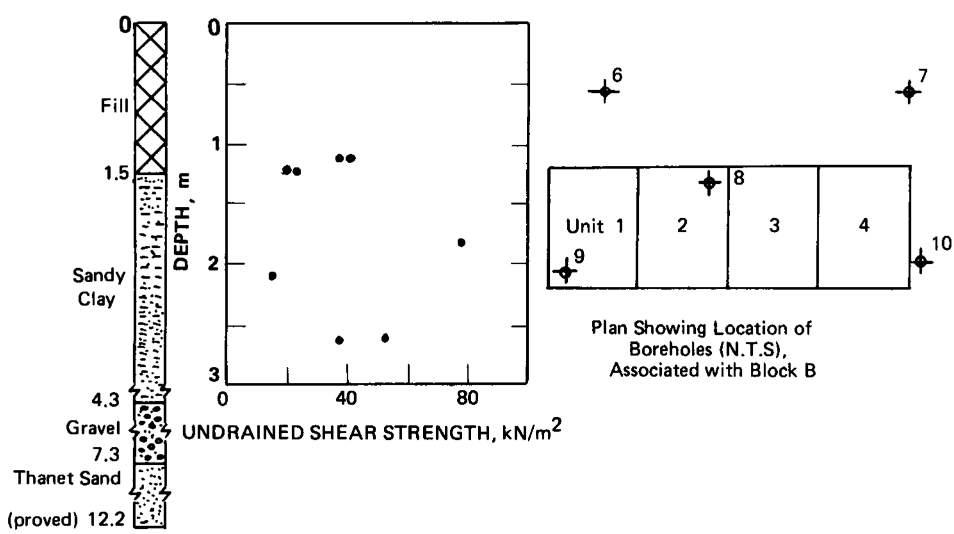

Figure 1 Soil Data

A total of ten boreholes were sunk using shell and auger equipment, the records of five of these boreholes, numbers 6 to 10, being relevant to the behaviour of Block B. Standard penetration tests carried out within the flood plain gravel and Thanet sand indicated that these materials varied from dense to very dense. The results of undrained shear strength determinations obtained from triaxial compression tests on sets of three 38 mm diameter specimens cut from 102 mm diameter samples are shown in Figure 1. Consolidation tests carried out on 76 mm diameter specimens of the clay, loaded in the oedometer over the pressure range 25 to 50 kN/m^2 produced average values of the coefficients of volume compressibility, m_v, and consolidation, C_v, of 0.335 m^2/MN and 0.7 m^2/year, respectively.

In determining the values of elastic moduli for the soil in order to conduct the analysis, the high compressibility of the clay layer has been assumed dominant. Two situations have been examined. First, the short term undrained behaviour with Poisson's ratio, ν_u, equal to 0.5 and second, the long term fully drained condition with $\nu' = 0$. From isotropic elastic theory when $\nu' = 0$, E' is equal to $1/m_v$ and the undrained modulus $E_u = 1.5 E'$. The values of the elastic moduli are summarised in Table 1.

Table 1 Soil Properties

MATERIAL	THICKNESS m	UNDRAINED E_u, N/mm^2	UNDRAINED ν_u	FULLY DRAINED E', N/mm^2	FULLY DRAINED ν'
Fill*	1.5	-	-	50.0	0.1
Clay	2.8	4.5	0.5	3.0	0.0
Gravel and Thanet Sand		Assumed incompressible			

* assumed fully drained at all times

Details of Monitoring

Shortly after completion Block B was used to store rolls of newsprint and although the distribution and movement of this load was outside the control of the authors, agreement was obtained to monitor the settlement of the ground slabs. Level readings on a grid of stations, marked on the slabs with paint, were taken using a Zeiss automatic level, capable of reading to 0.05 mm, and a specially graduated staff. The readings were referred to a series of permanent bench marks which had been established at the junctions of the columns and ground beams. The accuracy to which the level of a given point could be measured was determined with a 98 per cent probability to be $\pm$ 0.5 mm. However, it should be noted that due to the variable load distribution, it was not always possible to relocate all the stations during a survey.

THEORETICAL APPROACHES

Two distinct theoretical models for the soil are outlined and the results generated from each compared.

Elastic Continuum

In this approach the soil has been modelled as an isotropic layered elastic continuum of finite depth (4), with the additional assumption that the variation of

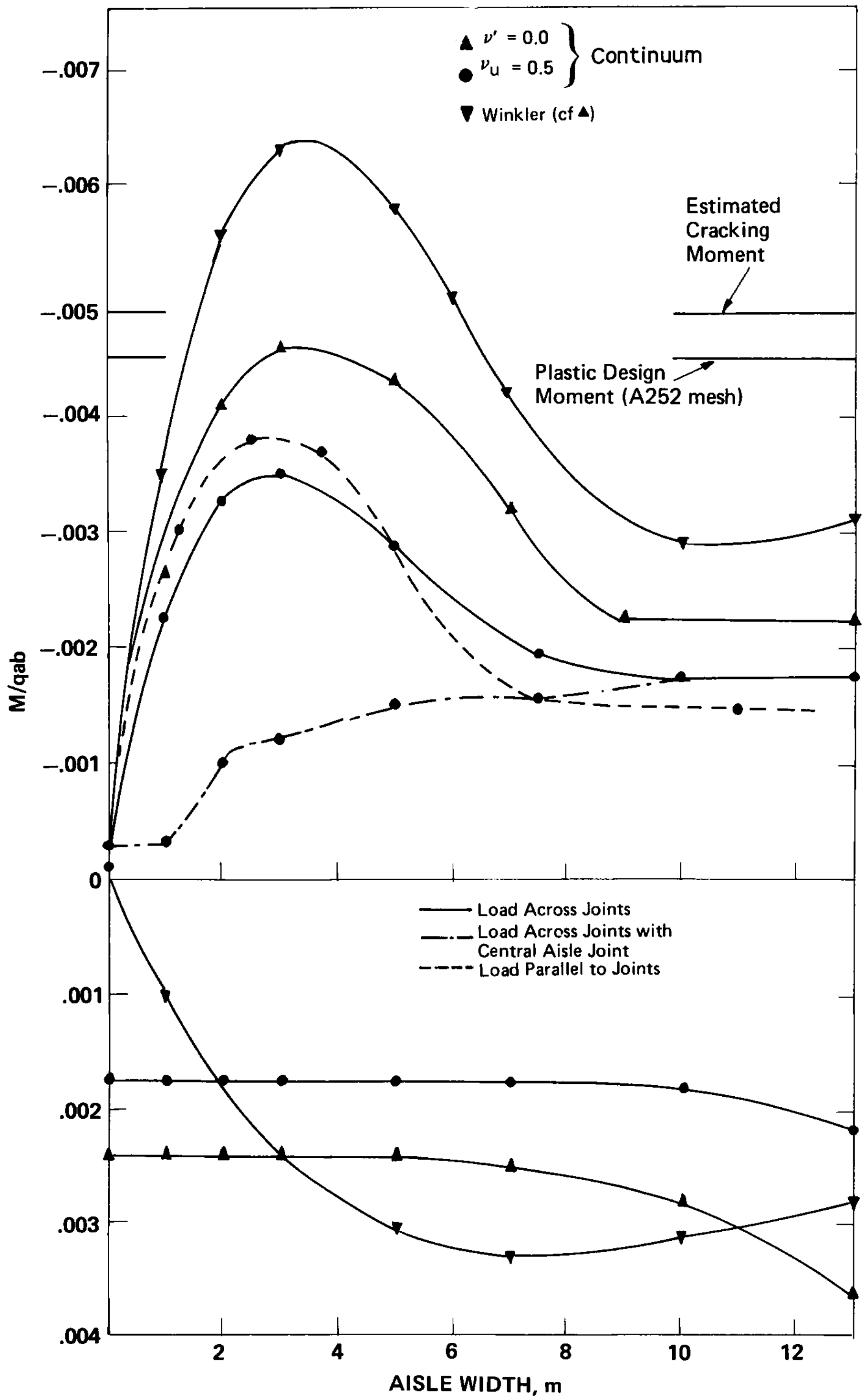

Figure 2 Variation of Moment with Aisle Width

stress within the continuum, due to the initially unknown ground reactions at the slab-soil interface, is identical to that for a homogeneous, half-space. The acceptability of the latter constraint has been demonstrated (2,5), and for the particular geometry considered, preliminary results for a 25 m square uniformly loaded slab yielded computed total and differential settlements within 10 per cent of the exact values (1). The slab has been modelled using plate bending finite elements (6) and the interface taken as smooth. Joints within the slab have been modelled as complete breaks with no moment or shear connection.

Beam on Elastic Foundation - Winkler

In this well established method (7) the soil is modelled as a bed of independent linear springs. For warehouses the slab is usually represented as one-way spanning beam strips (8), as adopted here, although plate bending finite elements could be used (9). The spring stiffness, k, termed the coefficient of subgrade reaction, is not an independent soil parameter but as stated by Terzaghi (10), is also dependent upon the structural form. Although a number of relationships between the coefficient of subgrade reaction and the corresponding elastic moduli have been suggested (11,12), it is clear that any such relationship can at best be only approximate. However, there are clear computational advantages in using the Winkler approach and for this reason it was considered worthy of investigation.

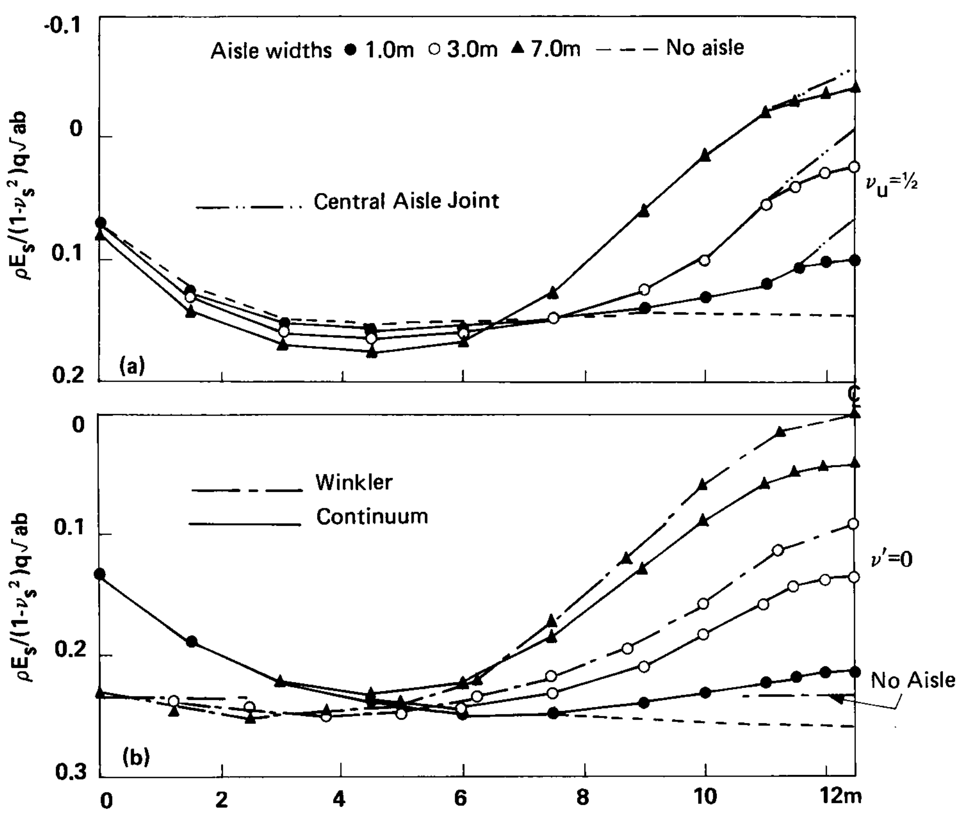

Figure 3 Settlement Profiles

Comparison Between Continuum and Winkler

In order to obtain comparable results the value of k employed in the Winkler model has been adjusted to produce almost identical central deflections to those obtained from the continuum model for a uniformly loaded slab. In Figure 2, the variation of maximum moment with aisle width is shown. In general, the Winkler approach yields greater hogging moments than the continuum model, due in part to the anti-clastic behaviour of the slab in the latter. Less acceptable agreement has been obtained for the sagging moments and these should be examined in conjunction with the settlement profiles shown in Figure 3(b).

For a fully loaded slab the Winkler model predicts uniform settlement and zero moments, in contrast to the dish-shaped deformation of the continuum approach. Indeed, the Winkler model continues to predict almost uniform settlement under the load when an aisle is introduced at the centre of the slab. Sagging moments predicted by the former are therefore small and increase only as the aisle width increases to give better agreement with those obtained from the latter. However, reasonable agreement between the two methods for both moments and settlements is found in the region of the aisle.

COMPARISON OF MEASURED AND COMPUTED SETTLEMENTS

Initial level readings were taken in Units 2 and 3 prior to any loading, whereas in Unit 1, the readings were obtained whilst the load was being placed. Two sets of readings have been compared with the computed values; namely after one month and after six months. The first set are thought to be representative of the immediate, undrained behaviour of the clay and the second set of the settlement at 50 per cent consolidation, based upon a value for C_v of 0.7 m^2/year with the computed settlements factored accordingly. As stated earlier, it was not possible to control the load distribution, however, over the first month the load remained sensibly constant.

Results for one month for Units 1, 2 and 3 are shown in Figure 4. The continuum model exhibits the same basic profile as the measured values but generally over predicts the settlement under the load, with a small under prediction of the heave away from the load.

The Winkler model indicates almost constant settlement under the load when the load is at one end as in Figure 4(a) and (c), with negligible heave away from the load. When the load is slightly away from the end of the slab the Winkler model results agree closely with the continuum approach beneath the load but again predict negligible heave away from the load.

Comparison between the measured and computed settlements at six months for three points in Unit 2 are given in Table 2, the loading plan being shown in Figure 5, and show good agreement for the continuum model but poor agreement for the Winkler model. More detailed settlement profiles could not be obtained owing to the loss of several stations.

Table 2 Settlements (mm) at Six Months

	LEVEL STATION		
	a	b	c
Measured	6.5	4.1	8.8
Continuum	4.2	3.3	8.6
Winkler	1.6	0.6	4.8

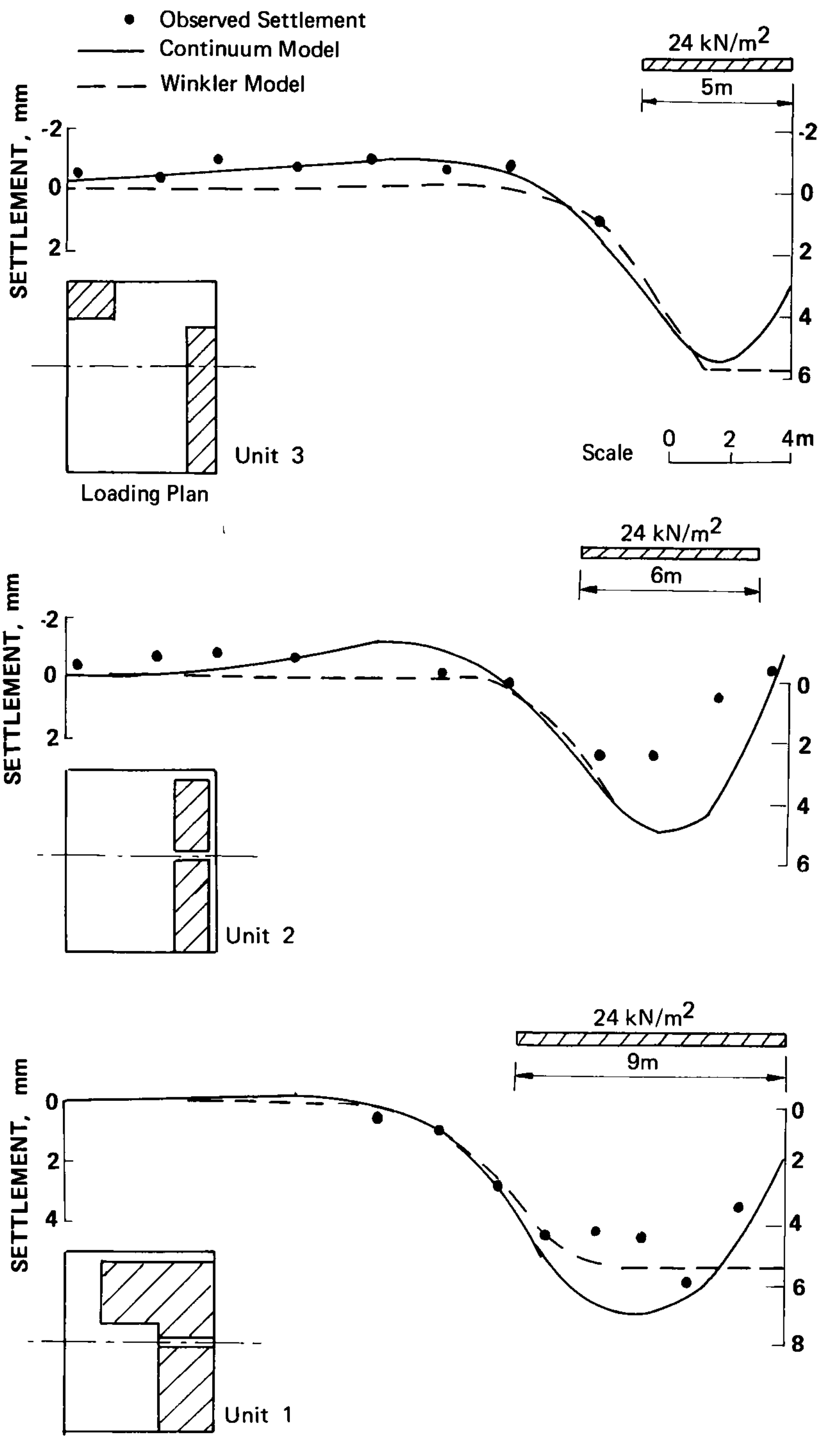

Figure 4 Measured and Computed Settlement Profiles at One Month

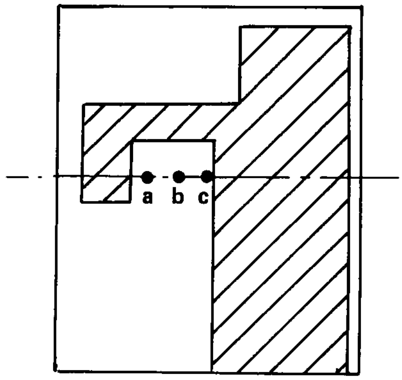

Figure 5 Loading Plan for Six Month Settlements

DESIGN CONSIDERATIONS

Detailed consideration is given to the results obtained from a parametric study using the continuum model, which would appear to represent a more acceptable predictive mechanism than the Winkler approach.

Settlement

Two important considerations are the extent of the loaded area that must be taken into account in the analysis of an individual strip (5m by 25m) and the effect of the joints. Results indicated that for this structure the inclusion of the load on a strip on either side of the one being studied was sufficient to accurately predict the behaviour of the central strip, and that the jointing arrangement has little effect on the settlements except when a joint occurs in the centre of an aisle. Typical settlement profiles along the centre line of the central strip are shown in Figure 3 for various aisle widths. The rapid build-up of hogging deflections within the aisle, as compared with the dish-shaped deformation with no aisle, is clearly demonstrated.

For aisle widths greater than 5m tensile ground reactions occurred within the aisle. However, except with a central aisle joint, the self-weight of the slab was sufficient to maintain compressive reactions for imposed load intensities up to 50 kN/mm^2. With a central aisle joint, tensile ground reactions occurred which were of sufficient magnitude to cause the slab to loose support in the area of the joint, for aisles as narrow as 1m.

Bending Moments

From the design standpoint, the magnitude of the hogging moments which may cause cracking in the upper surface of the slab are of paramount importance. The three major constraints which affect the intensity and distribution of these moments are discussed below.

Effect of Joint Arrangement

In order to study the effect of the positioning of the joints, two patterns have been studied, with the aisle running perpendicular and parallel to the joint direction. The moments acting in a direction perpendicular to that of the aisle dominate and are similar on all sections. Moment profiles along the centre line are shown in Figure 6. The results obtained for a continuous slab with no joints were identical to those shown in Figure 6(a). It is apparent that the introduction of joints parallel to the aisle direction, as in Figure 6(b), increases the hogging moments corresponding to aisle widths of less than 5m and decreases those occurring for greater aisle widths, see also Figure 2. This may be explained by reference to the profile for a 5m aisle where the point of contraflexure occurs at the joint and the hogging moments are unchanged.

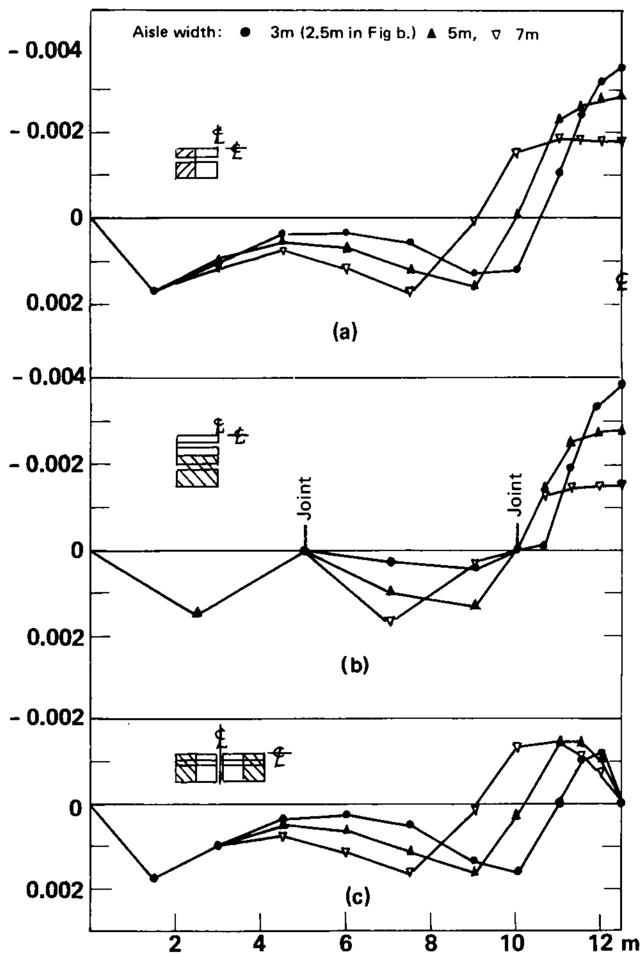

Figure 6 Moment Profiles (continuum; ν' = 0.5)

These results suggest that the jointing arrangement is of little significance in controlling the formation of the maximum moments. However, when a joint is introduced at the centre of the aisle, Figure 6(c), the maximum hogging moments are substantially reduced for aisles of less than 5m. As stated above, however, loss of support occurs within the aisle, with the possibility that contact may be re-established in a cyclic manner as vehicles pass along the aisle. In view of this, the moments shown are those produced with the tensile ground reactions acting.

Aisle Width

The variation of maximum moment with aisle width is shown in Figure 2. The magnitude of the hogging moments dominate for aisle widths between 1m and 7m, with maximum moments occurring at a critical aisle width of 3m. For aisle widths greater than 5m the maximum moment does not occur at the centre of the aisle. The maximum sagging moments are sensibly independent of the aisle width and the joint arrangement, although some increase in magnitude is indicated as the aisle widths become very large (above 10m), and they tend to occur towards the outside edge of the loaded area, where the sagging settlement profile is most severe. The long term action of the underlying soil ($\nu' = 0$) gives rise to the greatest moments which are less than the theoretical cracking moment for the slab (Winkler predicts that the slab will crack) but the critical aisle width remains at approximately 3m. The effect of different joint arrangements is shown for $\nu_u = 0.5$, similar effects being produced when $\nu' = 0$.

Relative Stiffness of Slab and Soil

All of the preceeding results have been based upon a particular set of values of the design parameters and represent what might be termed the design values. As with all soil-structure interaction problems, it is important to realise that it is the relative stiffness of the two components which governs the overall behaviour of the system. The relative stiffness may be quantified for convenience by a factor K, such that:

$$K = E_c(1 - \nu_s^2)\ t^3/E_s(1 - \nu_c^2)\ (ab)^{3/2}$$

Any variation in the soil and/or the slab properties may now be evaluated in terms of a change in K. The effect of variations in K upon the maximum hogging moments with respect to the aisle width is shown in Figure 7, from which it is particularly apparent that as the relative stiffness increases so the critical moment also increases. These values are also given in Table 3 for clarity.

Table 3 Variation of Moment with Relative Stiffness

K $\times 10^{-3}$	CRITICAL AISLE WIDTH, m	M/q a b CRITICAL	COMMENTS
2.6	2.0	0.0011	-
20.7	3.5	0.0046	'Design' values
165.6	6.0	0.0152	-

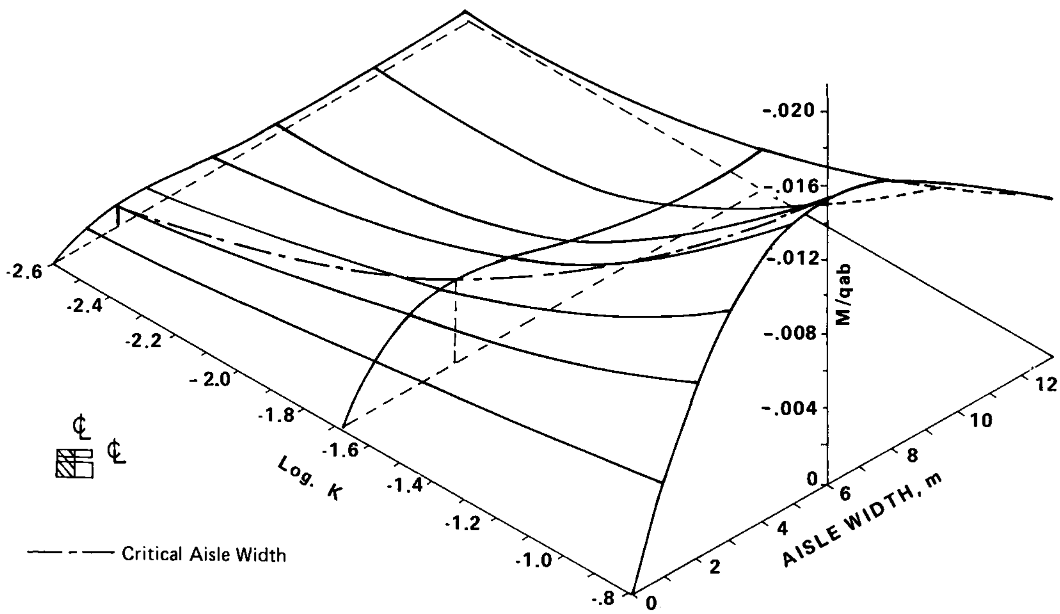

Figure 7 Variation of Hogging Moment with Relative Stiffness

CONCLUSIONS

From the comparison with the observed values of settlements predicted by both Winkler and continuum soil models, it is suggested that the latter is superior. The results of this preliminary study also lead to the tentative conclusion that the jointing arrangement adopted is of little significance in controlling the formation of the settlement pattern or the maximum bending moments induced in the slab. Of much greater importance is the width of the aisle.

ACKNOWLEDGEMENTS

The co-operation of Convoys Ltd., in allowing access to the warehouse is gratefully acknowledged. The computed results were obtained using the facilities of Queen Mary College and London University Computer Centres.

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DISCUSSION

Alan R. Selby. Firstly, I must commend Professor Kemp on the attractiveness of his strip method in its use and avoidance of major computer programmes. I would like to ask him if he has a proposal for solution of a slab which is simply supported along one edge with the opposite edge free. Referring to Figure 1 of his paper he showed a slab with a cantilever in one direction and a simply supported beam in the other. If we had a simply supported edge opposite to a free edge, we would then have a mechanism rather than a beam structure. Could I suggest that if his proposal is a torsion grillage to the slab, perhaps we are degenerating from the simplicity of the solution, and its appeal, towards a computer orientated grillage solution or even a finite element solution with a beam in both bending and torsion. Perhaps the benefits of the method are beginning to be reduced where we are considering equations to be solved, in your case, say, twelve simultaneous equations and the torsion grillage which would perhaps produce thirty-six simultaneous equations. This would then, however, obviate the need to go through the strip method having derived the loads in the first place. A grillage would have a field of moments which could be applied directly to the slab solution.

Kenneth O. Kemp. One of the dangers of suggesting a simple method that may be widely applicable is that someone may try to find a problem that you probably cannot solve. I am not sure that I have quite understood your problem, but could I first comment on the question of torsion? I would be against introducing torsion. First of all I only want approximate load distributions, so why introduce another unknown? We certainly do not want torsional moments in the final solution because that means you reduce a unique solution to a lower bound solution, so there seems no merit in introducing torsion.

Now the example you quoted was a slab simply supported on three sides and free on the other. Well there is no problem with that one. If you could design a grillage for it then you can design the slab for it. If some of the strips are nominally mechanisms then you get additional statical equations to help you solve it.

Alan R. Selby. I just could not understand how you were applying your compatibility deflection equations in assessing the load contributions in the two directions, where in one direction you had a structure and in the other a mechanism.

Kenneth O. Kemp. Well one set of strips is getting support from the other set of strips. Hillerborg deals with quite a few of these cases and I do actually give an example of the kind you are talking about in the paper I wrote for the Proceedings of The Institution of Civil Engineers*. The only problem I know that you cannot solve by this method, and it is obvious that you cannot, is the slab subjected to pure torsion. If you try and solve the problem of the slab under pure torsion, with torsion on the edges of, say, a square slab, of course you cannot solve it. That one is completely unsolvable by the method I have presented, but it is not a problem you are likely to meet often in practice. If you did, you would have to solve it by choosing the grid strips in the 45 degree directions, then it would work because then it is positive and negative moments. That is the only problem I have so far met for which you could not use the proposed method.

Hans Gesund. I believe Mr. Whittle was not quite able to finish his remarks during his keynote address and I would be interested to hear what he has to say about the problem of column support of slabs.

Robin T. Whittle. The problem in practice that I was going to refer to on flat slabs relates to the equivalent frame method and the moment that goes to the columns, which affects the magnification of shear. I am very interested in the information that is coming out from research at Queen's, Belfast, perhaps we should be taking account of the moment transfer for slabs at the edges and the corners. The concept of using total loading will actually affect the moment transfer as well, but the examples that I have gone through with you so far in fact do not show very much difference in terms of the final shear you have to use at internal and external columns on flat slabs. I believe that we still need to introduce into the code a means of reducing the column moment transfer from slabs to columns. A 30 per cent reduction of our column moment transfer from an elastic analysis using the equivalent frame might be suitable, for design purposes at least. The problem with which engineers are faced with the code as it stands, is that we have now to reinforce most of our slabs for shear; Figure 1(a) shows a simple case: it has only two perimeters of shear reinforcement and it is a situation where we would never have had to reinforce for shear previously to C.P.110. The situation in Figure 1(b) is a little more complicated. This has four rows of shear reinforcement and is typical of what is going on in construction at present. Figure 1(c) shows a coffered slab, the slightly denser areas mainly due again to the shear reinforcement that we are having to put in. In Figure 1(d) you will notice our standard practice which is to put in extra bars to which to link the shear reinforcement. We do not really know whether this is good or bad practice since there is not enough information from research to tell us. Figure 1(e) shows the actual shape of links that we are using and in fact in many situations we are finding that the links have to extend well into the coffers along the ribs. Sometimes we even find that links are required in the ribs where they cross outside the solid area. Figure 1(f) shows in more detail what is happening there and clearly it does complicate the construction quite considerably. Another means of reinforcing which is used for very heavy shear is shown in Figure 1(g). A structural steel shear head is a very efficient method of dealing with shear but is quite costly, especially when you have to make up the units specially. Although it looks quite satisfactory in the diagram, when you actually have a closer look you realise that there could well be a problem in placing the concrete.

K.O. Kemp.
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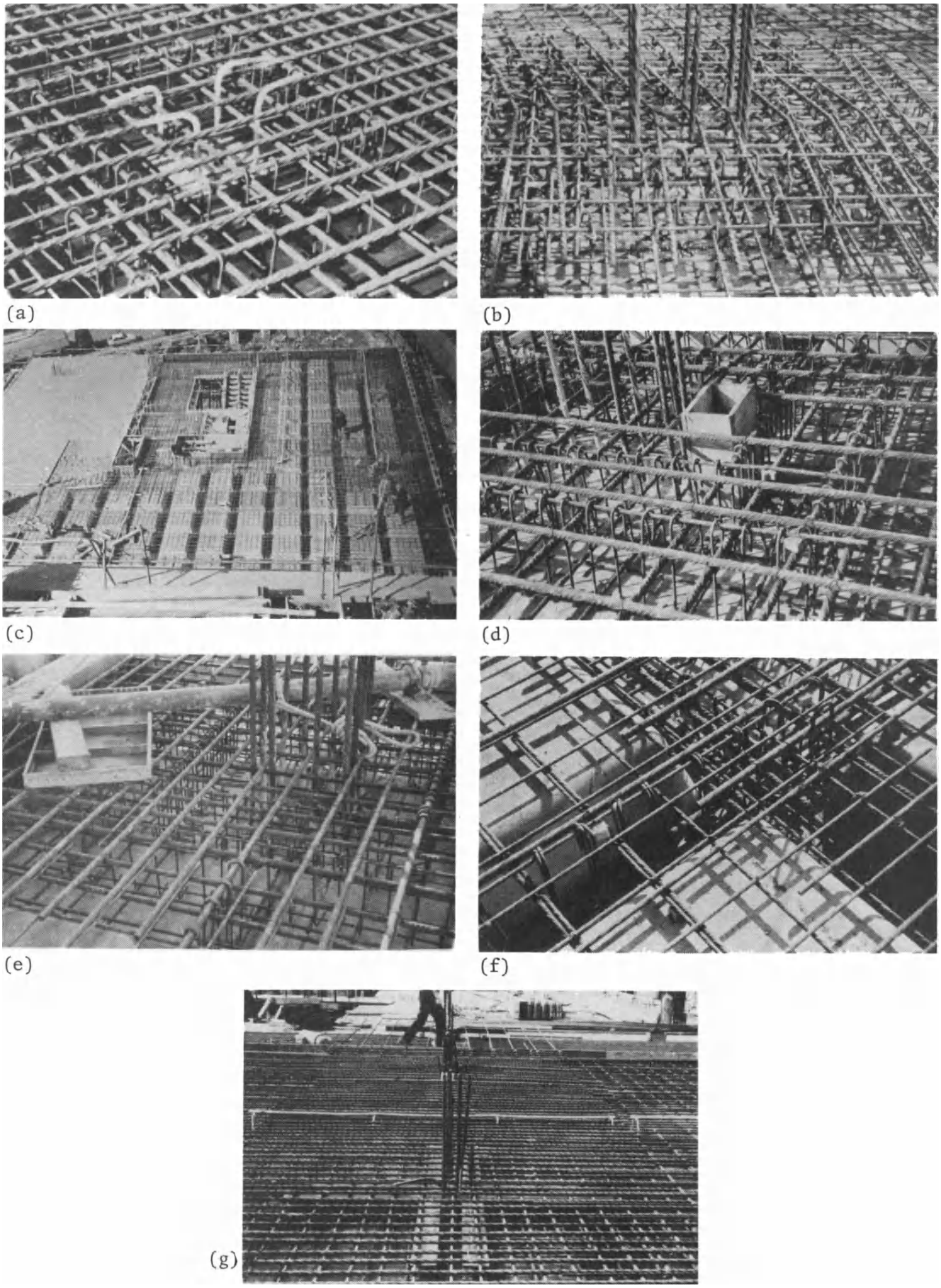

Figure 1 Methods of Reinforcing for Shear

Khalafalla B. Musa. My question to Professor Gardner concerns the continuous frame method for designing flat slabs. In C.P.110 there are distribution factors for the bending moment. For example, for the column strip 75 per cent for the negative bending moment and 55 per cent for the positive bending moment are specified and for the middle strip 25 per cent for the negative bending moment, 45 per cent for the positive bending moment. I would like to ask whether from your study you find that this confirms the distribution factors or otherwise. I would also like to ask about the position of the critical shear. In C.P.110 it is given that the critical section for calculating shear should be taken on a perimeter 1.5 times the overall slab thickness from the boundary of the loaded area. I would like your comment about this.

Noel J. Gardner. Living in North America, I am mainly concerned with the Canadian and American codes, which are identical, but they are very similar to C.P.110 in the distribution of the section moments across the slab. I am not totally happy with the distributions. In a negative moment area they seem to be correct on the basis of this model study. Our slabs only had an aspect ratio of 1.15 to 1. In C.P.110 you are only allowed to go to an aspect ratio of 1.33 to 1, but in the other three codes, the American, Canadian and Australian, you can go to a panel aspect ratio of 2 to 1. Whether our conclusions would hold for these extreme panels I do not know; for negative moments it seems fine, for the positive moments it does not seem quite so good. It seems that more of the positive section moments should be allocated to the middle strip and less to the column strips. In practice this probably does not matter much because in the middle strip you are probably governed by the minimum steel requirement for temperature and shrinkage effects, so in fact you are putting in more steel than you need purely for strength purposes. On the question of shear, from an elastic model like this we could not measure the shear so I can make no comments on it. I think there are people in the audience who are better qualified than I am to answer that question. However, both North American codes are similar, you take the perimeter of the supporting member plus half the thickness of a slab to find the critical section for shear.

Robin T. Whittle. On the first point of lateral distribution of the moments I have, in my paper, referred to a CIRIA report that is about to be produced which is based on Dr. Regan's work in London. In fact the code committee at the moment is actually looking at new tables of distribution of the moments, both positive and negative, for future inclusion in the code because it is felt that they are wrong for aspect ratios other than square in fact.

Leslie A. Clark. I would like to ask Dr. Gardner if he could elaborate on his statement regarding equilibrium because I am sure that he does not mean that equilibrium does not have to be satisfied for a single load pattern on the slab. Am I correct in thinking that his statements are based upon the fact that the negative and positive moment regions, for design consideration, arise from different load patterns?

Noel J. Gardner. You are quite correct. There is a slight difference here between C.P.110 and the A.C.I. code. The A.C.I. code insists on the equilibrium moment being satisfied as the positive moment is obtained by subtracting the negative moment from the equilibrium moment. If you are conservative on your negative moment, you are unconservative on your positive moment, automatically, and this means you are either using up some of your load factor, which was put there for some reason I presume, and you are relying on redistribution to take care of the moments due to the loading patterns which you are going to apply, which are not the pattern used to obtain the equilibrium moment, namely per panel $WL^2/8$.

There is a philosophical problem here concerning what to do about reinforcing for maximum negative and maximum positive moment. It is desirable to detail for maximum positive moment from one set of loads, maximum negative moment from another set of loads and reduce the load factors, as Professor Kemp suggested, rather than using our present method where we insist on equilibrium being satisfied, especially in the North American codes, and then hope that something else is going to take care of any pattern loading effects. Equilibrium for every load pattern should be satisfied.

Kenneth O. Kemp. I would just like to ask Professor Gardner what load factors is he using on the dead load when he is considering that load case which produces negative moments in the middle of the span. Is he using 1.4 times the dead load and if so, why?

Noel J. Gardner. Yes. I did use a factor of 1.4 on the dead load. It would seem rather inconsistent to be using 1.4 on the dead load for the outer panels and 0.9 for the dead load on the inner panel for the same load case, but it is a valid point.

Kenneth O. Kemp. And should you not be using a factor of 1 there?

Noel J. Gardner. A factor of 0.9 is the North American code recommendation, which would be even worse. We used 1.4.

Adrian E. Long. I would like to follow up a comment that Mr. Whittle made in respect to Professor Gardner's model. I think it is somewhat dangerous to extrapolate from an elastic model to what is actually happening in a real concrete structure. I would very much doubt whether you will get positive moments developed when you take into account cracking in concrete, as will invariably have occurred in the negative moment region. You may not get this transmission into the negative and into the positive moment region so I would have thought that before we accept completely what we get from an elastic model, we should very seriously consider doing tests on more realistic micro-concrete models, if necessary, or correlate back to the tests that were done originally in the P.C.A. and in North America and in the University of Illinois. I think it is somewhat dangerous to use very simple elastic models which really do not give you a result which is much different from an elastic frame analysis.

Anthony R. Cusens. Could I follow up that point about the model? Perspex is a material I have never liked for model work because it has such a very high Poisson's ratio. Could Dr. Gardner give us a little more detail about the way in which he actually worked out his values of moment? Was he in fact using strains in one direction or were the measurements based on strains in orthogonal directions? The Poisson's ratio effect would be important on a slab of that kind.

Noel J. Gardner. We were using strains in both directions and using a Poisson's ratio of 0.35 to calculate the moments given in the paper.

William P. Liljestrom. I would like to address comments to M. Poitevin and to Dr. Walsh. Each of us, and I think this is important in an international conference, has ideas and procedures which differ in different localities. I noticed on one slide, M. Poitevin, that you cast a slab directly onto a polyethylene membrane which was underlain by about 50 mm of sand. I have a philosophy, possibly I am wrong, which is that in this case you may promote warpage of the slab as it is beginning to cure. I prefer to have the slab, then the sand and then the membrane beneath, so that as the slab dries, the water can go in two directions. In the method you showed more water is floated to the surfact, bringing up fines and possibly promoting a surface that is less resistant to abrasion. More importantly however is the possibility of curling of the slab with the method you showed.

Now my comment to Dr. Walsh: I know that the Post-Tensioning Institute in the U.S.A. is undertaking a very extensive research programme which has an unfortunately unattractive title: SOG, standing for Slab-On-Grade. They are going into areas with heaving soils and are using post-tensioned slabs on grade, especially in residential areas. I thought that possibly you may wish to exchange information with them.

Paul Poitevin. I quite agree with you. Curling of the slab is a hazard with this kind of disposition, but I did not explain that it was only for house slabs, or very small slabs, with a maximum dimension of not more than 5 m. It is not for industrial slabs so the problem is not really acute with the kind of slabs for which it is intended. I agree with you, however, on the curling effect and the effect of moisture movement to the surface.

Paul F. Walsh. I visited the U.S.A. four or five years ago and at that stage I was told that the use of post-tensioned slabs had been banned by the Federal Housing Authority because of the large numbers of failures. Subseqently there was a fairly extensive development of committees to develop improved design methods employing somewhat heavier slabs than were used previously. John Holland also was over there and came back with perhaps a rather different impression because he went two or three years after me. As a consequence of these visits we have prestressed slabs on grade under test on the same site as used for the tests described in our paper. Economics in Australia do not favour them, however; we do not normally use ungrouted tendons and a few other factors like this tend to indicate that they are not going to be the solution to our problem.

R. Colin Deacon. I would like to make one comment and post one question regarding M. Jalil's paper. The comment: I understood that the recommendation for the spacing of expansion joints was 25 m. This seems to be rather unnecessarily close; my feeling is that concrete shrinks preferentially to expanding and that the need for expansion joints is minimal. We would recommend three times that distance and perhaps even more. My question concerns the opposition to the use of dowel bars. Taking up Mr. Whittle's point earlier on, would you explain why you do not approve the use of dowel bars?

Paul Poitevin. I agree with you. Shrinkage is very much to be feared and 25 m is very conservative and costly. As for dowel bars, I think that the French committee has followed the French practice for road slabs and airfield runways. I discussed this with engineers and researchers from the Laboratoire Central des Ponts et Chaussées and they were not happy with dowel bars. They think it is a costly method and in their typical design they avoid them; so for housing slabs this same trend was followed. I cannot give you more detailed reason because it is not my particular field.

John M. Rolfe. I was interested in the work Dr. Walsh presented because I have been in the privileged position of doing a considerable amount of work in towns where the building controls have been skimpy, to say the least. Consequently if I told the town engineer that what I was doing was all right, he would take my word for it without my having to comply with any bye-laws. We have done a considerable amount of work along the lines of Dr. Walsh's present recommendations for large flexible rafts, with great success. I would like to ask Dr. Walsh if his proposed regulations include limitations on length and height of building.

Paul F. Walsh. The limitations written into the regulations are for what we term Class 1 construction or similar, which is basically single unit houses, so this implies a limit of about 20 m by 10 m. There is the possibility that somebody could build a huge single house but we are not too worried about it since people doing that do not usually just work to the minimum regulations anyway, they usually employ an engineer.

Session **4**

Construction Techniques

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Swedish Cement and Concrete Institute, Sweden

Keynote Speaker: John R. Illingworth

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CONSTRUCTION TECHNIQUES

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INTRODUCTION

It is a regrettable fact of life that conferences related to concrete always produce considerable enthusiasm for design and technical matters yet, somehow, a disproportionate effort in relation to practical construction matters. Why this should be so stems, I suspect, from the continuing gulf between those who design and those who construct, in a large majority of instances. The designer, with all his modern aids, seeks continuously to move to more and more advanced design concepts, while those who execute the work struggle in a world of diminishing skill to produce efficiently and to the required standard the last fashion in design terms.

The organisers of this conference, in consequence, deserve praise for the extent to which they have introduced practical papers. Not only in this session - Construction Techniques - but also in the following sessions 5 and 6.

SESSION THEME

At first sight, a conference on Concrete Slabs relates to only a part of construction in concrete. If, however, we stop to think about it, concrete slabs, both ground bearing and suspended, represent far and away the largest proportion of concrete used in a structural role. It is equally true, as far as this session is concerned, that the construction techniques adopted will have a major role in determining the cost of the finished structure. Nor should we overlook the fact, in the case of suspended slabs, that safety in erection comes very much into the picture as well.

It is a matter for regret that within the papers presented there is little, if any, consideration given to the formwork and falsework involved in in-situ construction or, indeed, problems which can arise as a result of the reinforcement detailing and fixing.

All though this session the emphasis in the various papers is on concrete, when, with respect to in-situ suspended construction at least, the cost breakdown on repetitive slabs is likely to be 37, 25 and 38 per cent for concrete, reinforcement and formwork respectively. The concrete element is, therefore, only just over one third of the cost of the finished product. Perhaps these points can be taken up in the discussion periods.

PAPERS PRESENTED

In all, ten papers have been produced for this session. In general terms, the papers provide a good, if incomplete, mix and should give rise to lively argument, if only between the in-situ and precast lobbies.

CONSTRUCTION TECHNIQUES

It is now appropriate to consider the theme of this session in rather more detail. The construction techniques available, in relation to concrete slabs, are wide. Any given set of conditions will usually be capable of solution by a number of design approaches and the construction techniques related to them. Which option to adopt and the technique to employ should, ultimately, be decided on cost.

Cost

In assessing which technique will be most favourable in cost terms, one cannot ignore fundamental design. Unfortunately, design economy tends to be considered in isolation, largely on the basis of how much material is saved. What has to be recognised is that savings in material do not necessarily create economy in overall terms. Construction techniques in relation to an alleged economic design may well prove to be more expensive than the claimed design savings. Costs elsewhere also come into the picture. For example, the ease of installation of services, partitions and the cost of ceiling finishes.

Several of the papers presented today are guilty of generalised statements with regard to cost.

In the construction industry every contract is one off and what may be fine in cost terms for one structure is highly unlikely to be so on another. When evaluating, in comparative terms, the merits of alternative construction techniques an adequate and comprehensive comparison for the case in question should be considered. When done, the final result may well not be that anticipated. A paper by Gifford and Baker (1) is very instructive in this respect. Equally, this Author's views - albeit related to alternative construction methods only - were expressed in some detail to a conference held in this University in October 1975 (2).

Cost is related not only to design, but also specification. This in turn will affect the technique. Alternatively, techniques may be developed which cause alterations in specifications.

Specifications

It is an interesting fact of life that almost all major changes in concrete specifications of recent time, related to operational activities, have resulted from contractors trying something different and proving it works. That they have done so, initially, has been in an endeavour to keep their own costs down. Having succeeded, the advantages have ultimately been passed on to clients in later contracts.

One such change, of course, is covered in Deacon's paper. The Author's company was a pioneer in the field of long strip floor construction, together with the associated use of vacuum mats, for de-watering, discussed in a paper by Wallwork et al in Session 5. Using the above technique it was found possible, in the winter months, to improve productivity, in relation to factory floors, by about 25 per cent and reduce costs by 10 per cent. In fact, these figures relate to the direct costs in constructing the floor, other savings resulted in relation to follow-up trades. By way of example, construction of the floor strips along each side of the building as a first operation allows sheeting and cladding, and any service installations running along the wall, to proceed much earlier than would be possible with chequer board bays.

What, to the Author, is a serious omission from the papers presented, is the technique of large area pouring of suspended slabs, which comes very much under this heading of specifications. By this is meant the abandoning of small bay specifications and pouring slabs of considerable area in one continuous operation. The Author's company, again, have had a pioneering role in this respect in the U.K. Figure 1 illustrates the scale of what has been done on a recent office block in London.

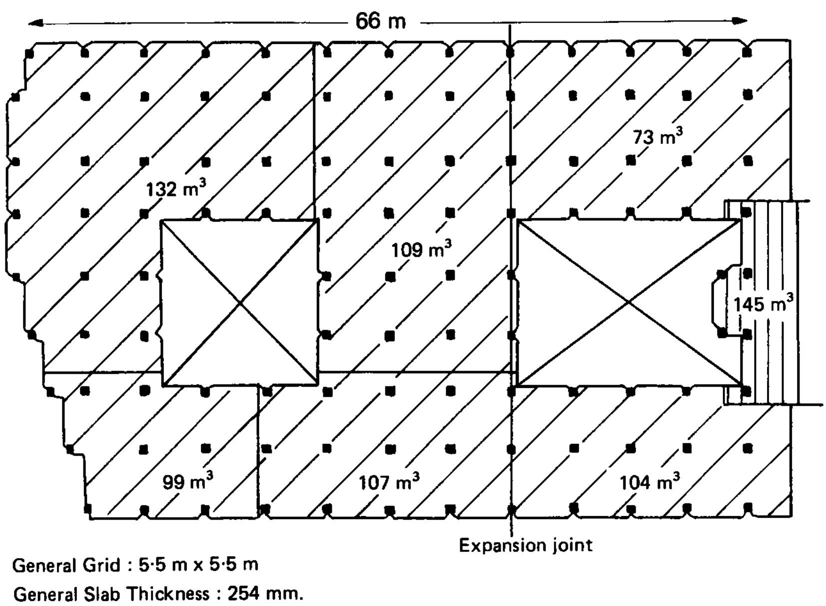

Figure 1 Large Area Pouring, all concrete pumped

To the uninitiated, the concept of pouring such large areas without joints raises the spectre of endless cracking. Clearly, consideration has to be given to the reinforcement in this respect. However, the work by Hughes at Birmingham University (3,4) has shown what factors need to be taken into account for a satisfactory result. In any case, with solid plate floors, the appropriate reinforcement is usually inherent in the structural design and requires little, if any, modification. The advantages of this technique are considerable:

1. The cost of step-ends is dramatically reduced. For example, on a seven storey office block containing 790 m^3 of concrete

per floor, the concrete was pumped in bays averaging 112 m^3 per bay, 300 mm thick. The saving in stop-end costs alone (1974 rates) represented £0.53 per m^3 of concrete placed, against the maximum 40 m^3 bays originally specified.

2. Pump efficiency requires high output. Large area pouring with pumps satisfies this with reduced cost.

3. The minimum number of construction joints improves the structural integrity of the floor. (contractors are not noteworthy in producing day joints to the standard the designer anticipates).

It is significant that, on the London office block previously mentioned, the engineer was so pleased with the results obtained, that the method has now been specified by him on more recent contracts.

DESIGN DETAILING

Another omission, in the Author's view, is the relevance of detailing to construction techniques. The efficiency of any technique ultimately rests on the adequacy or otherwise of the detailing.

1. However economic the design (in theoretical terms) all advantage will be lost if, for example, the detailing of the reinforcement is such that it is difficult and time consuming to assemble on site. Time and money will be expended in inefficient working and the contract time (money again!) increased. Equally, formwork costs are also increased.

   The training of reinforcement detailers is woefully lacking, today, in this respect.

2. Precast techniques largely stand or fall on the detailing of connections and the consequent ability to execute them in the manner intended by the designer. Too often, details of this type are impractical or do not allow an adequate inspection facility of the finished result.

   Today, the consequences are for all to see: weather penetration and structural failure to a greater or lesser degree.

CONCLUSIONS

In concluding this keynote paper, it is hoped that the points made will help to stimulate vigorous and informed discussion on both the papers presented and the aspects of construction techniques raised which have not been dealt with by others.

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GROUND FLOOR SLABS—UK PRACTICE IN THE SEVENTIES

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ABSTRACT Significant changes have occurred in the methods of designing and constructing concrete ground floors in Britain during the past decade. In particular, piecemeal chequer-board construction with applied toppings and screeds is giving way to continuous long-strip methods with the slab direct finished to give the required surface for wear or the application of floor coverings. Design recommendations for moving loads and the construction details which have been evolved to facilitate long-strip construction are given in the paper.

INTRODUCTION

The design and construction of ground floor slabs have never been, and probably will never be, exact sciences. Design is achieved by the application of rules of thumb and experience, rarely by calculation, with details being frequently left to the site; and construction methods tend to follow well trodden paths. The subject of floors has little appeal to the research worker and reliable data is scarce.

Of all the elements in a building, the floor slab probably suffers the most from pressures to save money by cutting down on quality of materials and thickness, and on insistence on rapid construction and early trafficking. The inevitable consequences of these practices are floors which perform badly, dissatisfied owners and clients and expensive maintenance.

This paper describes the changes which have occurred in British practice during the past decade, changes which have been prompted to a great extent by the shortcomings of conventional methods.

THE CONVENTIONAL FLOOR

Until the end of the 1960's the majority of ground floors constructed in Britain were of two layers, the base slab and a surfacing. The base slab concrete was of moderate quality, usually specified as having a compressive strength of between 20 and 25 N/mm^2, or of nominal volumetric proportions. It was laid in small bays often limited to 25 to 30 m^2 in area, with the stipulation that adjacent bays must not be laid for at least 7 days. Thus, floors were typically constructed on the alternate-bay or chequer-board principle. Reinforcement was most commonly of welded square mesh fabric laid in either a single layer or one layer top and bottom.

The reinforcement passed through all bay joints unless these coincided with the main movement joints in the floor. This required the use of split forms, and timber was the most common material used. If the floor was not to receive a covering, a topping was applied usually of high strength or granolithic concrete, laid either monolithically with the base slab or more often applied separately later with some degree of mechanical key to the hardened base concrete. Where the floor finish was of tiling, carpeting or thin sheet material, it was usual to apply a levelling screed of sand-cement mortar up to 50 mm thick to the hardened base. Faults frequently occurred which can be summarized as follows:

i) Break-down of joint edges, often due to poor finishing and compaction; all joints had to be finished by hand.

ii) Poor levels across joints, due to the same causes as (i).

iii) Debonding of toppings and screeds, leading to cracking, caused by inadequate preparation of the base slab and often poor workmanship and materials; this almost always occurred in separately applied toppings and screeds.

iv) Excessive wear and dusting, due to poor workmanship and materials.

A large proportion of all technical enquiries dealt with by the Advisory Division of the Cement and Concrete Association concerned these common faults in floors and as a result, at the beginning of the decade the Association commenced a study into methods of designing and constructing floor slabs, with the objective of improving techniques and performance. Design and construction practices and finishing techniques were all considered, as was the plant and equipment currently in use and new items becoming available. It was clear that the problems commonly experienced were mainly concerned with toppings and screeds and it was thought that if these could be eliminated many difficulties would also be removed. A second potentially fruitful line of attack was on the piece-meal methods of construction. If this could be improved to allow more continuous construction, greater efficiency in the use of labour and plant, and hence a potential saving in costs, would ensue.

TOPPINGS AND SCREEDS

Toppings and screeds appear to be used more widely in the U.K. than in other countries. They have some advantages, the main one being the ability of the main contractor to provide a firm base early in the job for the erection of steelwork and services without the need for strict control over levels and finishes. No damage occurs to the floor finish as this is generally applied by specialist subcontractor after the major construction operations are completed. Where heavy wear is anticipated the argument is made that special hard aggregate is required to give the necessary abrasion resistance, and this is most economically provided in a thin topping. A study of work done (1, 2) on abrasion resistance shows this argument to be something of a fallacy, the quality of the matrix of the concrete being generally of more importance than the selection of special aggregates. Thus the use of an appropriate grade of concrete, suitably finished, can produce the necessary wear characteristics without the need for toppings.

The successful use of toppings and screeds demands first class workmanship and materials which are frequently not available. Even so, these flooring operations often interfere with other trades, and in particular the introduction of a wet trade towards the end of construction can cause delays to the laying of final floor finishes because of moisture in screeds.

It became clear that with proper planning and selection of concrete quality, Table 1, the majority of industrial floors can be provided in a single layer, direct finished to give the necessary surface regularity and wear characteristics, thus eliminating the inherent problems of separately applied toppings and screeds.

Table 1 Grades of Concrete Suitable for Various Duties

CATEGORY	DUTY	GRADE N/mm^2	MINIMUM CEMENT CONTENT kg/m^3	TYPE OF FINISH
1	Light foot or trolley traffic, e.g. in offices, shops.	20	280	Thin sheet coverings, tiles, or carpets.
2	General industrial use; vehicles with pneumatic tyres; mild chemical conditions.	30	330	Structural slab finished as wearing surface. The minimum cement content is necessary to ensure wear resistance.
3	As 2, but heavy abrasive conditions (e.g. vehicles with solid wheels), or strong chemical attack.	25	300	Applied toppings to suit conditions.
4	Heavy industrial use; moderate chemical conditions.	40 or higher	400	Structural slab finished as wearing surface. Abrasion resistance increases with strength. Strength level according to degree of wear anticipated.
5	Heavy industrial use; heavy abrasion (e.g. by steel-shod wheels); impact; strong chemical attack.	30	300	Applied toppings to suit conditions.

Note: Requirements for both strength and cement content should be satisfied.

RATIONALIZATION OF CONSTRUCTION

The methods of the highway engineer in laying long continuous slabs and forming joints in-situ by crack induction techniques provided an attractive answer to the haphazard methods then in common use for concrete floors. It became apparent that in essence there was little fundamental difference between a road and a floor slab, the latter could be considered to be a road cut up into strips and laid side by side. During this period the work of various other authorities and firms who were also thinking along similar lines was examined. One of these was the English Industrial Estates Corporation, a government body which for many years has been responsible for the setting up of industrial estates and the building of advance factories in areas of high unemployment throughout England. Similar bodies carry out a similar function in Scotland and Wales. Generally standard units are built for leasing and at the time of construction the tenants' requirements are not known.

During the 1960's a method of detailing and construction was evolved in which the floor was laid in strips about 4 m wide and up to 50 m long between expansion joints. Transverse joints were induced by sawing generally along column grid lines. Reinforcement was continuous in the bottom of the slab and was carried through the longitudinal joint through a composite steel and timber side form as in Figure 1. The lower part of the slab was of moderate quality concrete but of low slump and was compacted by vibrating roller. It was surfaced by a 20 mm thickness of granolithic concrete laid monolithically with the base. The Cement and Concrete Association has carried out a survey of a number of these floors (3), and it has been found that many have performed well, but problems have arisen over the supply of ready mixed concrete of very low slump, and its compaction by vibrating roller. Of particular interest was the fact that the sawing of the induced joints presented few problems, and could often be delayed for a week or more without risk of premature cracking. It became clear that the early temperature changes which can cause premature cracking problems in roads with sawn joints are very much less significant in concrete floors laid under roof cover.

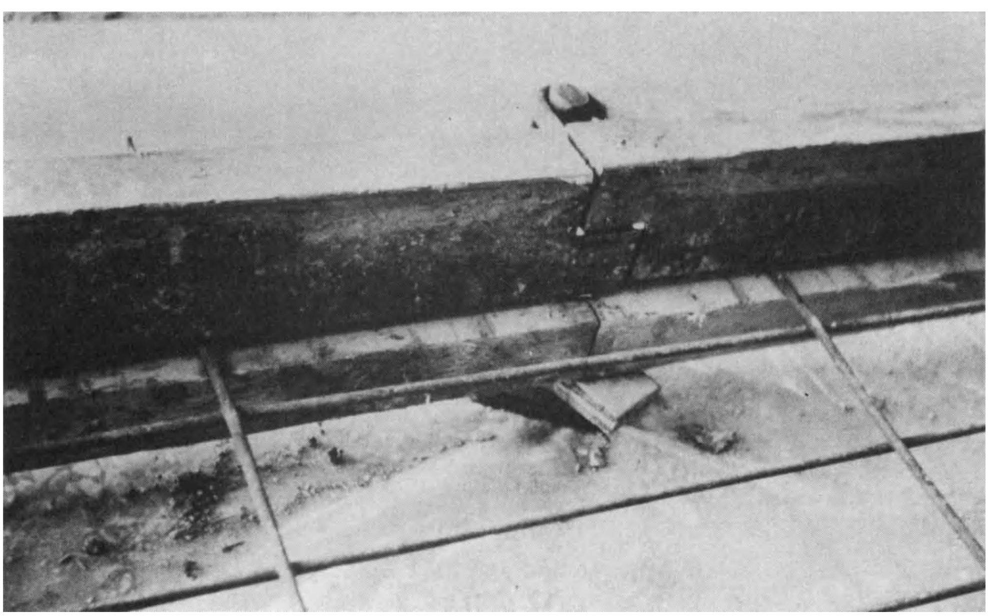

Figure 1 Typical Longitudinal Joint Construction Details in Earlier E.I.E.C. Floors

In the commercial field, a firm of reinforcement suppliers, the British Reinforced Concrete Co. Ltd., produced designs at this time which enabled floors to be laid in strips up to about 4.5 m wide in lengths to suit the dimensions of the floor.

This design was essentially of continuously doubly reinforced concrete, and they developed special split steel side forms to accommodate the reinforcement which was carried through the longitudinal joints. The reinforcement was of special welded mesh fabric with the bars placed at 300 mm centres to allow the concrete gang to place their feet between the bars and not on them. Special circular mesh spacers were also used to maintain the two layers of reinforcement in their correct location, see Figure 2.

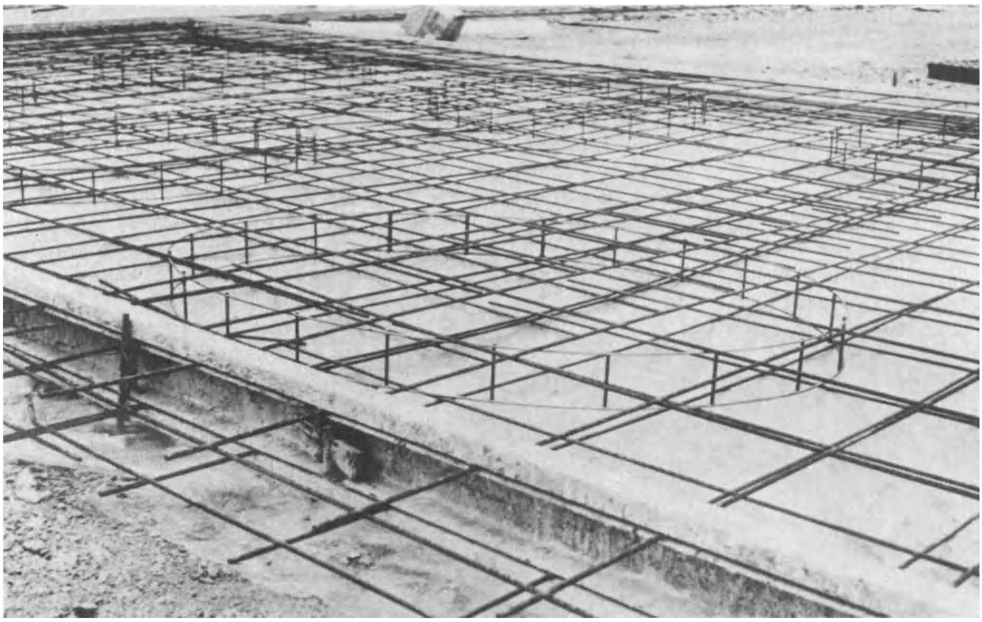

Figure 2 General View of Reinforcement Layout and Longitudinal Joint Construction to B.R.C. Design

From these studies it became clear that the concept of long strip construction was appropriate to industrial ground floor construction and would allow a more logical flow-line method of working with potentials for cost savings. It thus follows that if appropriate design details were evolved which allowed reinforcement and joints to be placed during concreting operations this would allow rigid steel side forms to be used and permit the prepared sub-base to be used as a haul road direct to the point of placing.

COMPACTING AND FINISHING

The Cement and Concrete Association received a great deal of help and encouragement from the manufacturers of plant and equipment in stimulating interest in new methods. The long established timber compacting beam is now giving way to the more effective and easily used double beam compactors and machines for floating and trowelling the surface are increasingly being used as traditional skills of hand trowelling become more scarce.

One major disadvantage of trowelling as a finishing technique is the extended time required to complete all the operations, which results in overtime working. This can be substantially reduced by use of the vacuum dewatering equipment which has been developed and is widely used in Scandinavia. When applied to the slab immediately after beam compaction, processing for typically 25 minutes for a 150 mm thick slab withdraws excess water from the concrete and causes rapid stiffening such that the first power floating operation may be started immediately afterwards.

An alternative technique is that of power grinding at early age. In 1972 the author visited Denmark where this technique was being developed (3). Its successful use relies on the quality of surface finish achieved while the concrete is still plastic, but with careful use of such simple tools as the skip-float the necessary standards can be produced without protracted periods of overtime working. The final finish is provided by the grinder a day or two later during normal working hours.

DESIGN

The design of floor slabs is generally based on experience or empirical rules of thumb. This is mainly because of a lack of data on the loading and performance of industrial slabs and the difficulty of applying theory with any degree of certainty because of the large number of unknowns and variables. In an attempt to give some more rational guidance for floors trafficked by moving wheel loads such as fork lift trucks, the methods of the highway engineer were adapted. Data from long-term tests on full size experimental roads showed that:

i) natural sub-grade soils could be classified into three groups in relation to their effect on slab thickness. The largest central group was classified as normal, the two outer groups being designated weak and very stable. For these latter two groups the slab thickness appropriate to normal soil conditions is required to be modified by +25 mm or -25 mm respectively. For industrial slabs, only normal and weak conditions have been considered.

ii) the required slab thickness increases as the cumulative axle loading throughout the design life of the slab increases.

iii) the damaging effect on the slab increases as axle loads increase, and enhancement factors related to a standard axle have been derived.

iv) with slabs over 170 mm thick, there is virtually no difference in performance between reinforced and unreinforced slabs. The effect of reinforcement is therefore one of crack control, not crack prevention.

By adapting the results of established highway experience, the recommendations discussed below were made.

Slab Thickness

Guidance on the selection of slab thickness is given in Table 2 for fork-lift trucks of typical load capacities, using the data of (5).

For static point loading from racking systems, and distributed loadings, recommendations are given in (6), using data from other studies.

Table 2 Recommended Thickness of Slab (mm)
for Typical Loadings from Fork-Lift Trucks

FORK-LIFT TRUCK RATING kg	MAXIMUM AXLE LOAD kg	8 HOUR WORKING DAY									24 HOUR WORKING DAY								
		Movements per hour																	
		4			10			20			4			10			20		
		Life, years																	
		10	20	30	10	20	30	10	20	30	10	20	30	10	20	30	10	20	30
2000	5500	125							150		125		150						
3000	7000	125		150											175				
3750	8500	150							175		150		175					200	
4500	10000	150				175			200		175				200				225
7000	15500	175		200					225		200		225			250			275

Notes: 1. The Table has been based on average axle loads, from typical two-axle fork-lift trucks.
2. Axle loads are average, maximum fully laden axle loads.
3. One movement assumes that a truck first crosses a point on the floor loaded, then returns unloaded over the same point (e.g. adjacent to a doorway, loading rack, or on a main aisle).
4. The life spans have been based on (a) 8 hour day, 6 day week, 50 week year, (b) 24 hour day, 7 day week, 52 week year.
5. Table is for normal subgrades, add 25 mm for weak subgrades.

Reinforcement

Recommendations for crack control reinforcement are given in Table 3. These are derived from the restraint forces (against the effects of drying shrinkage and temperature contraction) due to sub-base friction on the underside of the slab, acting over the whole length of slab between debonded movement joints. Friction is assumed to be minimised by the use of plastics sheeting under the slab. It is recommended that induced joints (frequently with reinforcement passing through them) be formed at spacings of 10 m maximum.

DESIGN DETAILS

In order to gain the maximum advantage from the use of long strip construction, it seemed desirable to establish the following principles:

i) To maintain control of surface regularity, rigid side forms, preferably of steel, should be used. It follows that mesh reinforcement should extend only within the limits of each strip, and any connection between adjacent strips should be by means of tie bars inserted through pre-drilled holes in the side-forms.

ii) The maximum strip width should be 4.5 m, as an aid to achieving good surface regularity, and to facilitate the placing, compaction and finishing of the concrete.

iii) To allow maximum use of the sub-base for placing the concrete directly in position from dumpers or truck-mixers, it should be possible to place the mesh reinforcement during the sequence of concreting operations.

Table 3 Fabric Reinforcement for Slabs of Various Thickness

THICKNESS OF SLAB mm	B.S. STANDARD MESH FABRIC TO BE USED FOR MAXIMUM EFFECTIVE LENGTH OF SLAB BETWEEN FREE JOINTS OF				
	15 m	30 m	45 m	60 m	75 m
125	A 142	C 283		C 385	
150	A 142	C 283		C 385	C 503
175	A 142	C 283	C 385	C 503	C 636
200	A 142	C 283	C 385	C 636	
225	C 283	C 385	C 503	C 636	C 783

Notes: 1. A 142 is a standard square-mesh fabric. The others are standard long-mesh fabrics.
2. If square-mesh fabrics only are obtainable, the cross-sectional area of main wires should be equivalent to that of the required long-mesh fabric.
3. If dowelled contraction joints are used in place of tied transverse joints, the required mesh weight is determined from the joint centres.

Details were therefore evolved in accordance with these principles, and Figure 3 shows a typical layout of joints for a floor to be constructed by long strip methods.

The tied transverse joint detail is shown in Figure 4. The short length of mesh near the bottom of the slab is intended to maintain aggregate interlock across the induced crack and hence a degree of load transfer. The surface groove may be formed by sawing or may be wet formed by inserting propietary steel or plastics strips into the surface of the concrete before it has hardened. Very careful workmanship is required for these wet formed methods. The bottom crack inducer was intended to reduce the depth of surface groove required and hence sawing costs, but there can be a risk of premature cracking and the use of the bottom crack inducer is losing favour. These joints, spaced up to 10 m apart, in conjunction with debonded contraction joints at wide intervals, and shown in Figure 5, have been found suitable for controlling random cracking in most conditions.

The longitudinal joint between adjacent strips is shown in Figure 6. This simple butt joint is formed initially against a rigid side-form. Steel side-forms with square top edges are recommended, but if timber is used the top edge should be capped with a small steel angle. These forms can be drilled at mid depth for the tie bars to be inserted as concrete placing proceeds.

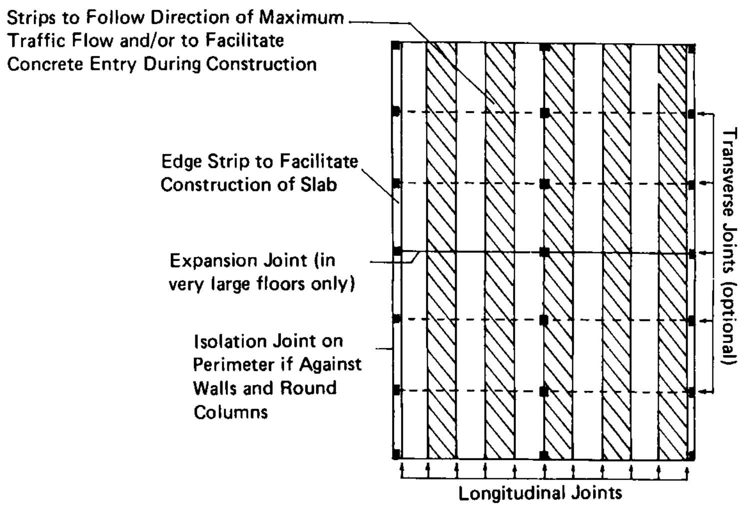

Figure 3 The Joints Used in a Typical Floor Layout

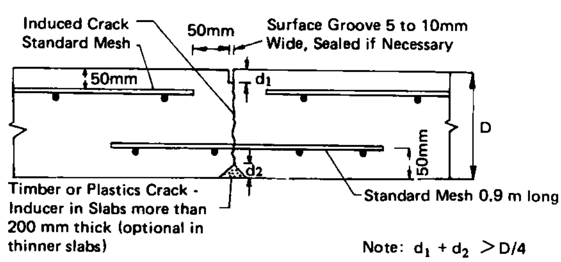

Figure 4 Tied Transverse Joint

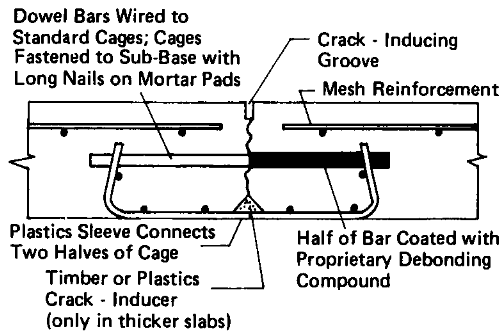

Figure 5 Contraction Joint

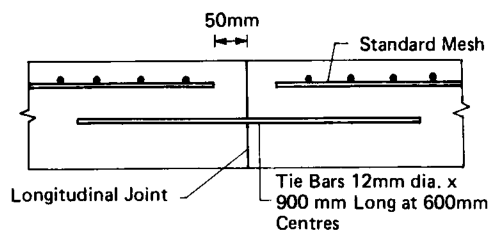

Figure 6 Longitudinal Joint

CONCLUSIONS

The first results of the study were reported at a symposium organised by the Yorkshire and Humberside Region of the Concrete Society in May 1972, and caused considerable interest. The symposium was repeated in various centres during the following year and in 1974 the Cement and Concrete Association published its recommendations (7, 8 and 9) and launched a campaign of up-dating meetings and demonstrations throughout the country, with the assistance and co-operation of many firms of suppliers, contractors, engineers, architects and clients.

These recommendations have formed the basis for the design and construction of a large proportion of all floors laid in the U.K. in the past six years. Further developments are now taking place with improved plant and methods, to allow larger areas to be laid and finished in one operation. The past decade has seen significant changes in British practice, and an upsurge of interest in the subject of industrial floors, and in the improved means now available for overcoming many of the common problems.

ACKNOWLEDGEMENTS

The author wishes to record his thanks for all the help received from colleagues throughout the industry during the past decade. This paper is presented by permission of Dr. R. Sharp, Director of Advisory Division, Cement and Concrete Association.

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THE UTILISATION OF FLOWING CONCRETE FOR SLAB APPLICATIONS

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ABSTRACT The distribution of concrete over large areas such as slabs and foundations can be both time consuming and labour intensive. In the U.K. this is exacerbated due to the use of concrete mixes with workabilities in the 50 to 75 mm range. However, the adoption of a highly mobile concrete mix, commonly called flowing concrete or superplasticized concrete, can generally speed up this operation and ease manpower requirements. These materials are not the universal answer to all concreting problems, as in many cases conventional materials will suffice, but they are a useful addition to the range. There are particular quality control procedures required to produce consistent flowing concrete, mixes normally need to be redesigned, site operations may require amending, finishing techniques should be considered, correct curing is to be recommended and finally the additional cost of the concrete must be justified.

INTRODUCTION

The concrete mixes commonly specified for use in the U.K. construction industry tend to be standardised on a 50 mm mean slump. However, there are exceptions, pump mixes require an average workability of 75 mm to enable them to be pumped and specialist activities such as piling or diaphragm walling use mixes with collapse slumps in the 150 mm range. Nevertheless, good practice is usually associated with stiff immobile concrete mixes, which require excessive labour and heavy vibration to ensure satisfactory placing without honey combing or voids. Consequently, the speed and efficiency with which concrete can be placed may be adversely affected by the consistency of the mix, hence equipment such as concrete pumps and truck-mixers will be utilised well below capacity. Slab applications are typical of those that can be time consuming and expensive in terms of labour, due to the difficulty in distributing concrete over large areas.

Although there are specialist flooring techniques which require stiff concrete mixes, in the case of numerous other applications the use of more mobile concretes will speed up the pour and ease placement. In the United States of America and other parts of the world, concretes with initial slumps in the 150 to 200 mm range are readily accepted by the construction industry. Conversely mixes with consistencies much in excess of 75 mm slump are viewed with extreme caution and suspicion in the U.K. Moreover, the inclusion of an admixture into the concrete to improve its workability in the fresh state is regarded as undesirable in many cases, and permission is generally required from the specifying authority before use.

The inclusion of dispersing agents in concrete to temporarily increase its workability to collapse slump is an accepted practice elsewhere in the world, yet despite the efforts of the Cement and Concrete Association (1) and the Construction Industry Research and Information Association (2), to publicise the usefulness of these admixtures termed superplasticizers, usage in the U.K. has been minimal. However, superplasticized concrete has a prestigeous track-record elsewhere in the world, particularly in Germany and Japan. Howard (3) reported that approximately twenty six million cubic metres of concrete containing a superplasticizer has been used in a variety of applications over the past twelve years. Concrete with strengths ranging from 30 to 80 N/mm^2, with workabilities ranging from 40 to 200 mm have been used in such applications as bridges, chimneys, multi-storey structures, aqueducts and a test track for Japan's high speed railway system. In Germany, Freese (4) has described the efforts made by the ready mixed concrete industry to come to terms with flowing concrete, with particular reference to mix design, quality control and service. Hence the concept has been proven outside the U.K. and there is abundant evidence to suggest that superplasticizers are not only beneficial to concrete but are also cost effective.

THE CONCEPT OF FLOWING CONCRETE

Concrete mixes with slumps in the 150 to 200 mm range can be produced to required strengths using conventional materials without the aid of an admixture. In the U.K. mixes of this type are generally restricted to piling and diaphragm wall applications; however, in the U.S.A., for example, highly mobile mixes are commonly used for a variety of applications. The inclusion of an admixture of the plasticizer or superplasticizer type can provide similar workability ranges, and generally reduce the volume of water in the mix as well as the cement content. This was illustrated by Sweetland (5) who composed batch figures for similar concrete mixes, one containing an admixture the other without an admixture. These proportions are shown in Table 1.

Table 1 Similar Mixes to Attain Collapsed Slumps and Similar Strengths with and without Admixture

	MIX WITHOUT AN ADMIXTURE	MIX WITH AN ADMIXTURE
Mass of Cement, kg/m^3	410	350
Mass of Fine Material, kg/m^3	530	655
Mass of Coarse Aggregate, kg/m^3	1190	1190
Volume of Water, l/m^3	215	185
Initial Slump, mm	200	50 to 75
Final Slump, mm	200	200
Aggregate-Cement Ratio	4.19	5.27
Water-Cement Ratio	0.52	0.52

It is evident that the use of an admixture in this particular case results in a water reduction of 30 l/m^3 and a cement reduction of 60 kg/m^3, which should cover the additional cost of the admixture. Nevertheless, despite the cost comparison, the resultant water and cement reductions should ensure an improvement in the hardened properties of the concrete, particularly shrinkage as this is associated with both the water and cement contents within the mix.

The admixtures which can be used to temporarily increase the workability of concrete mixes are conventional plasticizers of the lignosulphonate and carboxylic acid type and superplasticizers. The Cement and Concrete Association (1) during their investigation into superplasticizing agents categorised them into four main groups as given below, suggesting that a flowing concrete is produced when the addition of a superplasticizer gives a mix with a collapse slump which does not exhibit bleeding, segregation, air entrainment and excessive retardation.

Category A. Sulphonated melamime formaldehyde condensates

Category B. Sulphonated naphthalene formaldehyde condensates

Category C. Modified lignosulphonates

Category D. Others

The function of superplasticizers in concrete has been described as a two phase reaction beginning with absorption of the superplasticizer on to the hydrating cement particles (6). The anionic nature of the admixture then causes the cement particles to become mutually repulsive, they repel each other and the fine particulate agglomerates break down and separate into smaller units. This absorption and dispension reaction not only disperses the cement around the mix in a more uniform manner, but also allows the water normally encapsulated in the agglomerates to lubricate the concrete mix creating greater workability.

Producing Flowing Concrete

Flowing concrete is generally produced by the addition of a measured volume of superplasticizer to a fresh concrete mix having an initial slump of about 75 mm. There are exceptions to this rule however; superplasticizers are extremely efficient water reducing agents and consequently concretes with low initial slumps can be given collapsed slumps by increasing the admixture dosage accordingly. Water reduced concretes of this type tend to be used for special applications, such as high strength concretes, where high early strength is advantageous or possibly where a special finishing technique is required as in the case of floors. Nevertheless the variety of mix designs that can be considered by including superplasticizing admixtures is vast, and beyond the scope of this paper. Concrete mixes with slumps in the 150 to 200 mm range can be produced using materials other than superplasticizers; although these mixes perform satisfactorily, in some cases they exhibit retardation or air entrainment which may be undesirable.

The extreme mobility associated with flowing concretes has a limited life, the effect wears off gradually over a period of 90 to 120 minutes until it has reverted to its initial slump. According to the Cement and Concrete Association (1) this gradual loss of workability is associated with the admixture type, the age of the concrete at the time of addition of superplasticizer and the environmental conditions. Ramakrishnan (7) has demonstrated that a 50 per cent slump loss can be expected within the first twenty to thirty minutes. Consequently, advantage should be taken of its flowing properties as soon as possible after addition of the admixture.

Mix design

Successful flowing concrete should have extreme mobility without the attendant side effects of segregation or retardation. To achieve this effect it was found that the cohesive properties of the concrete mix were important, and Tutt (8) suggested that a pumpable mix with an initial slump in the 75 mm range would be a good starting point. Referring to German practice, Freese (4) recommended that naturally rounded aggregates with a continuous grading should be used wherever possible. However, the most complete approach to mix design has been produced by the Cement and Concrete Association (1) and is summarized in Table 2.

Table 2 Mix Design for Flowing Concrete

First Approach	
Mix with continuous grading and initial slump of about 75 mm.	Increase sand content of aggregate by 4.5 per cent.
Second Approach	
Maximum aggregate size 38 mm and cement content $\geq$ 270 kg/m^3.	Use a minimum of 400 kg/m^3 of combined fines (cement + sand) with a particle size less than 300µm and 24 to 35 per cent of sand passing the 1.18 mm sieve.
Maximum aggregate size 38 mm and cement content $\leq$ 270 kg/m^3.	Use a minimum of 400 kg/m^3 of combined fines and 35 per cent or more sand passing the 1.18 mm sieve.
Maximum aggregate size 20 mm and cement content $\geq$ 270 kg/m^3.	Use a minimum of 450 kg/m^3 of combined fines, and 24 to 35 per cent of sand passing the 1.18 mm sieve.
Maximum aggregate size 20 mm and cement content $\leq$ 270 kg/m^3.	Use a minimum of 450 kg/m^3 of combined fines, and 35 per cent or more sand passing the 1.18 mm sieve.

Variations in the shape, grading and type of aggregate will alter the design of the mix, and the admixture dosage will vary accordingly. In locations where sands are devoid of fine particles,as in the case of marine aggregates, it may prove necessary to add a building sand or a proportion of an inert filler. Currently, it is advisable to produce trial mixes to assess the visual properties of the fresh mix before supplying to the site.

Addition of Superplastizers to the Concrete

Many of the admixtures used in concrete are added with the mixing water at the batching plant; this helps to disperse the agent uniformly throughout the fresh mix. Superplasticizers, however, should be added at the site just prior to discharge, firstly to utilise the full flowing effect as long as possible, secondly to avoid spillage during transportation in truckmixers. Hence, these requirements place additional disciplines on the full scale production of flowing concrete, as it requires initial measurement of the admixture at the plant with eventual addition on site.

The ideal situation would be to assess the dosage requirements at laboratory level and use these measured volumes for full scale production. This would result in a procedure where the admixture is metered from a plant dispenser into a container attached to the truckmixer and when the vehicle arrives at site the admixture is added to the mix and thoroughly mixed with the concrete by spinning the mixing drum prior to discharge. However, this idealistic system depends upon rigid control of the initial concrete properties, particularly workability, if standard dosages are to produce consistent quality flowing concrete. Consequently, this automated dispensing procedure is seldom used in the U.K., as the initial quality control of the base mix is too variable. Instead the admixture dose is measured out at site level by competent technical personnel, the amount varying according to the initial slump, then manually added to the base mix in the truckmixer.

Quality Control On Site

The visual appearance of flowing concrete can be disconcerting to construction personnel usually acquainted with conventional stiffer mixes. Furthermore, the absence of a British Standard test method for flowing concrete can only raise further doubts regarding its integrity. However, the Cement and Concrete Association (1) recommend that flowing concrete should have a slump equal to or greater than 200 mm and a compacting factor between 0.96 and 0.98 when measured according to B.S. 1881. The mobility of flowing concrete is normally assessed using the flow table method outlined in the German Standard DIN 1048.

The flow table apparatus comprises a board hinged along one edge; the upper portion being able to be raised to a height of 40 mm and dropped. The testing method is as follows: i) the flowing concrete sample is compacted in layers into a truncated slump cone centred on the flow table, ii) the cone is removed and the concrete sample is subjected to fifteen bumps on the table, and iii) the spread of the sample is measured in two directions and if the spread values are between 510 and 620 mm the sample is regarded as satisfactory. In addition the concrete should look uniform and not exhibit any bleeding or segregation. Although this method is accepted by many of the European countries, Dimond and Bloomer (9) suggested that it is extremely sensitive to operator error; for example, the board can strike the upper limit when being lifted thus creating a double bump. In addition if the table only has a single stop a twisting motion may be imparted to the bumping which could possibly affect the final spread value. They suggested that the initial value of spread, i.e. the spread of the concrete after taking off the slump cone, is as sensitive a measure of mobility as the final spread, and recommended that the bump test should cease before it becomes misleading.

There is no doubt, the flow table test can give misleading results, but currently it is the only test method generally available, and it is simple and robust enough for site use. Experience with flowing concrete has also shown that visual assessment of the material is extremely useful in assessing the adequacy of the concrete. A free orifice rheometer may prove useful in measuring the flowing properties of the concrete (10). In this case a measured volume of concrete is placed in a vertical cylinder, the lower door is opened and the time taken for the concrete to discharge or to block in the tube is measured. Although there is further development work required, the appartus has potential and it is both simple and robust for site usage.

In the U.K., the compressive strength of the concrete is judged by crushing cube test specimens. It is accepted practice to make these cubes on site according to B.S. 1881, requiring the cubes to be cast in layers with each layer being thoroughly tamped to achieve full compaction. This poses the problem for flowing concrete as to whether cubes should be made in accordance with B.S. 1881 or simply poured into the moulds. Malhotra (11) has shown that the strengths of compacted and uncompacted test specimens with flowing concrete were statistically similar. Consequently, it would appear that either method would suffice, but definite guidance is needed.

Long-Term Experience

There exists abundant data to support the claims that superplasticizers have no deleterious effects on either the fresh concrete or its properties in the hardened state. The similarity between the heat of hydration graphs of cement pastes with and without superplasticizers were observed by the Cement and Concrete Association (1). They concluded that apart from an initial delay in hydration caused by these admixtures, the hydration was unchanged in the presence of superplasticizers at the usual levels of dosage. Howard (3) reported that the strengths of eleven year old cores tested in Japan showed a 54 per cent increase in strength for superplasticized concrete compared with a 42 per cent increase for conventional concrete (without admixture) over that period.

Tests to assess the effects of shrinkage, durability and permeability of concretes containing superplasticizers were carried out by Blundell (12). In the case of shrinkage no significant differences were observed, but there was a favourable improvement in both durability and permeability.

FLOWING CONCRETE IN SLABS

It was stated in the Introduction that the placing of concrete into slabs could be labour intensive and time consuming when conventional immobile concretes were used. However, flowing concretes are ideal for slab applications as their mobility permits the placing of concrete with the minimum of effort. In fact, the concrete is poured into the shutters rather than placed, as illustrated in Figure 1, where flowing concrete is being discharged into a suspended slab surrounding a swimming pool. Although these concrete mixes are extremely fluid and will flow under their own head, they are neither self levelling nor are they self compacting as has previously been claimed. The thixotropic nature of flowing concrete ensures that the mobility is restricted, movement takes place when energy is put into the concrete, e.g. its own momentum on discharge, and it ceases when this energy is dissipated. This results in a material which can be re-activated by further concrete discharge or by vibration, thus to some extent the movement of the concrete can be controlled.

Ground Floor Slabs and Foundations

Flowing concrete is being used in ground floor slabs and foundations in ever increasing amounts, albeit in volumes which are still small in the U.K. A recent survey into the use of this concrete conducted by a ready mixed concrete company showed that 45 per cent of the flowing concrete supplied went into ground floor slabs. In these applications the concrete can be placed directly from the truckmixer, or an extension tube can be added to the discharge chutes to extend the discharge range accordingly. Freese (4) has described the use of static fixed chutes for pouring concrete into a large foundation, with the length of the tubes exceeding 25 m and in some cases incorporating 90° bends. The use of such tubes increases the flexibility of the truckmixer, and it results in concrete being placed further, as well as into inaccessible parts of the job.

Figure 1 Flowing Concrete

The concrete is usually spread in the shutters with rakes, it is pushed from the point of discharge to where it is required. This method is far more effective than using shovels, as in conventional practice, because of the mobility of the concrete. This placing method applies irrespective of the discharge system used, however it is seen to its best advantage where the concrete is discharged swiftly in large amounts, for example, using a pump or by direct discharge rather than a skip. In simple slabs with minimal reinforcement,vibration of the concrete can probably be dispensed with as the movement of the concrete gang in placing the concrete is sufficient to compact and/or release any entrapped air. However, with more complicated reinforcement some vibration is necessary; it not only releases any air and compacts the concrete but it also helps the movement of the concrete. The amount of vibration required can only be judged qualitatively, when the concrete is on the move it is difficult to over vibrate it, but when the concrete is in place common sense is the best yard stick.

Suspended Floor Slabs

These structural elements usually embody similar features as ground floor slabs with the addition of supporting beams which can be heavily reinforced. Again flowing concrete is extremely useful, as well as fulfilling the requirement for easy distribution over the slab it is also ideal for penetrating the reinforcement and reaching the soffit of the slab. This results in a dense concrete which is tightly packed around the reinforcing bars giving the correct structural element as designed.

Flowing concrete has been used successfully in waffle construction suspended floors. In all cases the concrete was laid speedily without difficulty and when the plastic pots were removed the soffitt had a perfect finish. The obvious way to place concrete in suspended slabs is by pump, it permits fast delivery of the fresh concrete in as near continuous conditions as possible. Although flowing concrete has been placed by skip into suspended slabs, it is a slower disjointed procedure and does not realise the full potential of flowing concrete.

Concrete Finishes

Flowing concrete can be finished using those methods normally used for conventional concretes, the most common system being the simple tamping beam. To achieve the best results the slab should be initially levelled off by sawing the tamping beam across the concrete, then when the concrete has reverted back to its initial slump a second pass can be made in the normal way. Excellent finishes can be obtained using both vibrating beams and power floats. The vibrating beam should be used after the extreme workability condition has worn off, in this way the possibility of producing laitance is significantly reduced. Power floating, however, can be used similarly to the procedure adopted for conventional mixes. In some cases, the initial slump of the concrete is deliberately reduced to 40 or 50 mm before adding the superplasticizer, this permits earlier power floating as the hydration process is speeded up.

Gibbins (13) has recommended power grinding as a cost effective method of producing high quality finishes, quoting an example where flowing concrete was laid, levelled off with a double vibratory beam and eventually finished off with a power grinder. In his opinion the method provides a durable high quality hard wearing finish, which is far superior to any finishes obtained by power floating. This procedure would appear to have potential, however further experience to prove its cost effectiveness would be desirable.

Economics of Flowing Concrete

The extra cost of producing flowing concrete is equivalent to approximately 10 per cent of the cost of conventional concrete. This surcharge of approximately £1.50 to £2.00 per m^3 must be equated with the savings in manpower and time in laying slabs, and also to the overall shortening of the construction programme. The economics of using flowing concrete have been proved in Germany, but this country has significantly higher wages for construction personnel than the United Kingdom, and this is an important consideration. In addition, it is doubtful whether contractors would wish to reduce their manpower to levels where they would inevitably leave the industry, as specialist trades are difficult to recruit and retain. Hence, there is an obvious requirement to actively plan a construction programme which incorporates flowing concrete, only in this way can a subjective monitoring of potential savings be realised. Hewlett (14) has carried out two simple cost analyses, one showed a positive saving by using superplasticized concrete, the other a marginal increase in cost but a substantial reduction in the programme time.

Precautions

Besides the obvious care required when producing mix designs for flowing concrete, it is worth considering the effect of bleeding with flowing concrete in slabs. There is evidence to suggest that a mix containing a superplasticizer is more prone to bleeding than conventional concretes, experiences in the field certainly indicate this and a laboratory investigation by Rhodes (15) lends weight to this argument. His work indicated that the rate of bleeding can be greater for concretes containing superplasticizers, the effect varying depending upon the cement content, the fineness of the sands and the amount of 10 mm aggregate present in the mix. This initial work by Rhodes certainly indicates that there is a need for more research on mix design and its attendant effects.

Extensive cracking has been experienced in slabs cast using flowing concretes with simple tamped or vibratory beam finishes. Subsequent investigation did not completely explain this phenomenon, although the presence of polythene under the slabs appeared to aggravate the situation. Consequently preventative measures to reduce the bleed water loss were taken, namely effective curing of the fresh concrete which was extremely effective in preventing cracking, and proper curing measures are strongly recommended.

CONCLUSIONS

Superplasticizers present the opportunity to produce concrete mixes which can be tailored specifically to suit the particular application in hand. There is sufficient long-term data to indicate that superplasticizers have no adverse effect on the properties of the concrete, and in many cases have a beneficial effect. Further work is required on mix design to produce flowing concretes with the optimum fresh properties without adversely affecting the hardened properties.

There is insufficient data available to produce subjectively any useful cost analyses, the benefits are obvious but can only be stated in qualitative terms at present. This can be best resolved by deliberately incorporating flowing concrete into a construction programme, although this would initially require specification status and the active participation of all parties. Furthermore the performance and benefits of flowing concrete must be accurately monitored without exaggeration as this would only detract from its eventual assessment.

Finally it is recommended that flowing concretes should be considered and used for slab application and foundations, with the proviso that their inclusion should be treated with the correct degree of caution if a successful job is to result.

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THE EFFECTS OF TIME LAG IN PLACING CONCRETE

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ABSTRACT Delays in delivery of ready mixed concrete, resulting in its rejection, may cause construction problems. This study investigated in great detail the reported results of casting delay and disturbance of concrete after casting for periods of up to 8 hours from mixing. It was found that provided adequate compaction can be attained, casting delays result in gain of strength, and thus workability is the key factor. Furthermore, placing fresh concrete in contact with partially set concrete with consequent disturbance of the latter does not significantly affect the strength of the resulting construction.

INTRODUCTION

The British Standard for ready mixed concrete (1) recommends a time limit of 2 hours for the discharge of concrete transported in a truckmixer or agitator from the time of the introduction of the mixing water to the cement and aggregates. For concrete transported in non-agitating equipment, a time limit of 1 hour is recommended. However, the given time limits may be extended by agreement between supplier and purchaser.

In his investigations into the effects of revibrating concrete (2), Vollick concluded that revibration 1 or 2 hr after placing increases the 28 day strength of concrete by an average maximum of 13.7 per cent. Evidence from Taylor (3), Lea (4) and Neville (5) shows that for a normal Portland cement mix that has not been subjected to remixing, a maximum compressive strength is obtained if casting is delayed 2 to 2½ hr after initial mixing. Intermittent mixing for up to 3 hr, and in some cases up to 6 hr, is harmless as far as strength and durability are concerned, although workability falls off with time.

This investigation was prompted by the need for more detailed information on the influence of delayed casting of concrete on the strength of the structure, and was carried out to study the effect of: (i) delayed placing of concrete which has been subjected to intermittent remixing, (ii) delayed placing of concrete which has not been subjected to remixing, (iii) revibrating half-set concrete and (iv) casting a new lift of concrete on to half-set concrete, with regard to the compressive strength of the bond formed.

EXPERIMENTAL WORK

Materials

Ordinary pozzolanic Portland cement from the Athi River plant was used in all the tests, with crushed-rock coarse aggregate and local sand. The combined grading of the coarse and fine aggregate was as shown in Table 1.

Table 1 Aggregate Grading

B.S. Sieve Size, mm	20	10	5	2.40	1.20	0.60	0.30	0.15
Mass Passing, %	93	58	41	23	20	14	5	1

The mix was designed for a 7 day strength of 23 N/mm^2, a water-cement ratio of 0.5, an aggregate-cement ratio of 4.6 and medium workability.

Mixing

For the tests carried out in connection with (i), (ii) and (iii) above, the weighed ingredients were mixed in a tilting drum mixer for 3 minutes after the required amount of water had been added. For that in connection with (iv) a pan mixer was used.

Casting

(i)a Cubes prepared on vibrating table until full compaction achieved. Immediately after the initial mixing, two 150 mm (6 in) cubes were cast, using 60 sec vibration, and taken into the curing room (21°C, 90% R.H.). The rest of the mix was left in the mixer whose mouth was then covered with a wet sack to minimise evaporation of water. After 1 hr the concrete was remixed for 2 minutes and two more cubes cast, vibrating for 60 sec, and taken for curing. The procedure was repeated at further 1 hr intervals with the last two cubes being cast 8 hr after initial mixing. For the first four sets of cubes, vibrating for 60 sec was enough to bring about full compaction; for the remaining sets, vibration was continued until the surface had become fully plasticised.

(i)b Cubes given a standard vibration of 60 seconds with vibrating poker. Immediately after the initial mixing, two moulds were filled to overflowing with concrete. The poker was immersed vertically at the centre of the cube and slightly above the bottom of the mould. After 60 sec it was withdrawn slowly to allow the void to close behind it. The cubes were then trowelled and taken into the curing room (21°C, 90% R.H.) and the mouth of the mixer covered with a wet sack. After 1 hr, the concrete was remixed for 2 minutes and the procedure repeated. For the last two sets of cubes, the depressions left by the poker did not fully close.

(ii) Delayed casting with no remixing. Immediately after initial mixing, the first set of two cubes were cast, vibrated for 60 sec and taken to the curing room (21°C, 90% R.H.). The rest of the mix was poured onto a wheelbarrow, covered with wet sacking and then left undisturbed. At intervals of 1 hr two more cubes were cast. In each case the duration for which the cubes were vibrated was increased progressively until the concrete surface had become plasticised.

(iii) Revibrating half-set concrete. After the concrete mix was ready, all the 18 cube moulds, which had been prepared and arranged on the vibrating table, were filled with concrete to well above their rims. The vibrating table was switched on for 45 sec and then 16 of the cubes were trowelled, transferred to the floor and covered with wet sacking. The remaining two were vibrated for another 45 sec and taken to the curing room (21°C, 90% R.H.). At intervals of 1 hr two more cubes were revibrated for 45 sec and taken for curing.

(iv) The relative strength of the bond formed when a new lift of concrete is cast on to half-set concrete. Sufficient material for casting 4½ cylinders (150 mm dia x 300 mm) was mixed in the standard manner using a pan mixer. Seven cylinder moulds were labelled 0, 0, 2, 4, 6, 8 and 10. The two moulds labelled 0 were completely filled with fresh concrete while each of the remainder was filled to half height. The concrete in each mould was vibrated for 90 sec using a vibrating poker. The moulds marked 0 were then trowelled and taken for curing. At intervals of 2 hr, a batch sufficient to fill 1½ moulds was prepared in the pan mixer. This was used to cast a new cylinder and to fill up one of the half-full cylinder moulds. The full mould was vibrated in the same manner as the others. For the new concrete cast on to the one in the half-filled mould, the poker was held 225 mm (9 in) into the cylinder so that the older concrete was penetrated to a depth of 75 mm (3 in). Vibration time was again 90 seconds. The procedure was repeated at intervals of 2 hr until all the cylinders had been jointed. All the cylinders were cured in the same manner as the cubes (21°C, 90% R.H,).

TEST RESULTS

The cubes were tested for strength at the age of 7 days, immediately after removing them from the curing room in accordance with the British Standard specification for testing concrete (6). The results obtained are given in Tables 2 to 5 and shown in Figure 1. In Figure 1, medium workability refers to concrete with slump of 120 to 5 mm, low workability refers to concrete with slump of 5 to 0 mm, and unworkable concrete refers to concrete giving a Vebe time greater than 30 sec. The time limits for the various workabilities were determined by performing separate slump and V-B consistometer tests on concrete that had been subjected to intermittent remixing and concrete that had been left undisturbed. The concrete used had the same mix proportions and grading as previously stated. The curves in Figure 1 were derived using least squares regression analysis assuming second order equations. The resulting corrected initial strengths were used in determining the percentage cube strength variations.

Each cylinder was gripped in a compression machine at a load of 18 kN. Using a rebound hammer held perpendicular to the cylinder surface, 15 hammer readings were taken around the middle of the unjointed cylinders. For the jointed cylinders, 15 readings were taken round each of: (a) the middle of the upper half (b) the joint and (c) the middle of the lower half. Each of the cylinders was then tested to determine the compressive strength. For each set of 15 readings, the rebound number R was determined as given in the manual (7). The maker's calibration curve was used to obtain the corresponding apparent compressive strength. The results obtained are given in Table 6.

Table 2 Time Lag Between Initial Mixing and Casting with Remixing at 1 Hour Intervals, Each Cube Compacted on Vibrating Table

DELAY, HOURS	CUBE CRUSHING STRENGTH, N/mm^2		
	Cube 1	Cube 2	Average
0	26.1	22.8	22.2
1	23.0	23.8	23.4
2	25.4	25.8	25.6
3	28.4	27.6	28.0
4	30.2	29.4	29.8
5	30.2	29.8	30.0
6	30.2	29.0	29.6
7	30.6	29.8	30.2
8	32.0	32.0	32.0

Table 3 Time Lag Between Initial Mixing and Casting with Remixing at 1 Hour Intervals, Each Cube Compacted with Vibrating Poker

DELAY, HOURS	CUBE CRUSHING STRENGTH, N/mm^2		
	Cube 1	Cube 2	Average
0	19.7	19.9	19.8
1	21.6	21.6	21.6
2	22.8	22.0	22.4
3	25.0	25.0	25.0
4	26.7	27.7	27.2
5	27.7	27.1	27.4
6	30.1	27.7	28.9
7	32.8	33.2	33.0
8	29.8	31.0	30.4

Table 4 Time Lag Between Initial Mixing and Casting; No Remixing of the Concrete

DELAY, HOURS	CUBE CRUSHING STRENGTH, N/mm^2		
	Cube 1	Cube 2	Average
0	23.8	23.8	23.8
1	26.3	25.3	25.8
2	27.3	27.1	27.2
3	25.6	25.4	25.5
4	26.3	25.3	25.8
5	28.0	26.0	27.0
6	26.0	23.2	24.6
7	22.0	17.7	19.8
8	6.2	10.7	8.5

Table 5 Time Lag Between Initial Vibration of 45 seconds and Revibration of 45 seconds

DELAY, HOURS	CUBE CRUSHING STRENGTH, N/mm^2		
	Cube 1	Cube 2	Average
0	29.8	30.2	30.0
1	31.0	28.0*	31.0
2	32.8	33.2	33.0
3	31.9	32.9	32.4
4	32.8	32.8	32.8
5	30.5	31.9	31.2
6	31.5	32.9	32.2
7	30.5	32.3	31.4
8	28.4	29.2	28.8

*result disregarded

Table 6 Effect of Jointing Fresh to Partly Set Concrete

DELAY BEFORE JOINT IS MADE, hr	CYLINDER STRENGTH AS INDICATED BY IMPACT HAMMER AT MIDDLE OF INDICATED SECTION, N/mm^2				ACTUAL CYLINDER CRUSHING STRENGTH, N/mm^2	
	Lower Half	At the Joint	Upper Half	Control Specimen	Jointed Cylinder	Unjointed Cylinder
0	-	-	-	13.7	-	20.3
2	15.5	14.5	14.5	13.0	20.8	22.5
4	15.5	15.0	15.0	13.5	19.8	14.5
6	16.5	14.5	15.0	13.0	20.3	20.6
8	17.0	12.0	16.5	14.5	20.3	18.1
10	13.0	13.0	14.0	13.5	18.7	18.4

DISCUSSION

It is clear from Figure 1(a) that delay in the casting of concrete that is being subjected to intermittent remixing results in a parabolic gain in strength providing the concrete is adequately compacted. Since workability deteriorates with time of delay, prolonged vibration is necessary to achieve adequate compaction. In this respect a vibrating poker is more efficient than a vibrating table since the concrete compacted by the former showed a maximum gain in strength of 62 per cent occurring in concrete cast 8 hr after initial mixing, while the corresponding concrete compacted by the latter showed a maximum gain in strength of 42 per cent. However, after 6 hr such concrete becomes too stiff to be used on most concreting jobs.

In contrast to the strength - delay time relationships shown in Figure 1(a), the delayed placing of concrete without remixing resulted in a gain in strength in the initial stages only, as shown in Figure 1(b). The maximum gain in strength was 25 per cent occurring for concrete cast 3 hr after initial mixing. Casting such concrete more than 5 hr after initial mixing results in a rapid loss of strength. Workability deteriorates faster, Figure 1(b), than in the case where

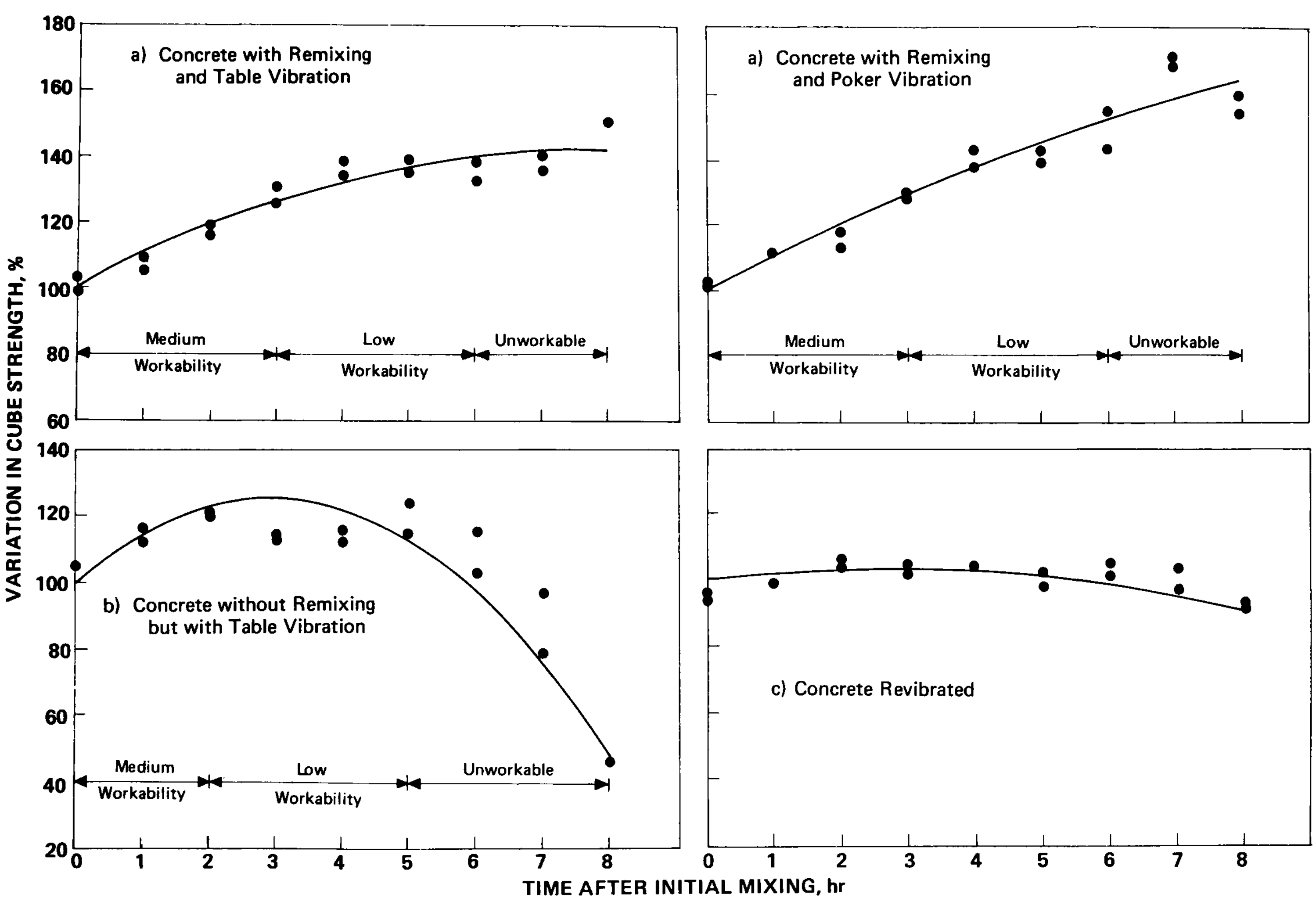

Figure 1 Effect of Delays After Initial Mixing of Concrete on its Strength

the concrete is subjected to intermittent remixing, Figure 1(a).

Revibrating half-set concrete results in higher strengths than those obtained for the same amount of initial vibration if revibration occurs not more than 5 hr after the casting of the fresh concrete, Figure 1(c). The maximum gain in strength is about 2 per cent, obtained for concrete revibrated 2 to 3 hr after casting. Revibrating concrete after this period causes a strength loss.

The results in Table 6 show that the apparent strength of the joint was lower than that of either the upper lift or the lower lift, but the differences were not generally significant, from which it would seem that the jointing did not have an adverse effect on the strength.

CONCLUSIONS

The time limits given in B.S. 1926 concerning specifications for ready mixed concrete (1) would appear to be based on a criterion of workability since the experimental results indicate that these time limits could be extended up to 5 to 6 hr providing the concrete can be adequately compacted.

In the case of vibrating half-set concrete, the test results are similar to those obtained by Vollick (2) except that the increase in strength obtained (2.5 per cent) is much less than that reported by him (8.5 to 17 per cent). This difference may have been due to the different aggregate grading, aggregate-cement and water-cement ratios. Thus, it is suggested that further tests should be made to investigate the effect of such variations on the results obtained in this study.

The major application of the results reported in this paper is considered to be in bridge piers, abutments and columns. However, in slab construction the conclusions obtained concerning delays due to transportation of ready mixed concrete are equally applicable, and furthermore the effect of revibrating partly set concrete has relevance to the effect of such delays on the casting of alternate slab panels. The tests on jointed cylinders, while primarily concerned with compression members, could also have practical applications to the construction of T-beam and slab joints.

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SHRINKAGE AND CRACKING OF CONCRETE AT EARLY AGES

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ABSTRACT Cracking of concrete slabs in the first 24 hours after they were placed in a wind tunnel was investigated, using different air humidities and wind velocities, as well as different concrete mix compositions. Simultaneously with the observation of cracking, horizontal and vertical shrinkage, water loss and ultrasonic pulse velocity were measured. The relations between evaporation of water, shrinkage, cracking at early ages and properties of concrete are discussed on the basis of the experimental results.

INTRODUCTION

There exists considerable literature on shrinkage cracking, especially regarding the influence of atmospheric conditions and dimensions of concrete members (1 - 5). It is also known that material properties of concrete are of secondary importance compared with the atmospheric influences, but a better knowledge of their influence may help to contribute to the practical prevention of shrinkage cracking.

The causes of cracks in concrete may be attributed to its mechanical properties, temperature and the evaporation of water from concrete. In practice, different causes may occur simultaneously and intensify each other. This paper deals only with cracks in concrete due to drying out at early ages. In particular, the relationships between water evaporation, shrinkage, deformability and cracking are discussed.

TESTING EQUIPMENT AND TESTING METHOD

At a constant curing temperature of 19 ± 1°C, concrete slabs, 2000 mm x 500 mm and 50 mm deep, were exposed in a wind tunnel to different air velocities. The drying shrinkage of the concrete was restrained by the reinforcement rigidly attached to the moulds; no cracks were observed with slabs reinforced only on the narrow sides, nor on smaller specimens. On other specimens from the same concrete mixes, the water loss by mass, the shrinkage in horizontal and vertical directions, and the ultrasonic pulse transmission time was measured in the same tunnel. The measurements were carried out from ½ to 24 hr after the addition of the mixing water. All concrete specimens were made using the same aggregate, complying with the Fuller grading curve and consisting of Swiss standard sand and natural gravel, 4 to 16 mm in size. Thus, the influences of aggregate and temperature were excluded from the investigation.

The effect of the following factors was examined, using a total of 104 concrete mixes.

1) Atmospheric conditions: humidity of air, 40 to 60 per cent; wind velocity, 0 to 10.35 m/sec; and time of wind exposure. During calm periods, the slabs were protected against evaporation by plastic sheets.

2) Concrete composition: with one cement, the cement content was varied from 228 to 550 kg/m^3 and the water content from 147 to 225 ℓ/m^3; and with 11 different cements, the water content was varied with a constant cement content of 300 kg/m^3.

TEST RESULTS AND DISCUSSION

Crack Types and Atmospheric Conditions During Their Formation

Some slabs developed one single crack of a few centimetres in length, while others developed several cracks of up to 200 cm total length. The location and length of cracks were not reproducible, as they occurred in fortuitously weak regions of the inhomogeneous concrete. Whether or not cracks occurred, however, was reproducible. With very few exceptions, the cracks went through the whole thickness of the slab. In order to obtain unequivocal results, only those cracks were considered which could also be seen on the opposite side of the slab.

The predominant role of wind velocity and time of wind exposure could be demonstrated with the experiments. The cracks were formed even at a low wind velocity, with their length increasing with increasing wind velocity. There is a critical period of time, between about 2 and 4½ hr, after the addition of mixing water when the rapid evaporation of water due to wind exposure was found to cause cracking of concrete. No cracks were formed, however, if the wind was turned off during this short period of time, even though the wind had blown with maximum velocity before and afterwards, see Figure 1.

On the other hand, cracks were formed if the wind was blowing only during this critical period. Also with persisting wind, crack formation could exclusively be observed between 2 and 4½ hr after the addition of mixing water. Slabs which had not cracked up to this time did not crack at all, even with maximum wind velocity up to 24 hours. It was interesting to note that the slabs did not crack even though their degree of drying and shrinkage exceeded that of the slabs which had cracked during a short period of exposure to wind, see Figure 2. Thus, it would appear that evaporation as well as drying shrinkage of concrete is greatly influenced by the wind, while cracks at early ages occur only when intensive evaporation takes place during a short, but specific, period of time, in this case between 2 to 4½ hr after the addition of mixing water.

Extent and Evolution With Time of Evaporation and Shrinkage

The drying shrinkage is measured generally on the hardened concrete, beginning at the age of 24 hr with its final value, after months and years, being 0.03 to 0.06 per cent, depending on atmospheric conditions. Considerably more (0.065 to 0.34 per cent) drying shrinkage was measured within the first 8 hours. During this period, the concrete had lost up to $^1/_3$ of its mixing water. The decrease in water content and concrete volume at early ages also influences the other properties of

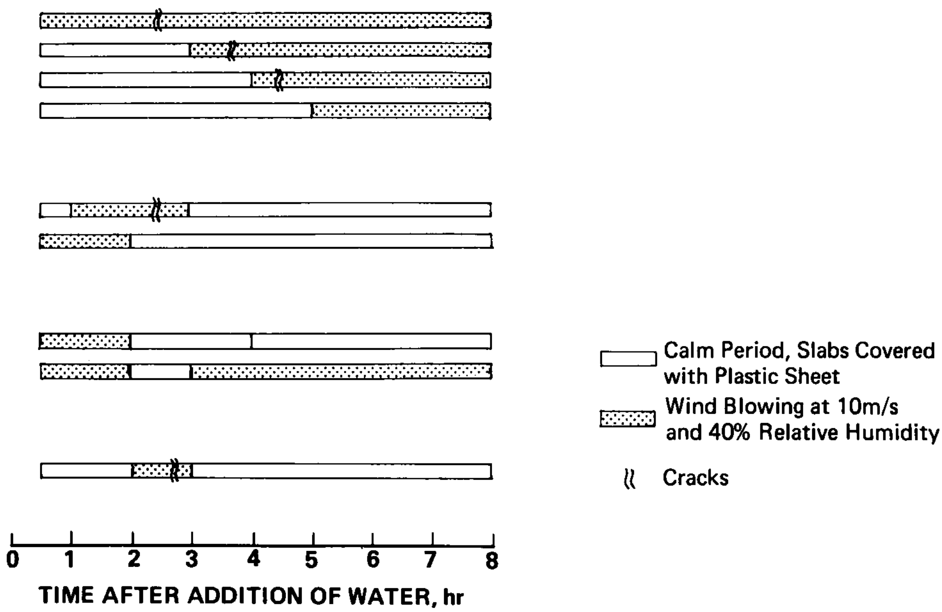

Figure 1 Influence of Wind on Crack Formation

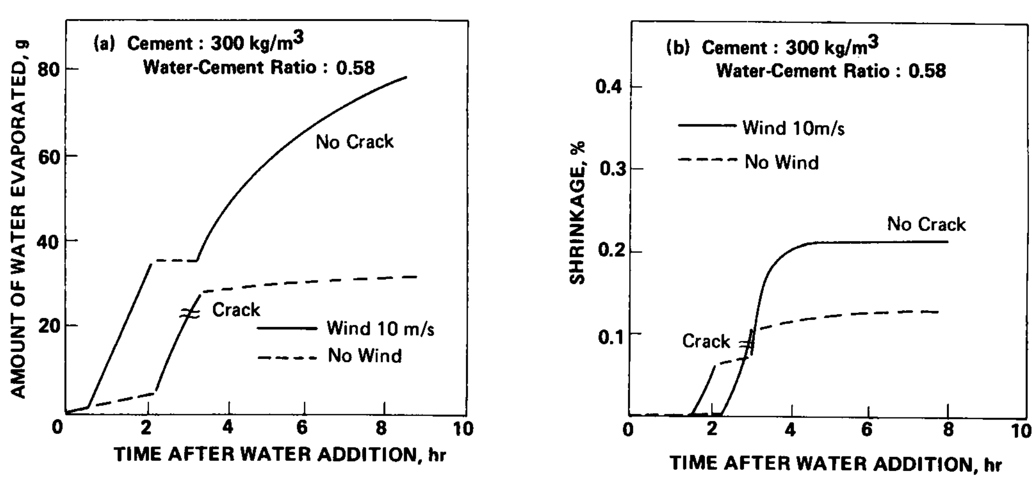

Figure 2 Influence of Wind Periods

the hardened concrete. Concretes which were allowed to dry out during the period immediately after placement had up to 20 per cent higher strength and also higher modulus of elasticity compared with concretes of the same composition which had been stored in the moist environment throughout. Nischer (6, 7) has also reported improved wear and freeze/thaw resistance due to early drying out.

It was found that under constant atmospheric conditions evaporation soon reaches its maximum rate, which then decreases rapidly, while shrinkage reaches its maximum velocity later, when evaporation has slowed down, see Figure 3.

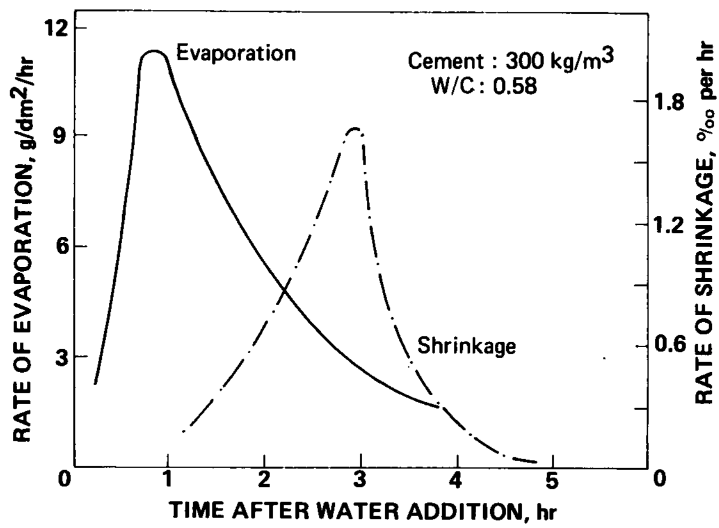

Figure 3 Rate of Evaporation and Shrinkage of Concrete

In agreement with the literature, no shrinkage could be measured in a first phase, terminating between 1½ and 2½ hours. The main part of the shrinkage takes place during a short second phase, lasting 1 to 1½ hours. After this, shrinkage proceeds only at a slow decreasing rate (third phase).

Measurements in a vertical direction showed that the concrete surface is lowered, Figure 4, and the stronger the wind and the lower the water content of the concrete, the more it shrinks. Therefore, it is not only a settling by gravitation, but rather a true drying shrinkage. This vertical shrinkage reached an average of 0.31 per cent with a maximum of 0.78 per cent for a prism height of 10 cm, before the horizontal shrinkage started.

Cracking was observed exclusively during the second phase with the most rapid horizontal shrinkage, see Figure 5(a). This phase of shrinkage is thus the most critical stage for the concrete with regard to shrinkage cracking.

The relationship between the decrease in volume of the concrete and the released water was found to vary considerably with the age of concrete, on average it was 1 : 5 during the first phase, almost 1 : 2 during the second phase and thereafter 1 : 15. The differences in these ratios, depending on concrete composition and age of concrete, are obviously caused by the varying deformability of the concrete. Measurements of the ultrasonic pulse velocity, made at 15 min intervals, indicated the development of the concrete's resistance against deformation. The examples in Figure 5(b) show that the velocity of sound in the concrete suddenly increases rapidly after a period of slow increase. This takes place before the initial setting of the cement measured by means of the Vicat needle. According to Figure 5(a), the rapid shrinkage of phase 2 and crack formation occur before the rapid increase of the sound velocity.

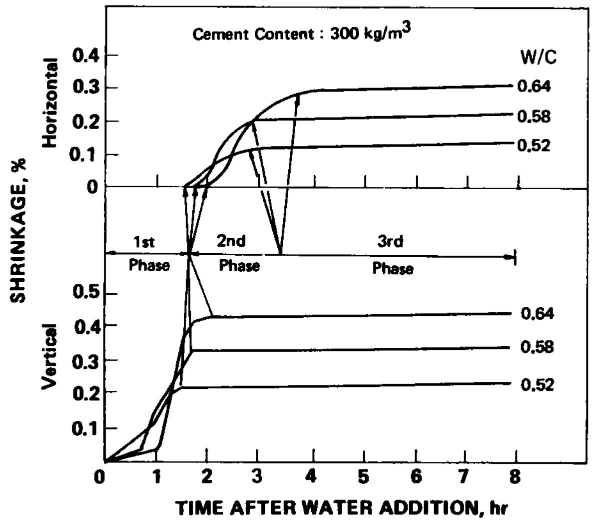

Figure 4 Effect of Water-Cement Ratio on Vertical and Horizontal Shrinkage

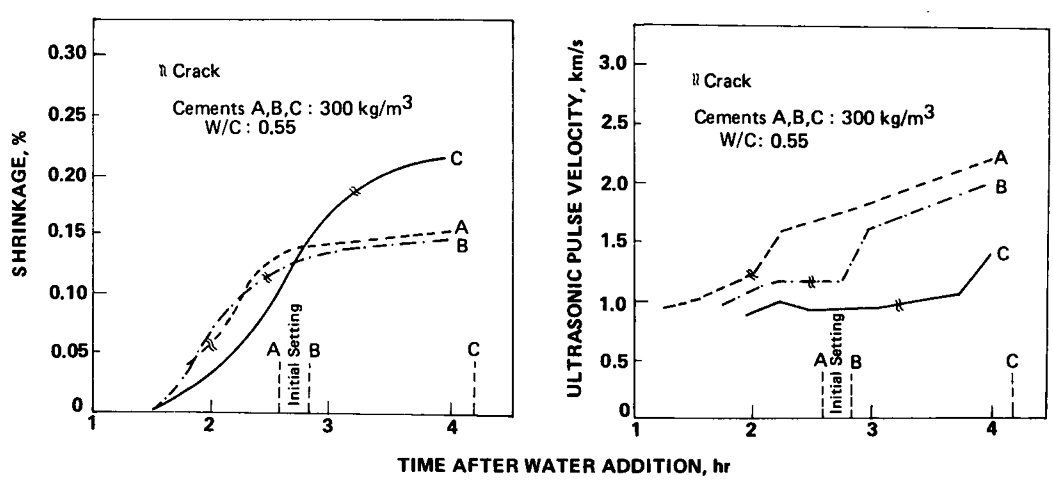

Figure 5 Time of Cracking

Influence of Concrete Composition on Evaporation and Shrinkage

Influence of Cement Properties

The examples of concretes made with different cements having equivalent water demand of standard paste, Figure 5(a), show that with equal concrete compositions, the shrinkage is more extensive and prolonged the greater the initial setting time of the cement used. Another influencing factor is the water demand of the cement measured on standard paste and it was found that the lower the water demand, the higher the shrinkage of concrete of the same composition. The evaporation of water itself was found to be related similarly to the initial setting time and water demand of the cement used. In practice, a comparison of concretes of similar consistency may be of more interest. By adjusting the water-cement ratio to the required consistency, the effect of the water requirement and the setting time of the cement are compensated and hence the influence of the cement quality in most cases is negligible.

Influence of Concrete Proportions

From Figure 6 it can be seen that the rate of evaporation of water from concrete made with the same cement increases as its i) water content increases, while the cement content remains constant (i.e. water-cement ratio increases), ii) cement content, and hence cement paste content, increases (i.e. water-cement ratio remains constant) and iii) cement content is reduced with constant water content (i.e. water-cement ratio increases).

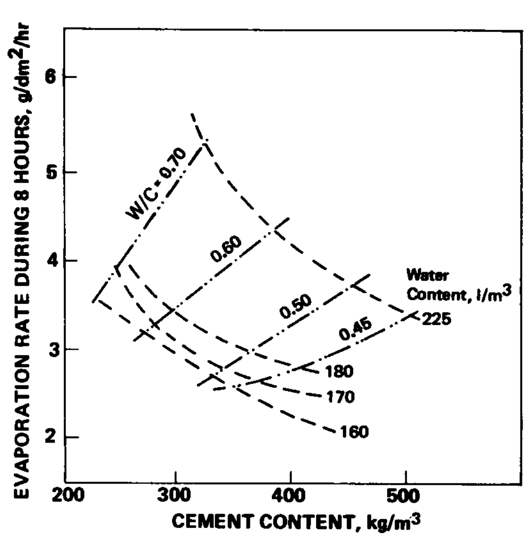

Figure 6 Influence of Concrete Composition on Evaporation Rate

It can thus be seen that the water content of the concrete is a decisive factor, with water evaporation being promoted by a higher water content, although this influence is diminished by a higher cement content or by an increased water requirement of the cement with the same concrete proportions.

Shrinkage is influenced by the concrete composition in a similar way. The relation of the shrinkage to evaporation, however, is not linear, as illustrated in Figure 7. If the water content exceeds a certain value, the ratio between shrinkage and water loss is reduced. This observation has a direct significance in connection with crack formation.

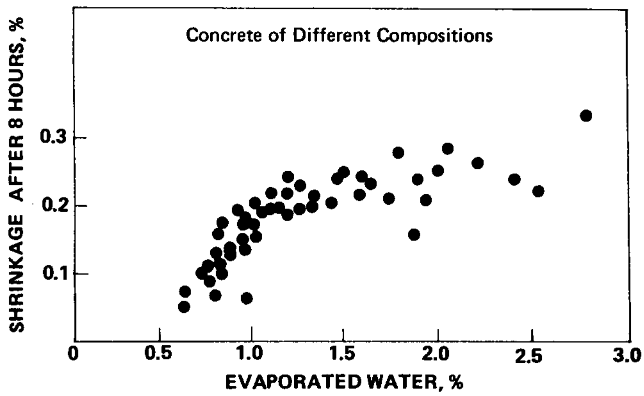

Figure 7 Relation Between Shrinkage and Water Evaporation

Influence of Concrete Composition on Crack Formation

The influence of concrete composition on the development of cracks is a complex phonomenon since different concretes a) release different amounts of water under equal external conditions, b) have different shrinkage values despite equal water loss and c) can tolerate different shrinkage without cracking, depending on their mechanical properties.

Cracks were formed with different shrinkage values (0.01 to 0.04 per cent) and water losses (0.5 to 1.7 per cent of concrete mass). Other concretes remained without cracks despite shrinkage values of over 0.2 per cent and a water loss of more than 3 per cent.

The results in Figure 8 show that concretes containing less water than 160 ℓ/m^3 remained without cracks even at maximum wind velocity; on the other hand, very soft or fluid concretes also did not crack.

The composition of concrete and quality of cement do not exert any influence for variations over a wide range. The setting of the cement can influence cracking insofar as cracking may occur earlier and at a lower shrinkage if the cement is setting rapidly, see Figure 5.

The Phases of Shrinkage and Cracking

The shrinkage, the reduction in the bulk volume of the concrete, is only a fraction of the volume of the water loss (8) with the larger part of released water coming from pores in the concrete. Furthermore, the shrinkage does not depend on the water loss alone, but also on the actual deformability of the concrete.

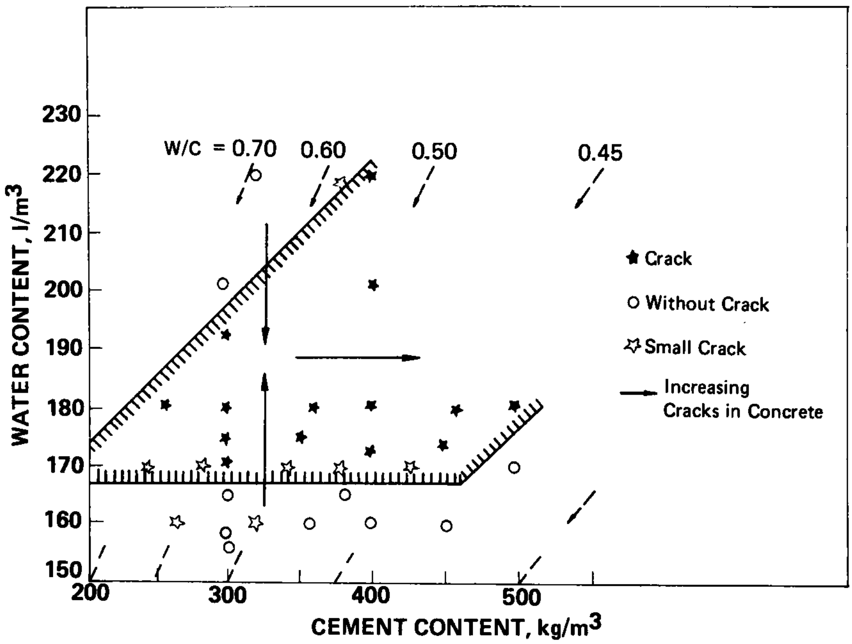

Figure 8 Influence of Concrete Composition on Cracking of Concrete

During the first phase, the low resistance against deformation permits the concrete to adapt itself to the mould. It does not shrink at all, or only in the vertical direction, without danger of cracking. In the second phase, the increased cohesion of the concrete enables it to withdraw from the walls of the mould. Since its resistance against deformation has increased only slightly, the evaporation of water causes rapid shrinkage. Thus, the danger of cracks arises as long as the tensile strength of the concrete increases more slowly than the induced stress. Then, the rapidly increasing resistance against deformation, see Figure 5, will be accompanied by a retardation in shrinkage and a faster increase of the tensile strength. By the combination of both effects, the danger of cracking at early ages ceases in the third phase. Cracks which occur later must have other causes.

Thus, cracking depends on the mutual ratios of water loss, deformability and strength. The ratios change continuously because of their different rates of development. Very stiff or rapidly stiffening concretes may not have a first or second phase. There is no danger of cracking if the development of the cohesion is faster than that of shrinkage or when drying out is retarded. With regard to the role of consistency and stiffening, even small quantities of additives and admixtures which modify these concrete properties may considerably influence the shrinkage and cracking behaviour of the concrete.

CONCLUSIONS

For practical purposes, the following conclusions may be drawn.

Shrinkage cracking at early ages occurs only if several unfavourable atmospheric conditions coincide, resulting in heavy evaporation during the critical period between 2 and 4 hours after mixing and placing. Shrinkage cracking of concrete can be avoided by employing mix proportions which reduce evaporation during this critical period of time. Protection and curing afterwards is required only for the purpose of hydration and strength development.

Water loss and shrinkage at very early ages have a beneficial effect on the quality of the hardened concrete without danger of cracking. This is due to the lowering of the water-cement ratio and the self-compacting effect. Thus, it seems appropriate to allow concrete to dry out, or even promote evaporation as long as the horizontal shrinkage has not yet become rapid.

Concrete composition is of importance only insofar as extremely dry as well as extremely fluid concretes are less susceptible to shrinkage cracking. With only a few exceptions, the type of cement has little influence with a given concrete consistency. Slower setting prolongs the initial period without danger of cracking. Additives and admixtures modify the cracking behaviour by their influence on water requirement and stiffening rate.

The tendency towards cracking of various concretes cannot be judged by their shrinkage values, since it depends also on their mechanical properties. A comparison is possible on the basis of concrete proportions, water requirement and setting time of the cement.

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THE CONSTRUCTION AND PERFORMANCE OF GROUND SUPPORTED AND SUSPENDED CONCRETE SLABS SUBJECT TO ABNORMAL WEAR IN RHODESIA

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ABSTRACT The author presents some of his experiences while in practice as a consulting engineer, in responding to the demands of industrial clients for increased life of floors that are subjected to abnormally harsh use, in a semi-developed country with severely restricted access to imported materials and expertise. The criteria to be met are set out, together with the measures taken to meet them, high-lighting some areas of particular difficulty. Four very different examples of floor use are described, of which three (two with an integral finish and one with an applied finish) were successful and one with an integral finish was a failure.

INTRODUCTION

The position of Rhodesia regarding technological development is ambiguous. It is very much a part of Africa in its struggle to make the giant stride from the age-old tribal society of semi-nomadic farmers, hunters and warriors of the nineteenth century to the technological world of the twentieth century in a few short years. On the other hand it possesses a sufficiently sophisticated infra-structure for it to be classified a developed country in some areas, such as that of educational literature. Perhaps, therefore, it is well placed to adapt modern techniques to primitive conditions. Furthermore, in recent years, Rhodesia has been the victim of international trade embargoes which have forced it to depend on its own resources wherever possible, rather than on importation of materials and techniques. When one surveys the chaos on the international monetary scene resulting from trade imbalances the benefits of such a situation are apparent.

This paper describes the responses to harsh demands in the circumstances described and seeks to show how the author has reached his conviction that a nominal Grade 20 concrete can be adapted to meet most demands. Much of what follows appears to be self-evident in retrospect. Nevertheless, the conclusions were only reached after the elimination of many other possibilities and the exploration of many blind alleys. The legal implications of sanctions regrettably preclude the naming of names and enforce certain undesirably vague descriptions.

LOCAL CONSTRAINTS

Many of the local conditions will be applicable to developing countries but not to developed countries, and it is necessary to set these out to allow proper evaluation of what follows.

The cost structure is obviously influenced by these considerations. Concrete is cheap in Rhodesia. Cement is manufactured locally from readily available raw materials. It is a good quality cement, approximating to the U.K. standards for rapid-hardening cement (1), but with a slow initial set allowing 45 to 60 minutes to elapse between mixing and placing even in the absence of special precautions.

A blend of 85 per cent normal Portland cement with 15 per cent of Portland blast furnace cement, which is commonly used in Rhodesia, was employed for the studies reported in this paper.

Much of Rhodesia is granitic. The decomposition products include a quartz sand which is deposited in the river beds from which it is dredged when water is low and frequently taken direct to the batching plant without further treatment. There are also extensive ancient sedimentary deposits of such sand, which tends to be finer than the river sand and is suitable for blending to ensure an adequate fine fraction. There are vast areas of unweathered granite which are quarried for coarse aggregate which tends to be elongated, flaky and dusty, but is hard and durable. Due to this, concrete mixes tend to be much more sandy than the traditional British mixes. The old 1 : 2 : 4 mix is practically unworkable without, for example, careful aggregate selection and use of plasticizing admixtures. In its place the proportions of cement to total aggregate are retained with increased fine and decreased coarse fractions, sometimes with nearly equal quantities of each. Crushed limestone aggregates are also used. Mix design is invariably based on water demand in accordance with South African practice (2). Aggregate specifications closely follow British practice (3), but aggregates frequently do not comply with respect to the grading envelope. This was sometimes the case in the contracts described. It was, nevertheless, possible to manufacture excellent concrete. The grading curves for the aggregates tested for one site are shown in Figure 1.

The cost differential between skilled and unskilled labour is very marked, giving traditional techniques a distinct advantage over those requiring specialist skills. Finally, climatic conditions are favourable in Rhodesia, it is seldom either cold enough or hot enough to require special precautions and rain is generally predictable enough to be allowed for in pre-planning, and conditions conducive to premature drying shrinkage only occur for a short period.

Some indications of current costs are given in Table 1 (Nov. 1978); it was difficult to decide on a currency base, in view of the rapid fluctuations.

GENERAL CONSIDERATIONS

Especial needs require especial responses and any specification for a hard-wearing floor must be tailored to meet specific challenges; nevertheless, general principals have been evolved.

Problem Areas

<u>Cost effectiveness.</u> All floors were for industrial concerns. Without exception the clients were prepared to meet any cost that would be effective in resulting in proportionate (or better) benefits in life expectancy of the floor, but were reluctant to commit themselves to an expensive technique which was not demonstrably cost effective. This eliminated many sophisticated modern materials which may be most effective in other environments but are adversely affected by the peculiar economic circumstances.

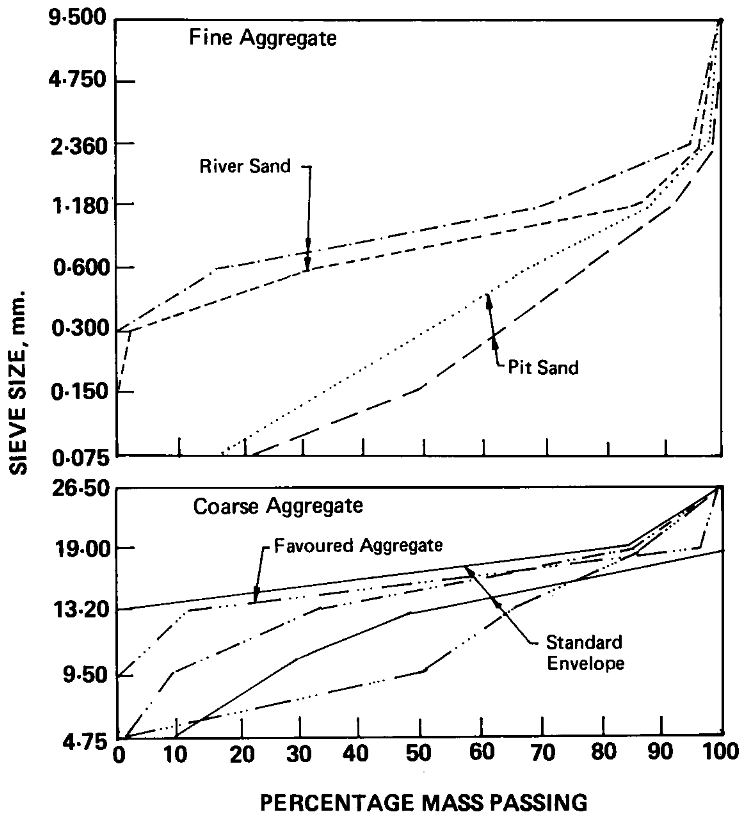

Figure 1 Grading of Typical Aggregates (a) Fine Aggregate (b) Coarse Aggregate

Table 1 Comparative Costs of the Constituent Materials per m^3 Concrete, November 1978 prices

	Rhod \$	S.A.Rand	Swiss Fr.	U.S.\$	U.K.
Cement	1.85	2.28	4.42	2.73	1.39
20mm Crushed Aggregate	4.68	5.78	11.19	6.90	3.50
River sand	5.60	6.91	13.39	8.26	4.19
Pit sand	4.50	5.55	10.76	6.64	3.37
Grade 20 Ready-Mixed Concrete	25.90	31.96	61.93	38.19	19.39
Floor Complete from d.p.c. Upwards 1976 Tender Price, m^2	5.80	7.16	13.87	8.55	4.34
Add, say, 38% for Escalation	8.00	9.87	19.13	11.80	5.99

Applied or integral finishes. Applied finishes fall into two categories. There are the above-mentioned sophisticated materials which often take the form of a thin skin applied to the surface of the base slab. These are found to increase cost by a factor of 2 to 200. Certain of these have been laid in small panels for extended evaluation. So far none has been found to yield benefits proportional to extra cost, and so none has been adopted for large-scale use.

The other category is the traditional granolithic or similar cement-based wearing course laid on a base slab. It is not news that such slabs do not, in general, withstand severe impact and abrasion, not least of the reasons being the fondness of the paviour for finishing the surface with a skin of almost neat cement for the sake of a short-term cosmetic effect. Thus, efforts have been directed towards integral finishes on conventional concrete in most cases.

Strength. It is common cause that a strong floor is generally a durable floor. However, with concrete, strength is associated with high cement content and high cement content is associated with high shrinkage. This leads to increased shrinkage cracking, increased opening of contraction joints and increased lifting of panel corners, which, lacking support, then tend to crack off under heavy loads. A cement content of about 300 kg/m^3 is accepted as the minimum necessary for durability, and is normally applicable to a Grade 20 to 25 concrete. On the other hand a hard-wearing floor should be of Grade 40 or more. Efforts were therefore directed at achieving Grade 40 concrete with a cement content of 300 kg/m^3.

Durability. Strength is not the only criterion for long life. The cement-sand matrix abrades fairly readily, while the aggregate does not. Attention was therefore given to maximizing aggregate content without sacrifice of cube strength and also to ensuring that the aggregate remains uniformly distributed up to the surface of the concrete instead of being overlaid with a substantial thickness of laitence.

Shrinkage. Low cement and high aggregate contents help to limit shrinkage, but other factors contribute. Attention was given to dealing with both the causes and effects of shrinkage.

Planning. Even if success attends one's efforts to extend the life expectancy of a floor from, say, 5 years to 10 years, nevertheless this remains a short life, and it will have to be replaced eventually. It is necessary to plan for this replacement before initial construction commences.

Joints. Experience has shown that break-up of concrete slabs almost always commences at a joint. Clients have pleaded for floors with no joints, but it has been deemed better to give attention to the provision of durable joints. Perhaps the worst culprit is the traditional V-joint, so beloved of paviours, which has perhaps done more to cause floor disintegration than would the elimination of joints. Due to their over-riding importance, joints are dealt with first in what follows.

SPECIFIED RESPONSES

Joints

Joints of all sorts lead to an unsupported face perpendicular to the surface of the concrete. Impact and abrasion in their vicinity cause the concrete to break back from this edge. Remedies consist in maintaining the support of the edge or protection of its vicinity from impact and abrasion.

Expansion Joints

These are sparsely provided due to the author's lack of conviction that concrete expands with temperature more than it initially shrinks. Wherever possible the floor is cast after construction of walls and roof and in some buildings where the process requires close control of temperature and humidity, no expansion joints have been provided. Where they are provided, they are positioned to suit construction planning, adjacent to walls, floor ducts and other features where construction is in any case interrupted. Where their presence in heavily trafficked areas is unavoidable the detail shown in Figure 2 is satisfactory.

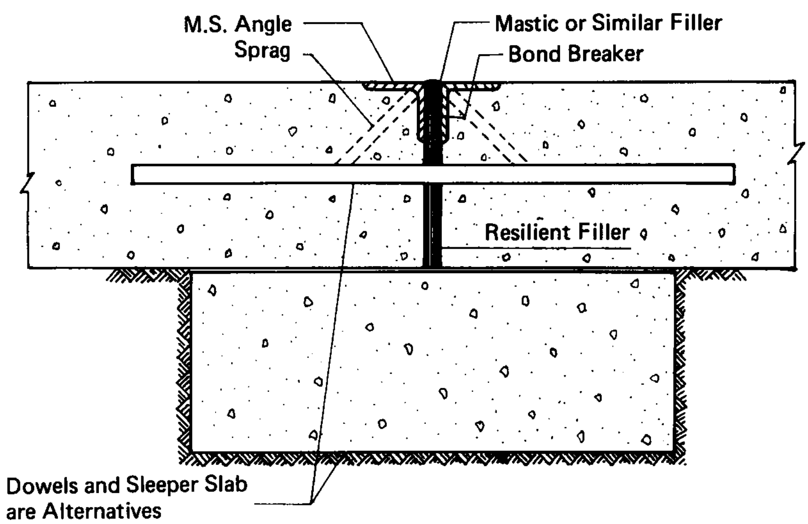

Figure 2 Section of an Expansion Joint

Careful and secure fixing of these inserts to formwork is necessary to ensure that they are flush with finished concrete to avoid over-sailing by a thin skin. The tendency of steel angles to enclose slightly less than a right angle needs to be watched.

Construction Joints

These are formed as shown in Figure 3. Edge forms must be in good condition with the top edges true and set accurately to level. There is a tendency for laitence to over-sail the top of the form forming a fin which leaves a rough corner when the form is removed, which subsequently initiates deterioration. For this reason insertion of a soft metal strip (brass or aluminium) is favoured. This is set

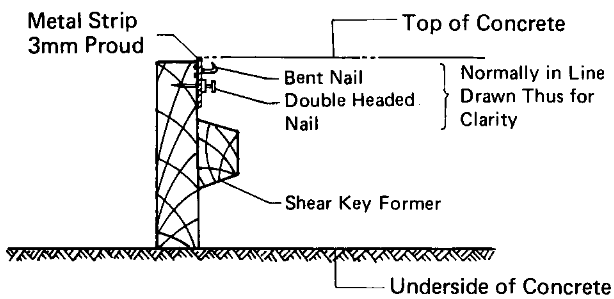

Figure 3 Section of an Edge Former

slightly proud of the form, otherwise it tends to get buried in the over-sail. It thus forms a bright edge to which screeding can be worked and eliminates fins. In use, it protects the edges of the panels and wears down with the concrete. Difficulty occurs with striking of the form, when the strip tends to come away with it. This is obviated by drilling the strip at about 250 mm centres and leaving bent nails projecting into the new concrete alternately with double headed nails fixing it to the form. Similar arrangements are made with metal forms.

Early stripping is avoided to minimize damage, formwork being left in position at least until the next day but one after pouring. The shear key is regarded as necessary to prevent development of an edge to initiate wear. Although cracks develop along the metal strips due to shrinkage, the support of the metal for loads traversing the joint seems effective in limiting damage to the adjacent concrete edge. Much concern has been expressed to the author about the use of aluminium in contact with wet concrete. Significant foaming only occurs when the aluminium is finely divided and the only deleterious effect observed in the use of concrete in conjunction with extruded or cast aluminium sections is discolouration, which is of no importance in this case.

An obvious development of the expansion joint detail would be the use of a light steel tee in a contraction joint. Unfortunately such sections are not available in Rhodesia.

Contraction Joints

Contraction joints are no longer provided in addition to construction joints. Experience with sawn joints has been adverse in that the normal filler is too resilient to give adequate support to the sawn edges under heavy traffic, and deterioration sets in rapidly. If a sufficiently stiff filler is used it tends to break away on one side of the joint leaving it unsupported. In either case a concavity develops as the concrete shrinks, leading to breaking down of the corners. Other disadvantages are higher relative shrinkage than occurs when pouring approximately square panels of about 20 m^2 in a chequerboard pattern, impossibility of visual checks on compaction, and less flexibility in the replacement of isolated panels of floor. In addition their cost has been found to be as much as double that of formed joints, although it is believed that this comparison was unrepresentative. One positive advantage of sawn joints is that they provide the only certain check on the final position of top reinforcement, if the appearance of sparks is noted. A cover-meter has been found to be useless, probably because of the abrasion of steel finishing tools leaving traces of iron in the concrete.

It is preferable to avoid staggering of joints, due to the tendency of the panel corners to rise slightly with shrinkage. However, when staggering has been unavoidable (as in the case of the long tracks subsequently described in case history 2), the provision of the shear key has been effective in eliminating differential movement without leading to surface cracking due to tension in the concrete.

Other Joints

This term includes interfaces between concrete and other floors such as steel duct covers. A popular detail for the latter is illustrated in Figure 4(a), together with the observed consequences. Wheels traversing the gap are caused to vibrate leading to abnormal wear. This is aggravated by the tendency of the grating panel to ride up on the root radius of the angle. The detail illustrated in Figure 4(b) has proved satisfactory. A detail for chequer-plate floor support is illustrated in Figure 5.

Strength and Durability

Sub-floor Preparation

No structure is better than its foundation and adequate sub-floor preparation is vital when heavy loads are expected. However, light loads can be highly abrasive, although they may not require a good foundation. It is only necessary to ensure that uneven settlement does not occur under the expected loads. Case history 3 graphically illustrates the consequences of sub-grade failure. Provision of an impervious membrane under the slab, normally polythene sheeting 0.5 mm thick, is regarded as essential since it is only possible to ensure close control of water content in the concrete if water loss into the sub-grade is prevented. Provision of such a membrane is by no means standard practice in a dry climate.

Materials

The normal contractual provisions are observed only perhaps a little more strictly than usual. The tendency of the quarry-master to let standards of grading and cleanliness slip once the order has been secured is checked by rejection of deliveries not equal to the sample held on site. This seldom presents a problem once a stand has been made. A coarse-graded nominal 20 mm aggregate is generally favoured.

Mixing

Like most optima, the optimum design mix proportions can be varied fairly widely on either side without a great effect on workability or strength. For durable floors the end of this range with the higher aggregate content is aimed at. Where time does not permit, or quantities do not warrant a designed mix, the following trial mix is used:

Cement	50 kg
Sand	130 kg
Crushed-rock Aggregate (20 mm nominal)	220 kg

If this is found to be workable, sand content is decreased and aggregate content increased in 5 kg or 1/300 m^3 steps until workability loss is marked, so as to select a mix for the contract. Workability is not sacrificed in favour of

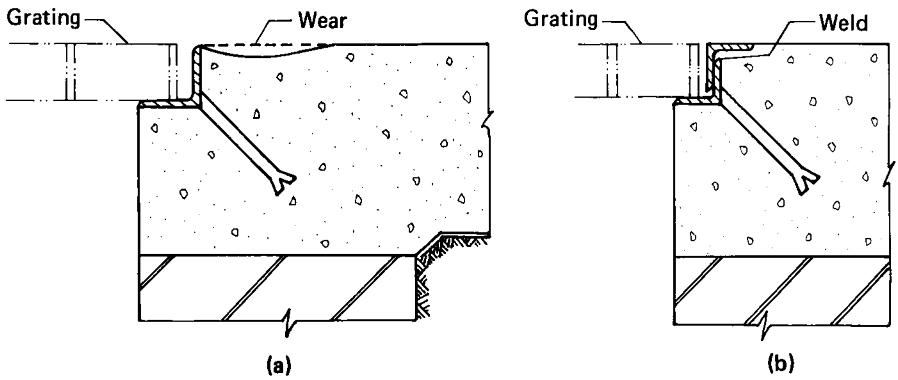

Figure 4 Sections of Steel Grating Floor Support

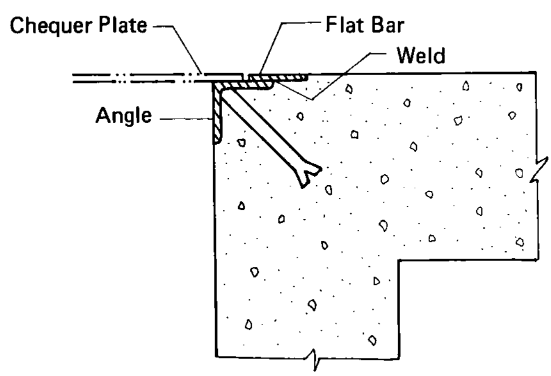

Figure 5 Section of Steel Plate Floor Support

aggregate content. Water content is limited to give not more than 10 mm slump with the standard slump test. Experience is that the foreman's initial reaction is that this is the usual crackpot idea of an impractical engineer who has a bee in his bonnet about wet concrete. It is only necessary to keep the foreman up with the finishing process for one whole night to bring about a situation where the only problem with water is getting enough into the concrete for proper compaction. After brief experience, most foreman achieve workable zero-slump concrete. Negative slumps, arising from friction with the mould, have been recorded on occasions.

Concrete is required to be of minimum Grade 40 strength. Characteristic strength is specified as mean cube strength less 1.28 standard deviations (10 per cent failure rate), instead of the 1.645 standard deviations implied by C.P. 110 (4) but, due to the close control, this difference is not highly significant. Actual grades achieved are typically 50 to 55 or more. This is attributed in part to the foreman's fear of having concrete rejected as a result of being required to produce high-strength concrete from a lean mix. Consequently he tends to give the manufacture of concrete the careful attention that it seldom otherwise receives.

Mixing is normally carried out in a rotating drum mixer and time of mixing is increased if necessary to ensure proper mixing. More than three minutes has never been found necessary, and this has never been a controlling factor in rate of placing.

Placing and Finishing

This is specified as a two-shift operation, the first one placing and compacting, the second finishing. Proper compaction has been the biggest problem, possibly due to the country's isolation making suitable equipment unavailable. The most successful compactor used was purpose-made and is illustrated diagrammatically in Figure 6.

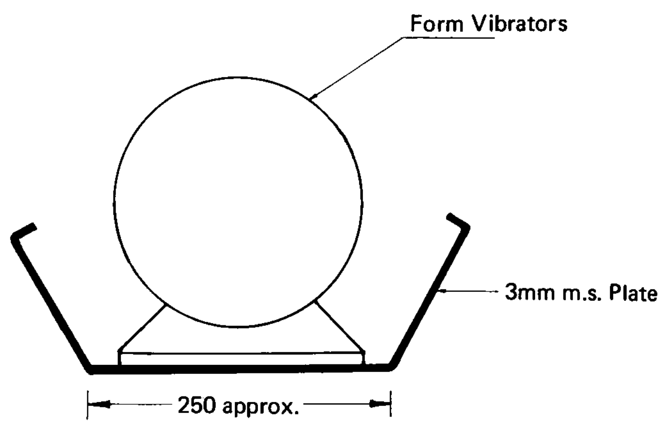

Figure 6 Cross-Section of a Vibrating Screed

It is necessary for the screed to be somewhat wider than the thickness of the slab to ensure that surcharge is driven downwards and not merely squeezed out sideways. The screed illustrated, being about 5 m long and carrying two form vibrators, is heavy, requiring four men to lift it. Attention is given to ensuring that it is moved by lifting, moving laterally about two-thirds of its width and lowering, and not by sliding, pushing the surcharge before it.

Forms are overfilled by about 20-25 per cent of slab thickness. This makes it extremely difficult to keep top reinforcement in place. After vibration and screeding to level the surface is 'closed' with a power float (rotating disc). With such a dry mix there is a tendency to have small voids on the surface which the float closes. In corners, adjacent to walls and such awkward positions, it is sometimes necessary to close by hand. Use of the float is held to the minimum necessary to provide a smooth, level surface, so as to avoid bringing laitence to the surface.

The concrete is then allowed to stand. No matter how dry the mix, bleed water appears, and it is left to disappear of its own accord, time being highly variable from about three to eight or more hours, depending on shelter, temperature, and humidity. De-watering mats have not been used, due to difficulty of importation.

After disappearance of bleed water, the surface is trowelled with a power trowel (rotating blades) commencing with the blades flat, and raising them up one notch at a time as hardening proceeds until no impression is made on the concrete at their steepest setting. Wear on blades is severe and it is essential to keep a supply of spares on site since development of a feathered edge is disastrous.

The final appearance of the slab is nearly black, slightly polished, and preferebly slightly mottled, revealing that the aggregate has been successfully kept near the surface. Voids can still occur even when great care is exercised. It has been found worthwhile to patch these with epoxy resin filled with sand, and a little cement powder for the purpose of colouring. Cement based patches tend to break out.

Shrinkage

Measures already described limit shrinkage. It is further limited by wet curing, whenever possible by ponding. However, this leads to problems with chequerboard construction and even more so when the floor is laid to a slight fall as it commonly is, so covering with wet sand, which is kept wet for at least seven days, is usually preferred. Reinforcement is normally provided to cater for shrinkage as well as structural loads, in two layers, top and bottom totalling about 0.25 per cent of the slab cross-sectional area each way, unless heavier reinforcement is indicated by loading conditions.

The author's experience is that the support systems favoured by contractors are totally inadequate and the only way to ensure that the top reinforcement is in the correct position is to place it there after pouring concrete and when compaction is almost complete. Even then and with the greatest care exercised, subsequent investigation has shown that a nominal 20 mm top cover can be anything from zero to 50 mm in a 150 mm thick slab; without care it can be equal to the slab thickness.

Planning

When a floor in a working area must be replaced processes can be rearranged and standby procedures mobilized to overcome problems. Replacement of passage floors on the other hand can become almost impossible. The author favours provision of double-width passages with a central construction joint to permit non-disruptive replacement.

CASE HISTORIES

1. A warehouse for a builder's and general hardware merchant. Previous floors completely disintegrated under a combination of static loads of up to 50 kN/m^2 and impact and abrasion from metal goods and containers. On taking occupation the owners complained that they were unable to shot-fix their racks to the floor, the heaviest charge resulting only in a bent nail and a small chip in the floor. Otherwise they are completely satisfied.

2. Numerous floors for an international manufacturer. The material handling includes the use of heavily loaded steel-shod stillages which are dragged as often as they are lifted, and trolleys with small diameter unsprung steel wheels.

Disintegration of floors has been a persistent problem in all their factories world-wide. Numerous techniques have been tried without finding one that can be regarded as a standard to be adopted for future use. In parts of the factory serious deterioration occurs within 12 months with normal concrete.

In certain areas of this factory in the past, certain loads running on metal wheels have worn grooves into the floors. This has been successfully countered by casting metal tracks in the concrete. These are tees cut from universal beams, welded to spacers to maintain the gauge during casting in. Guides comprising flat bars are bolted to holes tapped in the flanges. The guides are removed during concreting so that concrete can be finished flush with the face of the flange, as shown in Figure 7. The presence of these tracks has necessitated pouring panels nearly 30 m long without contraction joints. No adverse effects have been observed, and shrinkage seems to have been proportionately much less than in nearby panels with about 5 m between joints. This is possibly due to the restraining effect of the well anchored heavy steel sections, which has nevertheless not been sufficient to lead to visible shrinkage cracks.

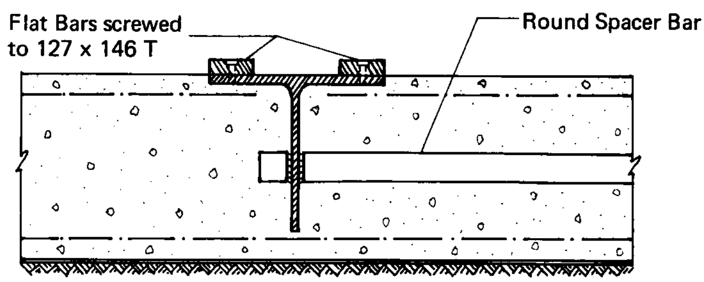

Figure 7 Section of a Floor Track

Deterioration commencing at joints continues to be a problem although a greatly reduced one, and until a more cost effective floor is discovered it is intended to continue to use the specification described.

In July 1975 it was necessary to reconstruct the floor of a passage subject to heavy traffic. Advantage was taken of this to construct half the length to the specification described above, and the other half to the specification of, and under the supervision of, the technical representative of an internationally known manufacturer of a floor-hardening compound incorporating iron particles, which had the effect of doubling the cost from approximately R$4.50 (then about £4) to R$9.00 per m^2. This passage has been observed annually since then and the only serious deterioration has been at the joint between the two types of construction which was improperly formed as a V-joint. At the age of three years it is possible to discern a slightly faster rate of wear in the plain concrete floor which has now lost its polish, but it is still a sound, smooth floor which will undoubtedly give many years of satisfactory service, and it appears most improbable that the hardened floor will outlast it by a factor of two.

It should be noted that the limited working space in the passage inhibited proper compaction and limited the use of machine tools. Also the expectation is that under heavy traffic the skin of the plain concrete floor will be lost but that the aggregate thus exposed will wear well thereafter.

3. A heavy industrial floor. For this, the operation was almost exactly analogous to that of the cartoon convict breaking boulders into road stone with a sledge-hammer. The material is, however, harder and more abrasive than rock. To prevent contamination it was necessary that the operation should take place on a floor. Understandably concrete floors had a very short life, of the order of 12 to 18 months between complete reconstruction. The author was asked to propose improvements, but unfortunately with such constraints that acceptable advice was effectively limited to a mixing and placing specification. Among these constraints was a ban on the use of reinforcing steel. Previous experience was that reinforcement was soon exposed, whereafter it broke and curled upwards, inflicting damage on the rubber tyres of vehicles travelling on the floor far exceeding the cost of replacing the floor.

The owners also dictated that construction should be a 120 mm thick high grade slab, to the specification described hereon, on black polythene sheet on a 120 mm thick Grade 20 unreinforced base slab. The intention was that the black polythene should act as an indicator that reconstruction of the upper slab was due. The site selected was a level area which had been subject to very heavy works traffic for many years and was impregnated with slag and tramp metal rendering compaction testing unrealistic. It was deemed better to leave the sub-grade undisturbed, beyond cutting to a uniform level. Unfortunately there was in the middle of the site a disused underground machine chamber and it was necessary to run a stormwater pipe across the site. Both the chamber and the trench were filled with waste material from the plant, which was not amenable to compaction testing but which was known from experience to pack down admirably and form a stable fill subject to far less settlement than conventional filling materials. Great care was taken with filling. Nevertheless, after about a year disintegration commenced over the filled areas and, due to a lack of co-operation between production and maintenance departments, was allowed to spread, resulting in no increase in the life of the floor. Clearly the fill should have been stabilised.

This example is cited to illustrate the importance of sub-grade preparation in extremely adverse conditions. It was nevertheless noticeable that the concrete only succumbed to 'breaking back from an edge', and that where constructed on the undisturbed sub-grade it successfully performed the function of an anvil, even though unreinforced, until reached by the edge. The production department ignored the appearance of the black polythene, merely complaining about the contamination it caused in the process, and went on to demolish the lower slab, which should be some kind of a lesson to the construction department.

APPLIED FINISH

The foregoing examples all deal with integral finishes. There are cases where an applied finish is necessary. The author has been concerned with several suspended floors which are subject to an unsprung moving point load of about 90 kN. The floors are also subject to a temperature gradient from about 400°C to ambient in about 10 m. Experience has shown the necessity for such floors to be steel clad and the favoured cladding is steel tiles, similar to those popular in bake-house floors, which are necessarily set in a screed applied after the construction of the structural floor. Experience has also shown that this screed is the component least likely to last four years between maintenance shut-downs, the tiles in the hot area lifting with breaking up of the screed, which spreads rapidly under traffic.

The author was consulted about this problem and proposed a procedure which has proved successful.

The screed mix was made leaner by the introduction of fine crushed aggregate of

10 mm maximum size, (for 40 mm screed thickness). The trial mix is 50 kg of cement, 100 kg of fine aggregate, and 75 kg of coarse aggregate, mixed with the minimum amount of water to ensure workability. A new structural slab is prepared by brushing with a stiff bristle broom after disappearance of bleed water. An existing one is well hacked to expose fresh concrete.

A PVA-based bonding agent is then applied in accordance with the manufacturer's instructions. The entire surface is then slurried with a slurry of the proportions of the screed, without the coarse aggregate, and after a short setting period the screed is laid and well tamped. The bonding agent is incorporated in the mixing water for both slurry and screed. The screed is reinforced with a single layer of welded steel wire mesh, especially ordered with wires at 100 mm centres each way; 5.6 mm diameter wire is used although the close spacing is regarded as being of more significance than the mass. The steel floor tiles are thoroughly degreased and immediately before laying their undersides are painted with a pure cement wash. Care is taken to avoid entrapping air or disturbing freshly laid tiles. Due to the protection afforded by the tiles curing can start in as little as 6 hours after laying.

Performance of floors laid to this specification has been completely satisfactory. As if conditions were not demanding enough, floors have been relaid in this manner and brought into use in less than three days after laying. In this case particular attention was paid to curing, the floor being inundated as soon as workmen were clear and being left thus until the water boiled off. The only adverse effect observed occurred when a salesman persuaded a representative of the owners that such high speed commissioning required the use of quick-setting cement, and came away with an order for high alumina cement. Without reference to the author, the owners' representative instructed the contractors to use this cement. Fortunately the salesman was a worse estimator than the representative and only supplied about one third of the requisite quantity. The remainder of the floor was constructed with normal Portland cement, there being insufficient time to obtain further supplies of high alumina cement. The author was duly criticized when one third of the floor disintegrated within a fortnight. This not unexpected result, considering the high temperature wet curing, may be something of a record. The remainder of the floor, however, successfully lasted to the next maintenance shut down.

CONCLUSIONS

In Rhodesia, and similar environments, attention paid to the improvement of the durability of conventional concrete is more cost effective than the use of sophisticated admixtures and surface treatments.

Concrete, like babies, responds to tender, loving care, and its quality can be greatly improved by devoting careful attention to all stages of its manufacture. Nothing can turn bad concrete into good concrete, and good concrete needs little assistance in doing its job.

The story of the horseshoe nail is nowhere more applicable than in the construction of large industrial floors where a small detail can frustrate a grand design.

Stabilization of under-floor fill is worthwhile when heavy loads are expected.

Do not believe that top reinforcement is actually in the top of the slab because the drawings say so. Unless vigorous steps are taken to keep it there, it is more likely to end up in the bottom.

When different organizations are responsible for construction and operation it is essential to ensure co-operation. The author has found it worthwhile to present owners with an 'Owners' Manual' giving instructions for the operation and maintenance of their new building.

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A REVIEW OF THE DEVELOPMENT OF THE CURRENT INTERNATIONAL PROCEDURES FOR THE DESIGN AND CONSTRUCTION OF PRESTRESSED SLABS IN BUILDINGS

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ABSTRACT The inclusion of prestressed flat slabs in buildings is a method of construction which has been used extensively in the U.S.A. and Australasia for over two decades. However, it has never achieved a real breakthrough in Europe, particularly in the U.K. The development of design methods and construction techniques are reviewed from the initiation of the subject up to the recent emergence of the new F.I.P. recommendations. A comparison of the current international design practices is made and some reasons suggested for the apparent lack of demand for this form of construction in the U.K. The paper concludes with a brief description of the prestressing equipment currently available together with some comments on detailing.

INTRODUCTION

Development of the Use of Prestressed Slabs

The idea for using post-tensioned, cast in-situ, suspended slabs in building construction was conceived in the U.S.A. in the middle 1950's as an extension of the lift slab technique for buildings of the flat slab type. The method developed and the 1960's saw an increased use of prestressed slabs not only in the U.S.A. but also in Australasia. By 1970 the vast majority of prestressed slabs in the U.S.A. were cast in-situ using post-tensioned unbonded tendons but in Australia and New Zealand bonded tendons were preferred. At this time the introduction of lightweight aggregates led to an increasing use of lightweight concrete in prestressed slabs.

The 1970's saw an extension of the use of this technique into the Far East and South America and in the U.S.A. construction is now proceeding at the rate of some 3 million square metres per annum. In contrast, a major breakthrough has not yet occurred in Europe, although prestressed slabs are being used increasingly in some countries such as Holland and Switzerland. In the U.K. there is no real market for this form of construction at present, with only a very few projects having been completed to date.

Advantages of Post-Tensioned Slabs

Prestressed flat slabs have generally been found to be economical for spans between

5.5 and 10.5 m; for spans in excess of this, drop panels around the column are usually required and over 14 to 15 m spans a beam and girder system or ribbed (waffle) slab is found to be more suitable. The major advantage of prestressed compared to conventionally reinforced slabs is the reduced deflection for a similar loading and slab thickness. In fact the major part of the permanent load can be balanced by the vertical component of the prestress force, thereby resulting in little or no deflection under working loads. In addition the compression force exerted on the concrete by the prestress reduces the chances of cracking in the slab itself. These two factors enable longer spans to be utilised, and hence provide large, functional, column-free areas, a primary requirement in modern buildings, and the introduction of lightweight concrete has enabled the use of even thinner slabs.

Prestressed slabs also have many other merits which arise due to the advantages discussed above. The smaller deflections minimise the possibility of cracking in partitions above and below each floor. The resulting thinner slabs can increase available headroom or decrease overall building height thereby reducing dead load and the size of foundations required. It has also been widely claimed that the method is economical in terms of construction time, materials, labour and hence capital cost. It is a flexible form of construction which can be utilised in a wide variety of structures.

On the debit side, the major problem which has to be overcome is the axial shortening due to elastic deformation and shrinkage and its subsequent effect on supporting elements. This means that there is a limit on the length of slab which can be stressed at one time resulting in the need for construction joints and careful detailing of slab to column joints. In addition there is a limit to thinness of slab which can be achieved in practice due to the tendency of the floor to become springy and disconcerting to walk on.

LITERATURE SURVEY

A comprehensive review of the early developments in prestressed flat plate technology in the 1950's and 1960's is given by Nasser (1), discussing the early papers (2, 3, 4, 5, 6) on the topic from its initiation in buildings constructed by the lift slab technique to the summary by Green (7) in 1962. The paper looks at the practice of designing a two-way spanning prestressed slab by assuming that the load is supported by systems of equivalent beams at right angles. The panels are split into column and middle strips with a distribution of prestress in the ratio 60 to 40 respectively. It was soon recognised that this equivalent beam (or frame) method gave acceptable results when compared with the rigorous approach using elastic plate theory.

The load balancing method of determining the required prestress force introduced by Lin (8) in 1963 was soon seen to be a powerful tool for the design of continuous prestressed slabs. In this method the dead load and part of the live load is balanced by the upward component of the prestress force. It therefore provides a control on deflection under dead load and simplifies the analysis of a highly redundant system. The unbalanced portion of the load is then carried by the slab acting under a uniform horizontal prestress. Further American papers (9, 10, 11) extended the theory of load balancing and looked at optimisation of design. Meanwhile, in Australasia, the method of prestressing flat slabs in buildings was also gaining favour and several papers in the mid 1960's (12, 13, 14, 15) described design techniques based on similar principles to those used in the U.S.A. The importance of the shear strength of the slab to column connection was recognised and in this context the need to concentrate tendons over the columns was emphasized.

Developments and laboratory testing continued in both parts of the world (16, 17, 18, 19) particularly with the introduction of slabs containing lightweight aggregates to reduce dead loads and improve fire resistance properties. The emphasis in the U.S.A. had been on unbonded tendons using non-prestressed bonded reinforcement to control cracking at the ultimate limit state. It was maintained (20, 21, 22) that this form of construction controlled cracking and increased the overall ductility of the structure thereby preventing sudden collapse, a particularly important criterion in the event of earthquake loading.

The year 1974 saw the emergence, for the first time, of formal recommendations for the design of prestressed concrete flat slabs. The American Concrete Institute and the American Society of Civil Engineers jointly published their tentative recommendations (23) and in the U.K. the Concrete Society published their recommendations (24). These documents form the basis of the comparison made later in this paper, together with the Australian Code for prestressed concrete (25), published in the same year, which now included specific provisions relating to post-tensioned flat slabs.

One of the first reports on the use of prestressed flat slabs in the U.K. was by Held (26) in 1975 in which he discussed two projects which had utilised this form of construction and had been designed in accordance with the Concrete Society recommendations. A cost study revealed a 6 per cent saving per m^2 for the prestressed slab over traditional reinforced concrete slab construction in one project and a 22 per cent saving over a waffle slab in the other. It is reported that these figures do not take into account any cost savings due to reduced building height or lower dead loads on the foundations.

In 1976 a symposium on the use of prestressed concrete in buildings was held in Sydney at which Matt and Thorpe (27) compared existing recommendations with current design practice, showing that the variety of existing recommendations led to differences in the completed product. At the same symposium Lin (28) presented the American case for the use of unbonded tendons with supplementary non-prestressed bonded reinforcement to control cracking and assist towards the punching shear resistance at ultimate conditions. Arguments put forward in favour of this approach, besides the economic case, were protection of the steel during construction, avoidance of grouting operations and low friction characteristics. A summary of the opposing Australian point of view for bonded tendons was presented by Pash (29), the main arguments put forward in favour of bonded tendons being that any corrosion or accidental fracture of a tendon only affects a localised area and not the complete span which could result in serious damage to the structure; the force is transferred to the concrete over a length of tendon and not solely at the anchorage areas; there are increased ultimate flexural or shear strengths with less or no reliance on non-prestressed reinforcement; there are fewer problems when eventual demolition is required.

In the U.K., Turner (30) has put forward some suggestions as to why this form of construction had not been favoured. These criticisms were generally aimed at the low average prestress and admittedly conservative shear design approach recommended by the Concrete Society. Recently Clark (31) has formalised the approach to shear design, based on C.P. 110 (32), and compared it with the American practice and a method based on C.E.B. Bulletin 117 (33), the implications of which will be discussed later. The method of calculating the allowable shear stresses, using the equivalent ratio of untensioned steel, is to be adopted in the new Concrete Society recommendations (34) which will, when published, be in accordance with C.P. 110. A design and cost study has recently been completed by Dowrick (35) in which costs of a given five storey building constructed with flat slab floors of reinforced or prestressed concrete have been compared. The designs were based on C.P. 110 and the new Concrete Society recommendations using the same span configuration in all cases. It was found that the prestressed coffered slab building was 2.7 per cent cheaper than a prestressed solid slab and 2 per cent cheaper than a reinforced coffered slab. For the structure alone the prestressed slab was 12 per cent cheaper than the reinforced slab indicating that the prestressed concrete alternative may be particularly competitive in those structures where the structural cost is relatively high, such as multi-storey car parks.

On the International scene an I.A.B.S.E. survey by Swiss engineers (36) looked at several aspects of prestressed flat slab design, recommending that a clear distinction must be drawn between suitable methods of analysis for bonded and unbonded systems. Ordinary elastic or plastic theory for thin plates with small deflections is proposed for a bonded system but in the unbonded case the in-plane forces need to be taken into account by using a flat tied arch model. Several of their suggestions on tendon layouts and detailing will be discussed in the last section of this paper. These conclusions have formed a significant contribution to the new F.I.P. recommendations (37) which have been prepared in order to achieve a set of guidelines which would be internationally acceptable and to lessen the gaps between the different approaches throughout the world.

INTERNATIONAL RECOMMENDATIONS ON PRESTRESSED FLAT SLAB DESIGN

A Comparison

The aim of this comparison is to observe the differences between the current U.K. recommendations and those pertaining elsewhere in an attempt to isolate those specific aspects where modifications in line with international practice may lead to more uniformity in design. The main comparison is with the A.C.I. recommendations since these have been used successfully on a large scale, but reference will be made to other international opinion where it is particularly relevant to the aspect of design or construction under consideration.

Discussion

In any comparison of this nature it must always be borne in mind that differing Codes of Practice will be based on varying assumptions and therefore a strict comparison is not always feasible. However, for those design recommendations considered here the basic limit state philosophy holds and although there are slight differences in the assumptions made, a comparison will certainly indicate the trend of thought. It should also be mentioned at this stage that the existing Concrete Society recommendations (C.S.R.) are based on a now outdated Code of Practice, C.P. 115 (38), except for shear design which is based on the new limit state code, C.P. 110. C.P. 115, although not recognised at the time as being based on limit state design philosophy, does require checks to be made under both serviceability and ultimate limit states. An important point in this context is that since 1976 it has been officially unacceptable to mix standards in any one design and this is precisely what is done in the C.S.R. It is important, therefore, that the updated C.S.R., which will be based entirely on C.P. 110, are available to practising engineers as soon as possible.

Methods of structural analysis (see Table 1). The A.C.I. - A.S.C.E. recommendations (A.R.) and the C.S.R. are essentially similar in allowing either rigorous methods or the equivalent beam approach to be used. The later recommendations from I.A.B.S.E. and F.I.P. stress the need for separate approaches for unbonded or bonded systems and suggest that the presently used methods of analysis in the unbonded case are unsatisfactory. It would be surprising, however, if designers were to immediately forsake the equivalent beam approach with some form of load balancing, a method which has been used successfully for many years.

Average prestress after losses (see Table 1). Both the A.R. and the Australian Code allow a much higher average prestress than is permitted by the C.S.R. Turner (30) suggests that the U.K. approach is extremely conservative and that a mean stress of 4 N/mm^2 would be more realistic without creating excessive problems due to creep and deflection. Certainly a higher average prestress would enable more of the allowable compressive stress to be utilised and improve the shear resistance. In the light of American experience it would seem sensible to permit higher prestress levels than are used at present in the U.K., if not to Turner's suggested value then at least to the 3.4 N/mm^2 specified as a desirable limit in the A.R.

Permissible flexural stresses in concrete (see Table 2). The C.S.R. make no distinction according to the method of analysis used whereas the A.R. and F.I.P. allow higher permissible stresses in column areas if a rigorous analysis is employed. Otherwise it can be seen from the Table that the stresses follow the same trends.

Shear (see Table 3). This is the area where perhaps the greatest differences in approach lie. The A.R. consider the shear resistance (a) due to the slab acting as a wide prestressed beam and (b) for punching on a perimeter at half the effective depth from the face of the column. It is recognised that the second check normally controls the design and this is based on limiting the ultimate shear stress to that which causes diagonal web cracking (v_{cw}). For all intents and purposes v_{cw} is simply related to the concrete strength and the mean prestress.

The C.S.R. considers the shear resistance to be limited by (a) the principal tensile stress on the centroidal axis on a perimeter at 0.75 times the overall slab depth from the column face and (b) the ultimate shear stress for a reinforced concrete slab on a perimeter at 1.5 times the overall depth from the column face based on an equivalent area of untensioned steel. The working party recognise that this latter approach may be conservative and Turner (30) emphasises this point strongly, claiming that the resultant slab thickness will be no different from an ordinary reinforced slab.

It can be shown that the A.R. check (b) would yield similar results to the C.S.R. check (a) if similar shear perimeters were utilised. However, C.S.R. check (b) is usually critical and Clark (31) has carried out several trial calculations to compare the two approaches together with a method based on C.E.B. Bulletin 117. He showed that the U.K. approach, based on C.P. 110, and the A.C.R. give very similar punching strengths for all thicknesses of a slab on a 200 mm square column but that the values obtained using C.E.B. 117 were approximately 20 per cent smaller, see Figure 1. As the column size is increased the U.K. approach yields more conservative punching strengths because of the differing critical perimeters used, see Figure 2. If tendons are concentrated over columns then the U.K. approach and C.E.B. 117 yield higher strengths than if they are uniformly distributed since the stress is proportional to the equivalent steel ratio. This does not happen in the A.R. since the strength is related to the average prestress which is independent of the tendon distribution, see Figure 3.

The I.A.B.S.E. recommend a cautious shear design approach unless ductile behaviour associated with shear strength can be guaranteed.

Table 1 Methods of Structural Analysis and Average Prestress after Losses

SUBJECT	A.C.I. - A.S.C.E.	CONCRETE SOCIETY	OTHER
Methods of Structural Analysis	Either: Equivalent frame (or beam). Moments due to prestressing using load balancing. Effects of reverse tendon curvature generally neglected for flexure but must be considered when evaluating shear carried by tendons over columns. Or: Rigorous using (a) Elastic theory of bending for thin plates (b) Finite element analysis (c) Finite difference analysis	Either: Equivalent frame (or beam). Moments due to prestressing using load balancing. Effects of reverse tendon curvature must be allowed for. Or: Methods satisfying principles of statics and continuity.	I.A.B.S.E. & F.I.P. Bonded system: Elastic or plastic theory for plates with small deflections. Unbonded system: Flat tied arch model. In-plane forces explicitly taken into account. Tendon force increase is primarily a geometrical problem.
Average prestress after losses	Minimum: 1.4 - 1.7 N/mm^2 (absolute min. 0.9 N/mm^2) to minimise cracking. Maximum: 3.4 N/mm^2 to avoid excessive shortening.	Keep low: 1.5 - 2.0 N/mm^2 to avoid creep, long-term deflection and springiness.	Australian Code Minimum: 1.4 N/mm^2 Maximum: 4.1 N/mm^2 but greater if slab can withstand axial deformation.

Table 2 Permissible Flexural Stresses in Concrete

A.C.I. - A.S.C.E.	CONCRETE SOCIETY	OTHER
For equivalent frame analysis with bonded or unbonded tendons:-	All methods of analysis:	F.I.P. As A.C.I. - A.S.C.E.
Compressive:	Compressive:	
Negative moment areas (columns)	Transfer: $0.4\ f_{ci}$	
$0.30\ f'_c$ ($\approx 0.24\ f_{cu}$)	(triangular prestress distribution)	
Positive moment areas	$0.5\ f_{ci}$	
Transfer: $0.60\ f'_{ci}$ ($\approx 0.48\ f_{ci}$)	(uniform prestress distribution)	
Service: $0.45\ f'_c$ ($\approx 0.36\ f_{cu}$)	Service: $0.33\ f_{cu}$	
Tensile:	Tensile:	
Negative moment areas (columns)	Transfer: Unbonded tendons	
No u.p. reinf.: 0	No u.p. reinf.: 0	
With u.p. reinf.: $6\sqrt{f'_c}$ ($\approx 0.50\sqrt{f_{cu}}$)	With u.p. reinf.: $0.48\sqrt{f_{ci}}$	
Positive moment areas	Bonded tendons	
No u.p. reinf.: $2\sqrt{f'_c}$ ($\approx 0.17\sqrt{f_{cu}}$	No u.p. reinf.: $0.24\sqrt{f_{ci}}$	
With u.p. reinf.: $6\sqrt{f'_c}$ ($\approx 0.50\sqrt{f_{cu}}$)	With u.p. reinf.: $0.48\sqrt{f_{ci}}$	
Rigorous analysis with u.p. reinf. Higher allowable stresses permitted for peak service load moments at columns	Service: Unbonded tendons	
	No. u.p. reinf.: $0.24\sqrt{f_{cu}}$	
	With u.p. reinf.: $0.48\sqrt{f_{cu}}$	
	Bonded tendons	
	All cases $0.48\sqrt{f_{cu}}$	
Specified cylinder strength (p.s.i.) f'_c - 28 days f'_{ci} - transfer	Characteristic cube strength (N/mm^2) f_{cu} - 28 days f_{ci} - transfer	

Table 3 Shear Design Approach

A.C.I. - A.S.C.E.	CONCRETE SOCIETY	OTHER
Most severe of: (a) Shear design as a wide beam (as per A.C.I. 318-71 Cl 11.5 for prestressed members) (b) Shear design for 2 way action (usually controls shear design). Critical section: At half effective depth from face of column. Ultimate shear stress, v_u, not to exceed v_{cw}, the shear stress to cause diagonal web cracking. $v_u = \frac{V_u}{\phi b_o d}$, $\phi = 0.85$ $v_{cw} = 3.5\sqrt{f'_c} + 0.3f_{pc} + \frac{V_p}{b_w d}$ ($f_{pc} \ngtr 500$ p.s.i.; $f'_c \ngtr 5000$ p.s.i.) If shear reinf. is provided v_u may exceed v_{cw} by 50%. If shearhead reinf. is provided v_u may exceed v_{cw} by 75%.	Most severe of: (a) Limiting principal tensile stress, f_t, at centroidal axis, on shear perimeter 0.75 h from column face, to 0.24 f_{cu} (for prestressed members). (b) Limiting shear stress, v, on shear perimeter 1.5h from column face to v_c $v = \frac{V}{u\ crit.d}$ $v_c = (\rho^* f_{cu})^{1/3}$ (reinf. members) $\rho^* = (\frac{A_{ps} f_{pu}}{410} + As)/bd \ngtr 3.0$ (Usually controls shear design but recognises this may be conservative). v_c may be increased by ξ_s which varies from 1.3 for $h \leqslant 150$ mm to 1.0 for $h \geqslant 300$ mm. If $h > 200$ mm shear reinf. may be provided but $v \ngtr 0.75\sqrt{f_{cu}}$	C.E.B. No specific proposals for prestressed slabs but if area of tendons converted to equiv. area of u.p. reinf. $V_{rd1} = 0.5f_{ctd}k(1+50\rho_1)u\ crit.d$ k - slab depth factor f_{ctd} (design tensile strength)$=0.14(f_{ck})^{2/3}$ u crit = shear perimeter at half effective depth from face of column. I.A.B.S.E. Part of load transferred to column by vert. comp. of prestress force, the remainder as in r.c. slabs. Tendons concentrated over columns act as suspension net. Always strengthen with u.p. reinf for ductility and crack control.

f'_c - characteristic cylinder strength (p.s.i.)
f_{pc} - average stress due to prestress (p.s.i.)
b_o - critical perimeter
f_{cu} - characteristic cube strength (N/mm^2)
f_{ck} - characteristic cylinder strength (N/mm^2)

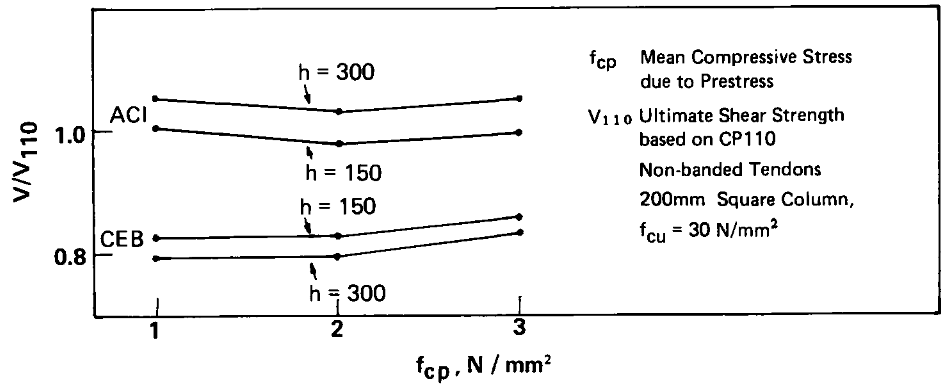

Figure 1 Punching Shear Strengths (due to Clark)

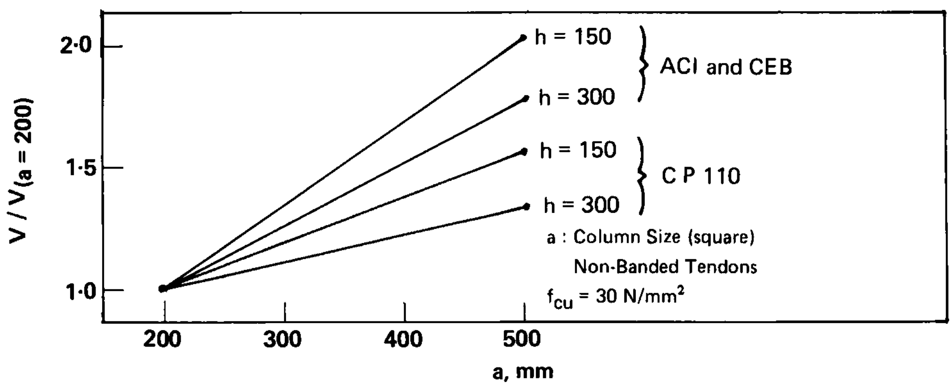

Figure 2 Effect of Column Size on Punching Strength (due to Clark)

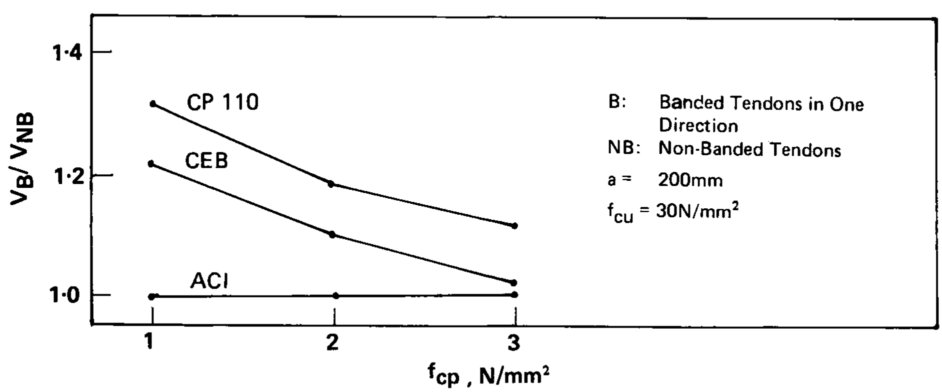

Figure 3 Effect of Banding Tendons (due to Clark)

Ultimate strength (see Table 4). All recommendations essentially provide simplified data for determining the ultimate strength of bonded and unbonded slab systems in lieu of a rigorous strain compatibility approach. In each case allowance is made for the tendon stress increase in unbonded tendons. The I.A.B.S.E. survey has pointed out that this increase is related directly to the total change of tendon length due to deflection and movements at each end. In an unrestrined slab this will be solely dependent on the span-depth ratio but if restrained it is also dependent on the initial tendon shape. Figure 4 shows how the present A.R. and C.S.R. take account of this stress increase, both of which are independent of the span-depth ratio. The F.I.P. have adopted the method of C.P. 110 (also shown in Figure 4) which does take account of the span-depth ratio.

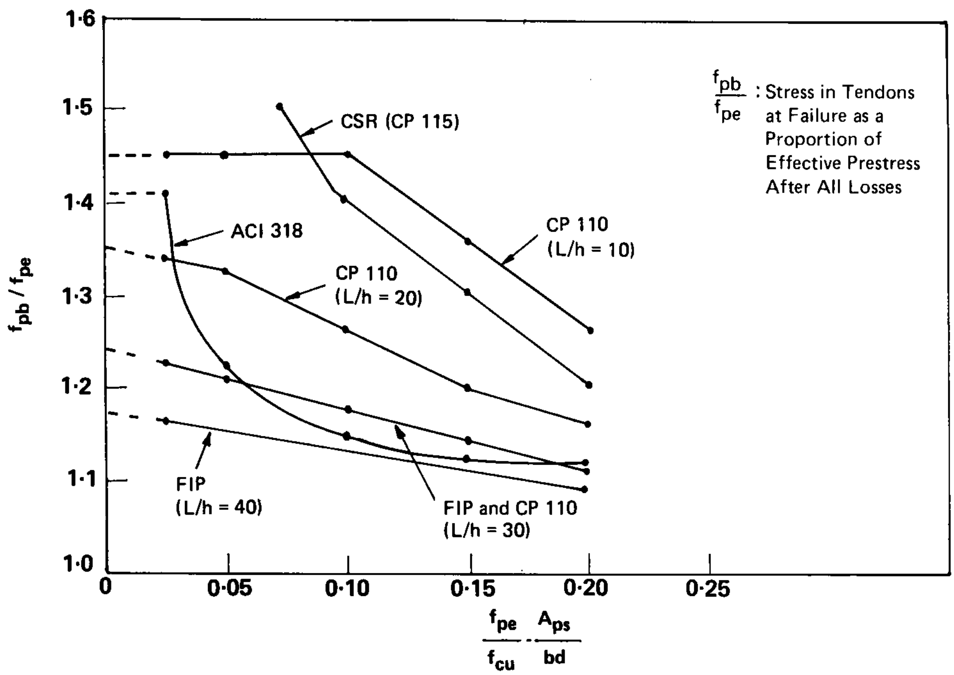

Figure 4 Tendon Stress Increase in Unbonded Tendons

Deflection (see Table 4). Similar limits in all cases.

Tendon spacing (see Table 4). The C.S.R. allow a larger spacing for unbonded tendons whereas the A.R. do not differentiate presumably because the use of unbonded tendons in the context is rare.

Tendon distribution (see Table 4). The C.S.R. and the A.R. both use the traditional concept of column and middle strips with similar proportions of tendons to be placed in the column strips when the slab length-width ratio is less than 1.33. The C.S.R. give additional guidance for larger length-width ratios. The I.A.B.S.E. survey introduced the concept of the column line comprising imaginary beams running

Table 4 Ultimate Strength, Deflection, Tendon Spacing and Tendon Distribution

SUBJECT	A.C.I. - A.S.C.E.	CONCRETE SOCIETY	OTHER
Ultimate Strength	A.C.I. 318 loading cases Bonded tendons: $f_{ps}=f_{pu}(1-0.5\rho_p \frac{f_{pu}}{f'_c})$ Unbonded tendons: $f_{ps}=f_{sc}+10{,}000+\frac{f'_c}{100\rho_p}$ $10{,}000+\frac{f'_c}{100\rho_p}$ - tendon stress increase	C.P. 115 loading cases Tables to determine $\frac{f_m}{f_u}$ and $\frac{d_n}{d_1}$ for bonded tendons and $\frac{f_m}{p_e}$ and $\frac{d_n}{d_1}$ for unbonded tendons (no allowance for effect of $\frac{\ell}{d}$ ratio as in later CP110)	F.I.P. Stress increase in unbonded tendons based on C.P. 110 with additional values based on $\frac{\ell}{d} = 40$
Deflection	Span/depth ratios for slabs continuous over 2 or more spans. Floors: 40 (max. 48) Roofs: 45-48 (max.52)	Span/depth ratios for slabs continuous over 2 or more spans Floors: 42 (max. 48) Roofs: 48 (max. 52)	
Tendon Spacing	Column Strip (width 0.25ℓ on each side of column centreline) Max: 4h Middle strips: Max: 6h but can increase to 8h in short spans	Column Strip (as A.C.I.) for unbonded tendons Max: 4h Middle strip for unbonded tendons Max: 6h Generally Max: 9h	
Tendon Distribution	Simple spans: 55-60% in column strips Continuous spans: 65-75% in column strips If $\ell/_w > 1.33$	As A.C.I. but if $\ell/_w > 1.33$ 50% of tendons in longitudinal direction in column strips and 100% of tendons in short direction in column strip.	F.I.P. and I.A.B.S.E. Column line (column size plus slab depth) to contain at least 50% of tendons).

f_{ps} - stress in tendons at design load (p.s.i.)
f_{pu} - ultimate tendon stress (p.s.i.)
f_{se} - effective stress after losses (p.s.i.)

f_m - stress in tendons at design load (N/mm^2)
f_u - ultimate tendon stress (N/mm^2)
p_e - effective stress after losses (N/mm^2)

through the columns of width equal to the column width plus slab depth. Any arrangement having at least 50 per cent of the tendons acting in the column lines is considered acceptable. This procedure has been adopted by F.I.P. since tendons close to the columns contribute more to the load carrying capacity. It also concentrates tendons within the shear perimeter thereby assisting in the shear carrying capacity of the slab. A typical tendon layout is shown in Figure 5. Dowrick (35) has attempted to utilise this new concept of banding in his cost study in accordance with recent American practice (39). However, difficulties were experienced in the case of the coffered slab because the position of the tendons is restricted to the ribs.

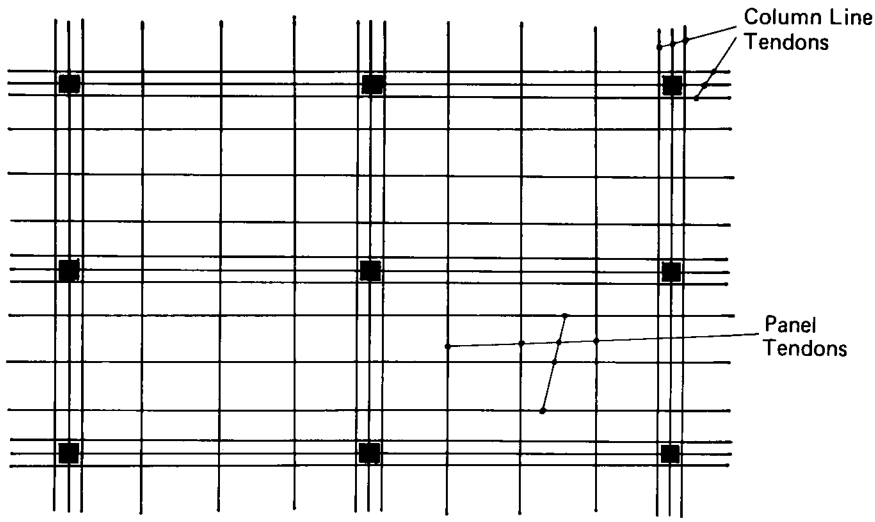

Figure 5 Typical Layout for Banded Tendons

DETAILING AND CONSTRUCTION

A particularly important aspect of detailing in post-tensioned slabs is that carried out to compensate for the horizontal movement due to prestressing and subsequent shrinkage and creep. All documents warn of the need to consider restraints from other structural members and the degree of prestress absorbed by them. The use of temporary open construction joints between adjacent slab sections is one solution, the alternative being the use of sliding joints or hinges. The A.R. suggest a maximum length between joints of 45 metres. If unbonded tendons are used it is preferable to subdivide the slab into sections by providing intermediate stressing anchorages and thereby preventing complete collapse in the event of excessive forces due to explosions or vehicle impact. Typical construction details are shown in Figure 6.

The tendon layout usually places the point of contraflexure at the $^1/_{10}$ span points and aims to achieve the maximum drape possible. The tendon distribution shown in Figure 5 may be impractical due to the concentration of tendons in the column head area and it may be necessary to increase the spacing of tendons spanning in one direction. There are several proprietary flat slab anchorage systems on the market generally utilising 13 mm or 15 mm, 7 wire strand. Anchorages are available for single strands or for horizontal groups of up to four strands and most systems can be adapted for either bonded or unbonded use.

The use of shear reinforcement will, of course, be avoided wherever possible but if deemed necessary it should be arranged on the basis of idealised beams framing at right angles into the columns and enclose both tension and compression reinforcement. The use of castellated shear links has been found convenient in some cases (26). Alternatives are the use of a structural steel shear head but this may be costly and mats of heavy non-prestressed bars over the columns may be more suitable provided the concrete can be adequately compacted.

For economy in construction a rectilinear plan form should be adopted wherever possible with end spans shorter than intermediate spans and components such as columns standardised to permit re-use of formwork. Shear walls will be required for buildings above 15 to 20 storeys and these should be located centrally if possible to reduce the effects of prestress shortening.

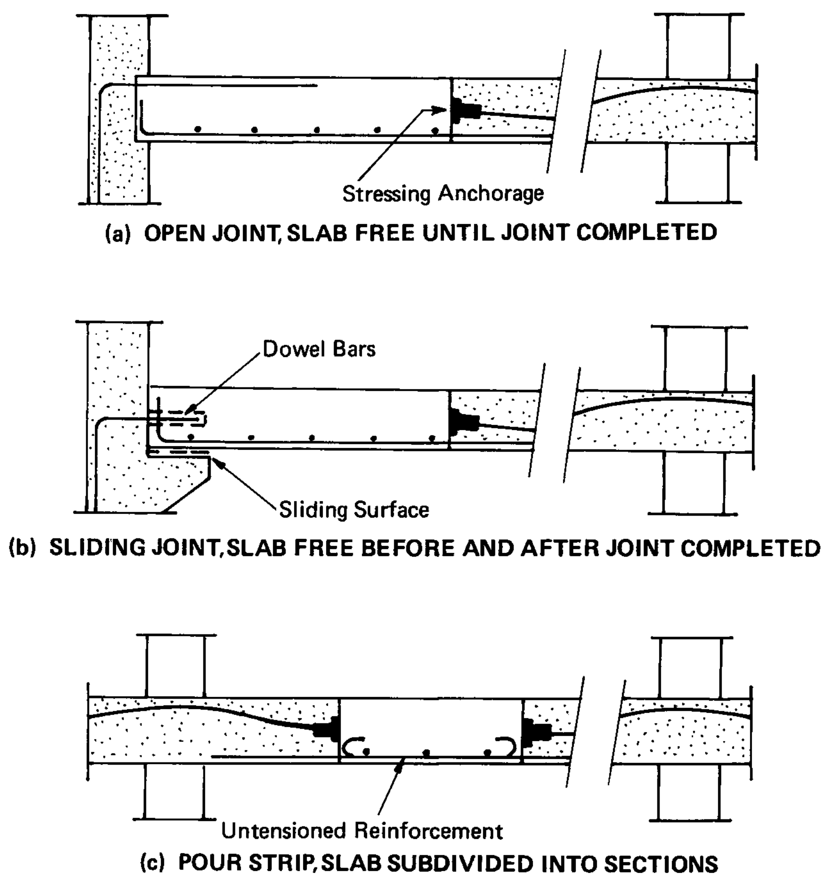

Figure 6 Typical Construction Details

CONCLUSIONS

It is to be hoped that the current effort to regularise design procedures for post-tensioned slabs will lead to an increased use of this potentially economical form of construction in Europe and particularly in the U.K. The new F.I.P. recommendations are a first step in this direction.

At the present time several National Codes, including C.P. 110, do not make specific recommendations for prestressed slabs. There is an urgent need for the new U.K. recommendations based on C.P. 110 to be made available and serious consideration should be given to incorporating them in the Code itself. It is hoped that in reviewing their recommendations the Concrete Society have drawn on the extensive experience gained in other parts of the world particularly with regard to allowing higher average levels of prestress, provided due consideration is given to the effects on supporting members.

The question of shear design approach has yet to be resolved, in particular with regard to the critical punching shear perimeter, the contribution of the compressive stress due to prestress and the validity of the use of conventional reinforced slab theory with an equivalent area of untensioned steel. The Concrete Society recommendations are admittedly more conservative than the American but based on an understandable caution to avoid any possibility of sudden shear failures. Further testing and theoretical investigation is required to establish the true behaviour and strength of slab/column connections in a continuous slab system.

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SLAB CONSTRUCTION FOR HABSYSTEM MODULES

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ABSTRACT A unique precast concrete box system for buildings has been introduced in the U.S.A. Known as the HABSystem, it breaks away from the constraints of other box assemblies in that it is essentially a column and beam assembly rather than a bearing wall assembly. Production of precast slabs for the system posed a number of unusual design and construction problems. These problems were solved through a combination of approaches involving structural design, innovative forming and casting techniques, concrete mix characteristics and load tests.

INTRODUCTION

A new and unique precast concrete box system for buildings has been introduced in the U.S.A. It has been used on several buildings, an eighteen-story hotel in Stamford, Connecticut, being the most recent and refined example. The system, known as the HABSystem, breaks away from the constraints imposed by other box assemblies in that it is not a bearing wall structure, but is essentially a column and beam assembly as shown in Figure 1. Each box acts as a beam spanning between cast in-situ columns spaced across the width of the tower.

Although other precast box systems are currently available, most are variations of bearing wall systems with upper floor boxes bearing on lower ones. Bearing wall approaches usually lead to a solid wall configuration often combined with solid ceilings and floor slabs, resulting in a comparatively heavy unit. One objective of the HABSystem has been to reduce module weight significantly by using a ribbed configuration in walls, ceiling and floor slab. Thus, savings accrue in materials, transportation (both land and sea), erection and foundations.

The system maintains the flexibility to be used as a load bearing system for low rise construction. However, it is also designed and has had principal use in high rise structures by supporting the modules in a uniquely cast in-situ frame, using the modules themselves as formwork.

The HABSystem deliberately set out to fit into the middle ground between an open and closed system. The system is adaptable to many architectural variations as described by McDonald and Rich (1), but they must fit within the context of the mass production operation, for it is through standardization and assembly line techniques that cost savings are made, and more importantly, by imposing repetitive operations, productive capacity is easily expanded. Principal applications of the

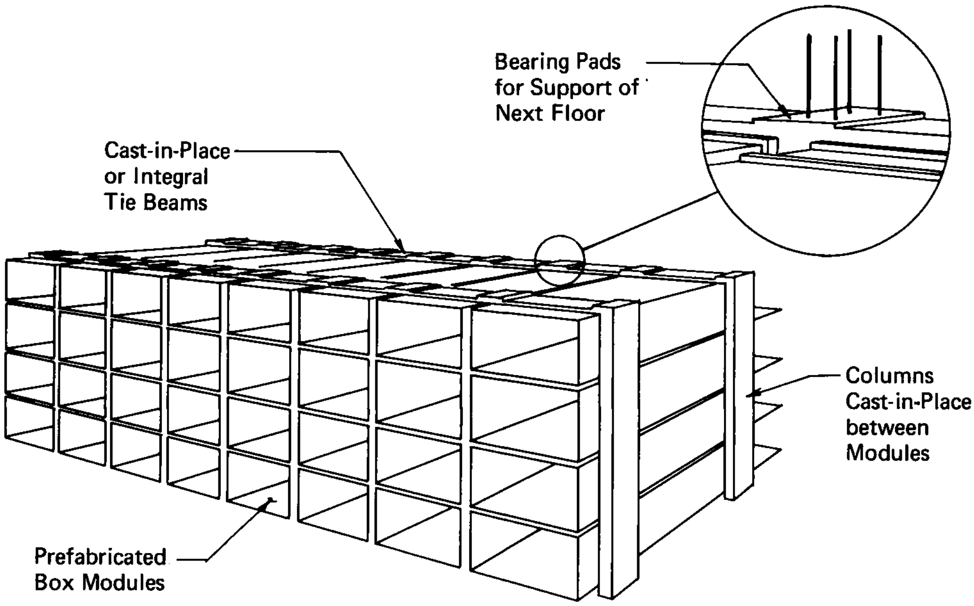


Figure 1 Schematic Layout of a Building Utilizing HABSystem Modules

system include hotels, multi-family housing, hospitals, dormitories, military barracks, and other applications with largely repetitive subdivided space.

Structural plans and details of a typical module for the above mentioned hotel are shown in Figure 2. A number of variations were also utilized, including modules having shorter length, major openings for stairwells and elevator shafts and door openings in walls between adjoining rooms. Since each box is a self-contained unit, it is susceptible to pre-finishing. For the hotel, for example, interiors, plumbing, wiring, heating and cooling units were all factory installed. Only risers and plug-in connections were required at the site (2).

The module is factory produced using an assembly line approach as shown in Figure 3. The present plant has four production lines with each subdivided into construction activity stations. The floor slab is cast at the first station, then the walls and ceiling are cast at the second station to complete the tunnel configuration. The tunnel mould is stripped at the third station and the structure inspected, and at subsequent stations, partitions, piping, plumbing fixtures, electrical wiring and fixtures, mechanical units, doors, windows and interior finishes are added. The production of precast slabs for the HABSystem, as shown in Figure 4, posed a number of unusual design and construction problems. These problems were solved through a combination of approaches involving structural design, innovative forming and casting techniques, concrete mix characteristics and load tests.

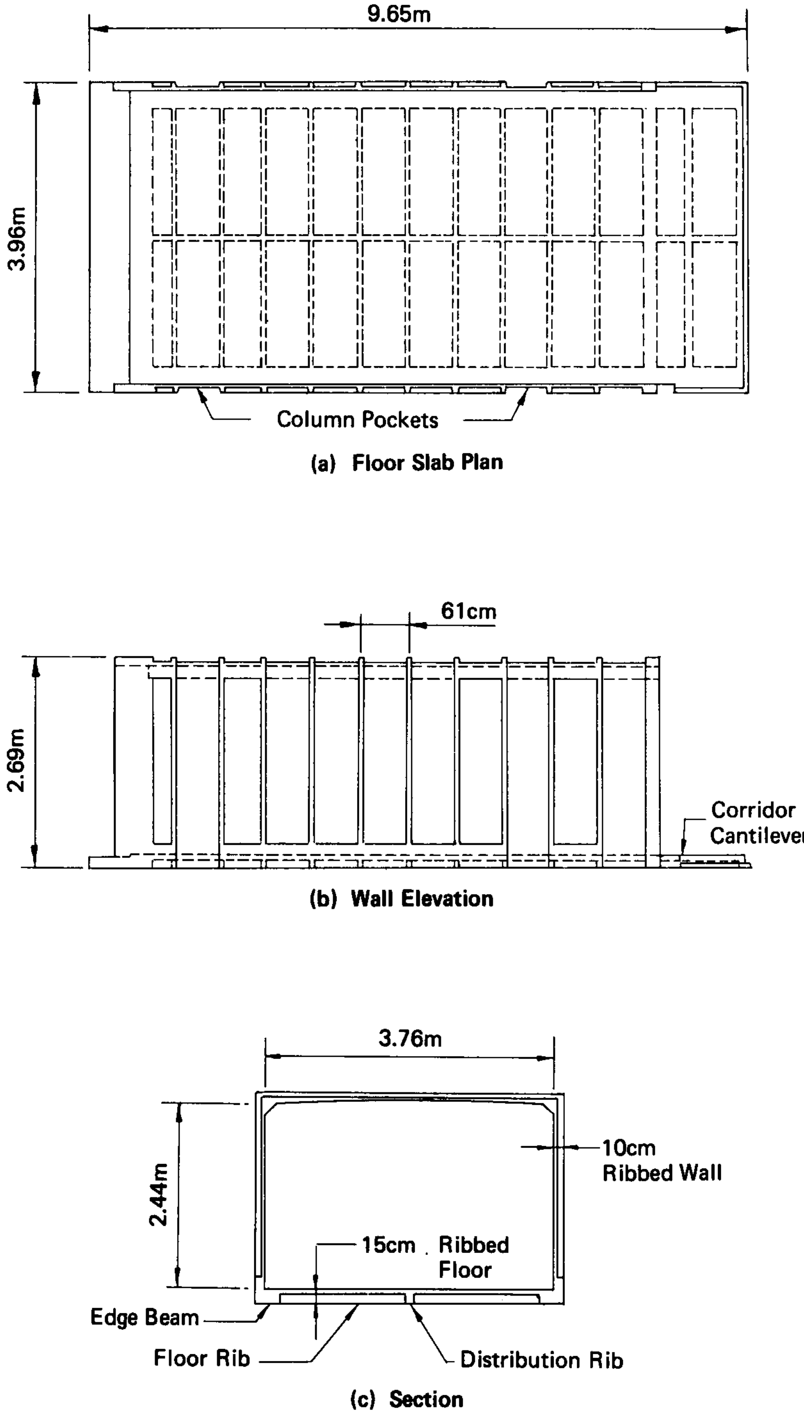


Figure 2 Typical Box Module

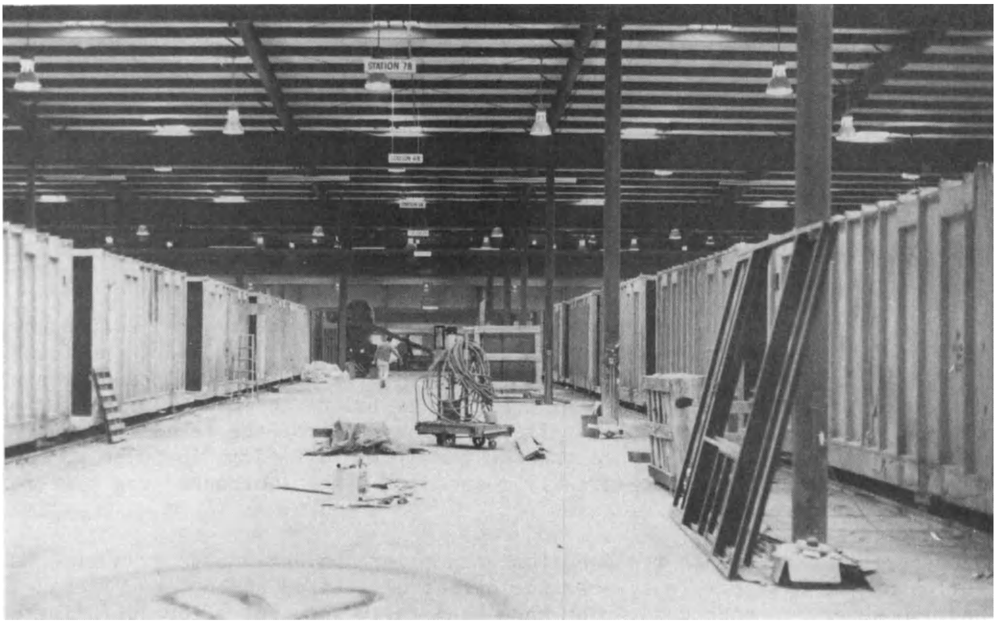


Figure 3 Module Production Lines

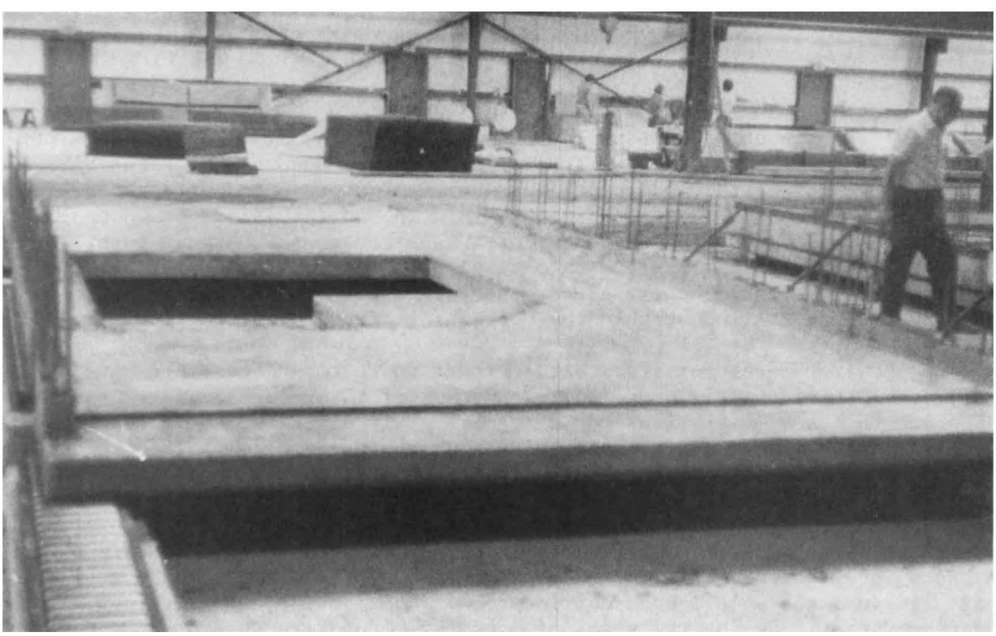


Figure 4 Stripped Floor Slab with Stairwell Opening

FORMING SYSTEM

Although the first prototype module was produced using timber and plywood formwork, it was recognised from the beginning that a more sophisticated approach was necessary. Several factors were important in evaluating the formwork system, these included initial cost, number of re-uses, cleaning effort, appearance of cast surface, flexibility, availability, stiffness, weight. Steel seemed to be an obvious material, it already being used extensively in the U.S.A. for pans in one and two way ribbed slab construction as well as in mass produced precast elements. However, forming sheet steel to the desired floor and wall pan configurations proved to be an expensive approach when only limited numbers were needed for initial production in 1969.

The search for a suitable material led to fibreglass. At that time, unlike now, the use of fibreglass forms was unusual. Nevertheless, the necessary pan forms could be made to order for an acceptable price in view of the developmental stage of the system. Performance of the fibreglass has led to its use in forming virtually all surfaces of the module. After an assessment of the technology and equipment required, it was decided to make the fibreglass forms in-house at the HABSystem plant. This approach greatly reduced cost and shortened lead time for formwork preparation.

The contact forms have been replaced for each major project due to gradual evolution of the system. Thus, possible number of re-uses has not been established. However, in each case, none have been fully worn out after approximately 100 re-uses of the slab moulds.

For the floor slab, the fibreglass pans are attached to a permanent frame of mixed timber and steel with articulated side forms. The exterior wall pans are mounted on steel frames which are hydraulically retracted and extended. The interior form is a fibreglass covered steel tunnel with retracting sides and ceiling.

TOLERANCES

Dimensional tolerances of most precast concrete building system components are usually of great concern due to fit up requirements. However, with the HABSystem, overall tolerances of the module are less critical since both vertical and lateral variations can be absorbed to an extent within the cast in-situ components. Each floor is easily pre-levelled when concrete is placed at the bearing pads. Since each story of modules is levelled onto itself, tolerance growth problems are effectively eliminated. Nevertheless, the tolerances adopted are those recommended by the precast industry in the U.S.A. for precast construction.

The slab, however, does require special attention to tolerances in two respects. First, since the module is principally composed of thin plates, it is possible that plate thickness variations could significantly alter the total module weight. An excess thickness of only 5 mm on all plates could increase the module dead load by 15 per cent. Dimensions are, therefore, monitored during production and furthermore, selected modules are weighed to assure control of both dimensions and concrete unit weight. Second, maintaining close tolerances on both slab thickness and top of slab elevation (as a horizontal plane) is necessary to produce a level building floor. Many applications of the system have called for floor plans using double loaded corridors. In these cases, the corridor floor is produced by cantilevering the slabs of opposing modules to the corridor centreline. Although the module bearings are levelled, variations in slab thickness or any built-in warp of the slab could result in an uneven corridor. Thus, both thickness and levelness of the slab are monitored during production. Success of the system in controlling dimensions is primarily due to built-in controls imposed by the repetitive use of the mechanized forming system.

CASTING, FINISHING, AND CURING PROCEDURES

The HABSystem factory is equipped with a batch plant for concrete production. The concrete is delivered to a portable pump for placement in both the slab moulds and the tunnel moulds. The nature of the mix used makes it ideal for pumping efficiently. The slab is subdivided into several sections for placement, and within each section, concrete is first placed in congested areas and ribs. An immediate second pass deposits concrete for the plates and upper portion of the ribs. Vibration of the floor slab concrete is accomplished with an internal pencil-type vibrator (external form vibrators are used at the tunnel mould). The kerb concrete is placed in a final pass.

Screeding of the floor slab to a level plane is critical, as discussed under tolerances. Projecting dowels and the wall kerb make screeding across the short direction of the slab impossible. Thus, the screed must span the length of the slab, a distance of approximately 10 to 12 m. The use of a magnesium screed has proved most effective.

The slab is then floated and finished conventionally to meet surface requirements for the particular project. Curing approaches vary according to production schedule demands for re-use of forms. Since the factory is enclosed, ambient temperatures up to 20°C can be maintained and the concrete mix developed can reach the desired stripping strength within the normal 24 hr cycle. Earlier stripping times require accelerations of curing by use of a controlled cycle heat or steam process. Concrete test specimens are left in the same environment as the slab during the first 24 hours.

SLAB CONSTRUCTION LOADS

During the process of module fabrication, the slab is subjected to a series of construction loads. As with most precasting operations, re-use of formwork during each work shift reduces overall cost of manufacture. Fortunately, by designing both the module and the production system to minimise the severity of the construction loads, concrete strength required at demoulding, 13.8 N/mm^2, is relatively low.

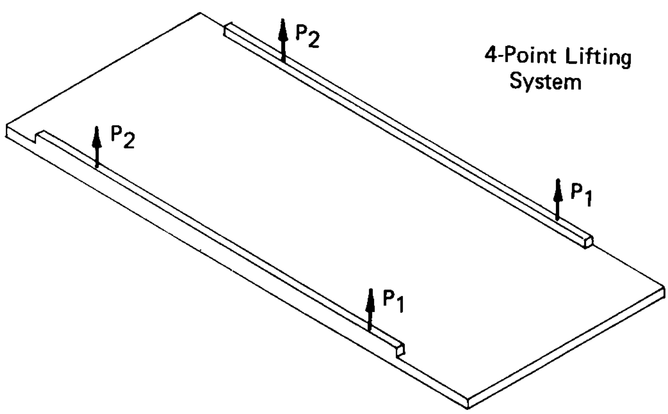
Under normal production demands, one shift per day is currently employed, although a shorter cycle can be achieved with a corresponding acceleration in curing. The first loadings are applied when the slab is removed from its form at the slab moulding station. The slab is lifted vertically by an overhead crane, using the four pick-up points shown in Figure 5 (a). Under this condition, the loading is the slab self weight. The typical floor ribs normally span to the module side walls. However, without the side walls in place, the floor ribs are supported only by the slab edge rib. The edge rib reinforcement is usually governed by this loading condition, which produces concentrated shear forces at the pick-up points plus positive and negative bending moments.

A rectangular spiral is used where edge rib shear reinforcement is required. The shallow depth makes proper anchorage of conventional stirrups impractical. The spiral also provides torsional reinforcement in localized areas as required. From a structural point of view, the lifting inserts would normally be located at the mid-width of the edge rib. However, this location on the floor would later require either patching the floor if a recessed insert were used or cutting off projecting hooks. To avoid both, the projecting inserts are located in the kerb and are later buried within the wall concrete pour.

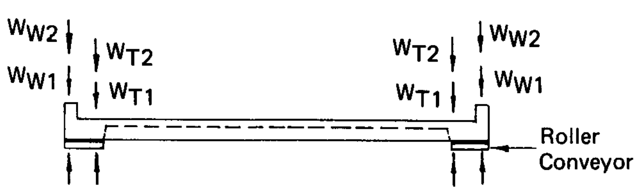
This loading condition is very temporary and its severity is relieved as the slab is immediately deposited on two lines of roller conveyors. The conveyors provide essentially uniform support under each edge rib so that the slab spans simply in its short direction. The slab is then rolled forward into the tunnel forming station.

The next loading imposed is that of the tunnel form as it rolls into position over the slab. The tunnel form moves on two lines of rollers, imposing essentially line loads W_{T1} due to the tunnel weight, as shown in Figure 5 (b). By strategically locating both the tunnel rollers as well as the wide conveyor rollers under the slab, the tunnel load can be transferred directly through the slab without distress. Placement of concrete imposes additional loads W_{W1} due to weight of wall concrete and W_{T2} due to weight of ceiling concrete. Both are also transmitted through the slab. When the tunnel is retracted and removed, the ceiling load is transferred to the wall as W_{W2}.

In service, the slab spans between and hangs from the side walls of the module. The balcony and corridor slab projections are supported by the cantilevering edge ribs. Service live loads are usually uniform but concentrated live and dead loads are easily accommodated in the design.



(a) Lifting Scheme for Stripping



(b) Tunnel Mould Loads

Figure 5 Fabrication Loading Conditions
(a) Lifting Scheme for Stripping and (b) Tunnel Mould Loads

SLAB DESIGN AND LOAD TESTS

For the hotel project, the slabs were designed in accordance with the applicable building codes in the U.S.A., including the Building Code Requirements for Reinforced Concrete of the American Concrete Institute (ACI 318). The required loading of the slabs, on an ultimate strength basis, was equivalent to approximately 4.8 kN/m^2. The computed capacity of the slab was in the neighbourhood of 5 kN/m^2 and two separate load tests have been conducted on the slab, one to a level of 5.9 kN/m^2, another to a level of 7 kN/m^2.

To substantiate the capacity of the slabs, specimens were selected at random from a production line. Not only was the general performance of the slab scrutinised, but special attention was directed to the behaviour of the slab-wall connection. Typical slab details are shown in Figure 6. In the first test, one-half of the load was applied within a 15 min period and allowed to remain in place for one hour. Measurements were taken, and the balance of the load was placed, again within a 15 min period. Measurements were taken immediately upon application of the full load, then the structure was unloaded and remeasured. All deflections conformed to theoretical predictions; only a few hairline, flexural cracks near midspan were observed and upon unloading, the structure returned to its original position.

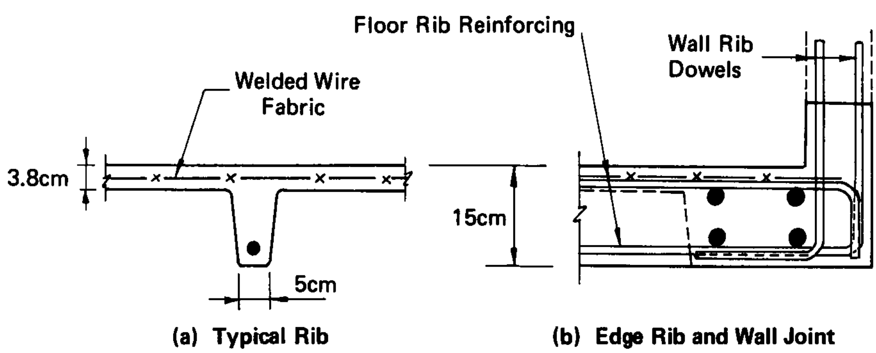


Figure 6 Floor Slab Details

In the second test conducted, the load was allowed to remain on the slab for a period of approximately 8 hr, after which sudden collapse occurred. Prior to collapse, flexural cracking was visible near the midspan region and cracks were also observed on the top surface of the slab near the supports. Cribbing had been placed under the slab to prevent debris from obscuring the performance of the slab. Consequently, the failure was clearly discernible as a flexural shear failure. Large diagonal splits existed at approximately the one-fifth point of the span. In light of the absence of shear reinforcement, the collapse mechanism is predictable. The magnitude of the sustained load and the results of split cylinder tests projected into calculated capacities would indicate that the shear strength was in the upper reaches of the scatter of test results reported by the A.C.I. Committee 426 (3). It is conjectured that either overabundant material properties were enjoyed or the walls of the box were engaged to a greater degree than might normally be expected.

A third test was prompted by building code requirements for concentrated loads. It was felt that there was not sufficient confidence in the structural analysis of the thin 38 mm plate and that load testing was therefore warranted. The applicable code required that the plate be capable of sustaining 2250 N on 50 mm x 50 mm test areas. To duplicate this, a three-legged table was constructed and loaded with sand bags. Care was taken to minimise impact effects but fatigue operated in a contrary direction. The legs of the table were positioned to produce maximum positive bending in one slab span and maximum negative bending in another. In fact no identifiable results could be measured; deflections were monitored and cracking was closely observed, but neither deflections nor cracks could be detected.

CONNECTION DETAIL

The slab-to-wall joint, as shown in Figure 6, is a critical detail. This detail was used because of the constraints established by the architectural requirements as well as for reasons of practical construction. It was recognized that the imposed loading would tend to open the joint. The reinforcing bars that are engaged are, in effect, spliced within a relatively confined region. Similar joints reported in laboratory tests (2) did not perform as well as joints with diagonally placed reinforcing bars. Indeed, the laboratory test results could be viewed as indicating more capacity than the joint in question might develop. It should be noted that the loading of the structure imposes a tension force across one face of the joint. This tension force applies at a construction joint which might be considered as a plane of weakness present in the HABSystem box construction and not present in the laboratory tested specimens.

Given the above concerns, the satisfactory performance of the joint under full-scale tests was reassuring. Further, additional supportive data was obtained from a significant number of experimental units that were produced. These units had large size openings piercing the walls of the box. In order to accomplish these openings, the edge beam of the slab must transfer the load of the joists encompassed within the opening over to the jamb line. At these locations, considerable additional tension as well as a complicated three-dimensional state of stress is superimposed. None of the experimental units displayed difficulties at this jamb connection.

Finally, neither the cracking patterns nor the capacities that might be anticipated from laboratory test results were observed in any of over 500 production units that were examined. This satisfactory performance establishes some confidence in the detail utilised.

FLOOR SLAB OPENINGS

Construction experience has indicated that providing a separate, conventionally constructed core or tower for elevators and stairs can slow construction and hamper access of workers during erection. Hence, incorporation of stairwells and elevator shafts within modules is desirable. As shown in Figure 4, it is possible to design the floor to accommodate large openings. Similar openings can be provided in the module ceiling.

The location of large openings is largely determined by architectural requirements. Floor ribs can be interrupted where necessary and larger ribs provided to frame around the edge of the opening. Loads imposed by shaft walls, elevator guide rails, stairs, etc. are supported by the floor slab spanning between module walls.

Elevator guide rail brackets, shaft walls, stair support inserts, etc. are installed at the factory. Steel stairs are installed immediately after the module is erected, which provides immediate vertical access for construction workers and with significant portions of the elevator shafts and inserts in place, installation of the elevator is accelerated.

Smaller penetrations are occasionally required through the slab for piping, etc. These openings are located to avoid the floor ribs. In general, small penetrations through the floor plates pose no structural problems unless significant groups occur.

Design of prefabricated systems normally entails careful integration of all building systems. Piping and mechanical layouts are predetermined so that penetrations of the slabs are formed or sleeved in the factory. Field coring expense and uncertainties are therefore avoided.

CONCRETE MIX DESIGN

The concrete mix design employed for most of the units produced has included a lightweight expanded shale sand mixed with a Type 1 cement, water, vinsol resin and a plasticizer in the following proportions per cubic meter:

Cement	712 kg
Sand	955 kg
Plasticizer	858 ml
Vinsol Resin	663 ml
Water	238 l

Compression tests on 150 mm dia. x 300 mm long cylinders were used to monitor the quality of concrete production, as is standard in U.S.A. construction practice. Strengths averaged 34 N/mm^2 at 28 days with a standard deviation of 2.07 N/mm^2 and a coefficient of variation of 7.06 per cent. The high cement content was chosen to facilitate placement of the mix in highly confined moulds. The complete mix design was selected after extensive investigation of alternative combinations and the impact of other influencing factors including fire resistance ratings, costs and early strength for stripping. In addition to the variations in mix designs, investigations were conducted to determine the usefulness of super plasticizers, fibrous concrete, artificial heat, slurry mixing and re-vibration (5).

Since it is recognised that the current mix design does not optimise cost, these investigations are on-going. The results of these investigations are difficult to implement since fire resistance requirements must be maintained, and there is very little empirical basis for extrapolating data in fire resistance projections. Fire resistance testing, on the other hand, has been found to be time consuming, expensive and erratic.

DRYING SHRINKAGE

During initial development of this system, concrete precasting operations were approached with some reservations and alternatives were under consideration. Concern centered on the amount of shrinkage cracking that might be encountered. After the first results came in, the use of concrete was re-examined since shrinkage cracking was in excess of even the most pessimistic projections. It had been recognised that a large volume of water was in the concrete and since this is the primary cause of shrinkage every recommended precaution was taken, including lowering the temperature of the mix to between 6 to 12°C. Nevertheless,

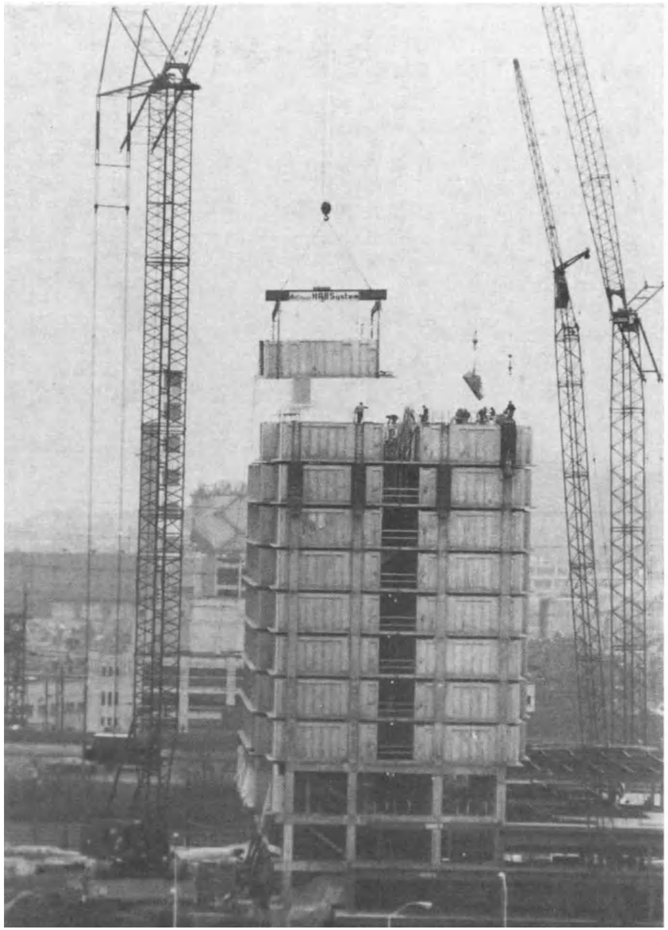


Figure 7 Erection of HABSystem Modules

Figure 8 Completed Hotel Project

initial results were uniformly poor. Cracks appeared after one day and within three days gave the appearance of plastic shrinkage cracking. That is, deep, wide and abundant crazing developed, similar to mud cracking in a dry river bed, which would have made the product unacceptable. The results were not confined to a single slab casting but occurred in all early developmental prototypes cast. However, the shrinkage cracking was eliminated prior to commercial production. It is believed that early stripping may have been, by itself, a good and sufficient solution to the shrinkage problem. Normal precautions have been maintained but it is held that the solution was found by removing the slab from the forms and setting it on movable supports. The slabs are stripped in 12 to 16 hr and placed on rollers which permit nearly unrestrained shrinkage, the only restraint remaining coming from the embedded reinforcing steel. This phenomenon holds true for the production of the complete box unit as well as for the slabs.

Occasional cracks, attributed to shrinkage, have been observed in some boxes which were incorporated into building structures, and thereby restrained, at an early age. Other boxes, allowed to mature during factory installation of utilities and finishes, as well as during a storage period, have performed satisfactorily.

SLAB PERFORMANCE

Performance of the slabs as an integral part of the modular box system has proved satisfactory in the applications undertaken thus far, principally apartment, motel and hotel buildings. Erection operations for the hotel structure previously mentioned are presented in Figure 7. Three hundred and seventy-five modules were used in construction of the tower and the completed facility is shown in Figure 8.

CONCLUSIONS

The production and performance of concrete floor systems is basic to virtually all building structures. The development of new building systems must recognise the importance of the slab. In the development and design of the HABSystem, it has been necessary to consider many factors, including production requirements, forming systems, construction loads, mix characteristics, fire ratings, shrinkage, erection tolerances, in-service strength and deformation, in order to arrive at a workable solution.

ACKNOWLEDGEMENTS The HABSystem was developed by Alexander D. McDonald and Frank D. Rich, Jr., president of the F.D. Rich Housing Corporation. The hotel used as an example is located in Stamford, Connecticut, U.S.A. The architect was Victor H. Bisharat, the structural engineer was McDonald & van der Poll and the general contractor was the F.D. Rich Housing Corporation, all of Stamford.

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PRECAST CONCRETE FLOORS

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ABSTRACT Precast flooring is a proven economical construction system; types of floor and their suitability for various purposes are reviewed. Design in accordance with C.P. 110, and the effect of the Building Regulations in relation to fire resistance, sound insulation, and stability are discussed. The provision of bearings, services and the effect of dimensional deviations are considered together with costs of various types of construction.

INTRODUCTION

Despite recent changes in Building Regulations and adverse publicity brought about by the use of high alumina cement and calcium chloride, the precast concrete industry, although much reduced in size, has a well recognised part to play in building construction.

The arguments in favour of precast construction are well known and are particularly applicable to flooring, where there is no doubt that for most circumstances a fully precast or a composite precast and in-situ solution is available in most areas of the U.K., at less cost and with a shorter construction time than the fully in-situ alternative. The main purpose of this paper is to review the application of precast construction for concrete floor slabs.

TYPES OF FLOOR

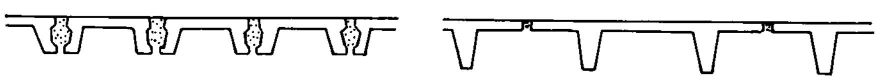
The more commonly available types of floor are: i) fully precast, ii) composite plank or plate floors and iii) beam and block or rib and pot, as shown in Figure 1.

The fully precast floor is generally the quickest and most economical. The cored or voided units are available in a variety of sections, thicknesses and widths to suit requirements for loading, span, fire period, sound insulation and access for erection. The finished floor is light, easily holed for services and can have a soffit suitable for direct decoration. The troughed or T floors have many of the virtues of the cored floors and for long spans could be lighter and more economical; although a ceiling may be required, the troughs can be useful for service runs.

The composite plank or plate floor is slower and more expensive to erect than the first category and tends to be used only where the design requires exceptionally heavy isolated loads to be carried, or there is a particular need to provide structural continuity. The planks or plates come in a variety of forms all being basically permanent shuttering, generally containing the main reinforcement. Those that are prestressed will often require temporary propping during construction, whilst those that are provided with normal reinforcement welded into a lattice often will not.

The beam and block or rib and pot floor is probably only a little slower to fix than the first category and may be cheaper if the advantage of the finished soffit provided by the first category is of no consequence. The beams or ribs come in many shapes, as do the blocks or pots, although there is probably not much to choose between the various types and it is likely that the nearest manufacturer to the site will be able to supply the most economical solution.

(a) Hollow or Cored, Prestressed or Normal Reinforced Concrete



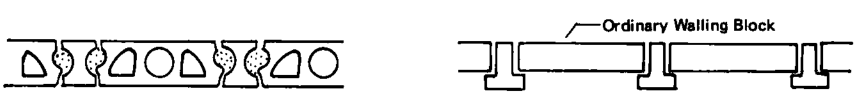
(b) Troughed, Single or Doublet, Prestressed or Normal Reinforced Concrete

Category I Fully Precast

(a) Solid

(b) Cored as I(a) with Structural In-Situ Topping

Category II Composite Plank or Plate Floors



Category III Beam and Block or Rib and Pot

Figure 1 Types of Floor

SUITABILITY FOR PARTICULAR STRUCTURES

The cored or voided floors are suited for use with masonry walling for domestic buildings, particularly where there is a high degree of repetition and where access makes the use of wide units possible and the elimination of plaster to the soffit practicable. They are also very suitable for use on steel frames in offices and shops and in their deeper prestressed forms particularly useful where long clear spans are required. The troughed forms probably have a more limited use, but are suitable where the lightest possible construction is required for a long clear span and advantage can be taken of the troughed form to accommodate services, especially the large trunking required for air conditioning. Double T units have often been successfully and economically used in car park construction.

DESIGN

Most precast floor units are prestressed since there can now be no doubt that with the saving in material content, given an established factory, it offers the most economical product. It took some time to live down the image unfortunately fostered in the early days of prestressing that concrete suitably prestressed could be made to behave like a spring board. However, it has now been amply demonstrated that the product is reliable and the deflections are governed by the normal rules of nature and the modulus of elasticity of the concrete used.

The normal reinforced concrete units do still have a place in the product catalogue, particularly where it might be uneconomic to set up a prestressing bed, the required profile is unsuitable for prestressing, or unusual and requiring special moulds and also where the required rate of production is low.

Whilst the design of units is straightforward in accordance with C.P. 110 (1), it should be noted that the means adopted to control deflections, which were a cause of some embarrassment when designs were stretched to the limit of C.P. 116 (2), is now a matter of trial and error making the normal method of assessing the comparative cost of slabs of different thickness and percentage steel inordinately tedious by hand. It is perhaps also worth noting that, whilst C.P. 110 recognises that the calculation for camber and deflections of prestressed slabs is essentially approximate and allows a variation between units of the same design to be at variance by up to 50 per cent, no similar advice is given for reinforced concrete slabs, where in fact many of the factors influencing deflection are the same and it is evident that apparently identical reinforced concrete slabs can have deflection characteristics that are equally variable. Attempts to calculate deflections of cored reinforced concrete units in accordance with C.P. 110 have only been completed with the assistance of the programme developed for this purpose by Beeby (3).

The design of prestressed units can be approached in two ways in accordance with C.P. 110 and it is with concern that authors find the more rigorous method leads to more conservative design than the simplified method. Examination of the two methods reveals that although simplified methods should result in larger load factors, the reverse is the case at present. Clause 4.3.5.2. has also caused considerable consternation in that it is now apparent that with uniform loads critical sections for shear are at about one-third span and not near the support where they are normally found.

The requirements for the fire resistance of prestressed floors, which appeared for the first time in C.P. 110 and could be incorporated into the next amendment to the Building Regulations, are already being reviewed and it seems likely that the advice given in the Institution of Structural Engineers publication, 'Design and Detailing of Concrete Structures for Fire Resistance' (4), could be incorporated in the next revision of the Code. Whilst the new thinking is splendid and logical, the industry will be faced with a considerable expenditure in amending the designs, details and manufacturing equipment to meet the proposed requirements.

BUILDING REGULATIONS - STABILITY REQUIREMENTS

This is an area, particularly for buildings of four storeys high or less, where ever since the incident at Ronan Point and the publication of advice pointing out the importance of longitudinal, transverse, peripheral and vertical ties to limit the extent of damage due to accident, the designer of precast flooring involved with many local authorities and consulting engineers, has been continually amazed with the extreme variety of interpretations of the available advice. Over four storeys the requirements are clearer as at least the building regulations indicate what has to be achieved, although the methods adopted have been equally varied and inconsistent.

It is necessary to point out that much of the present difficulty stems from the use of such words as 'robust and stable design', first appearing in C.P. 110 and subsequently in the Institution of Structural Engineers' report, 'Criteria for Structural Adequacy of Buildings' (5), and more recently in B.S. 5628, 'Structural Use of Masonry: Part 1' (6). It is evident that while everyone knows what robust means, it does appear that there is no common definition in useful engineering terms, and with no agreement the matter continues to be confused.

In 1978 the Institution of Structural Engineers held a symposium to discuss 'Low Rise Hybrid Building' (7) specifically to try and define the requirement for buildings of four storeys high or less and reference to the symposium proceedings shows the difficulties in arriving at a workable set of rules. The committee for B.S. 5628 in earlier drafts made efforts to codify the requirements in engineering terms, but in the final publication have had to admit defeat and return to the same phrase, i.e. 'to ensure a robust and stable design'. It is surely time that the B.S.I. forbade the use in Codes of words such as adequate, suitable, generally, etcetera and robust, unless they are defined in quantified engineering terms.

We have, however, progressed from the days when the Greater London Council were busy turning load bearing wall jobs into virtually fully framed buildings, when it was thought necessary with timber or precast floors to provide an in-situ concrete capping course to walls to contain the vital transverse ties, and when the vertical ties could only go in an in-situ concrete vertical member formed where a section of wall was omitted. It is also seldom, in recent months, that the specifier has explicitly stated his requirement that floor ties be provided as recommended in C.P. 110. There is a general acceptance that a steel frame usually does ensure that the building is effectively tied together, and a cellular masonry building has never been prone to excessive damage due to accidental causes.

There does, however, remain the light steel frame that is not adequately braced, and the masonry construction that is not cellular. Both may be structures sensitive to movement and liable to what has commonly been called progressive collapse. In these cases it is clear that longitudinal, transverse and peripheral ties in the floor may be very useful indeed in achieving that highly desired quality of robustness. In circumstances where the floor ties are a vital requirement, composite construction clearly offers the same facility as in-situ construction for the provision of steel in all directions; however the low cost of machine made cored floor units demands very careful examination of the possibility of providing adequate ties within the disciplines imposed by the nature of the product.

It will be appreciated that a slab made by a slip forming or extrusion process and subsequently cut to length, cannot by its nature have reinforcement projecting from its surfaces, or have ends other than square, unless the additions are made by hand immediately after the passage of the machine. It was found impractical, with the very dry concrete used, to poke reinforcement into a unit and expect to develop any bond. However, it has been found practical, not demanding excessive skill and not excessively costly, to carry out minor hand forming work immediately after casting.

The most simple operation is to form the mini notch, Figure 2(a), which will provide the space necessary to accommodate a transverse tie bar. This is then extended to the maxi notch which will accommodate more and bigger tie bars or may be used on steel shelf angles, Figure 2(b). Where narrow units are used there seems little equivocation nowadays that short bars laid in joints at the support do constitute effective longitudinal ties, although it is difficult to understand why units built into a solid wall in the usual way are not always deemed effective. This appears to be particularly illogical in the case of an end wall where it has been contended that the arrangement shown in Figure 2(c) provided a better longitudinal tie than that in Figure 2(d). Where wider units are used the mini or maxi notch can be combined with end slots, as shown in Figure 2(e), which permit additional tie bars to be laid into the cores and concreted in on site. The provision of end slots also leads to the possibility of extending the length of a precast unit and this is considered later under bearings for precast units.

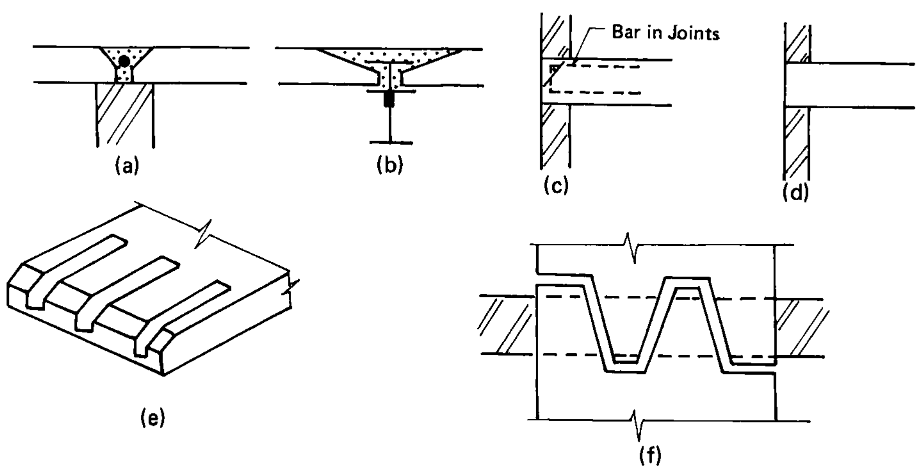


Figure 2 Different Forms of Slab Ties and Bearings

Other Requirements of Regulations

It is perhaps worthwhile to mention two other aspects of the regulations which have to be borne in mind when detailing floors. These are the sections concerned with sound and fire resistance. They are relevant when designing precast units, as cored slabs may be comparatively light and will generally be prestressed, their lightness being relevant to sound insulation and their prestress to fire resistance.

With regard to sound there are several aspects of regulations that should be noted. The first is that the regulations only state that the sound insulation should be adequate, nothing else. There then follows the 'deemed to satisfy' clauses mostly dealing with construction, but one dealing with a test, which if passed will permit the construction to be deemed satisfactory. Whilst not disagreeing with the principles of the 'deemed to satisfy' construction, it is our experience that construction in accordance with the 'deemed to satisfy' clauses will not generally pass the 'deemed to satisfy' test, and even if it does, the construction may still not be regarded as adequate, a truly confusing and iniquitous situation.

One other sound requirement worthy of mention is where the edges of floors are required to be built into the inner skins of cavity walls, thus ensuring that load may be carried, or the floor is supported where it was not intended to be, or in practice, a bit of each. This is of course a most undesirable construction feature; it would be preferable for the slab to be able to move, relative to the wall.

With regard to fire the current position has been referred to already in the section dealing with Design. The proposals contained in Reference 4, whilst affecting both the rules for reinforced and prestressed concrete, do appear to have a touch of common sense and it is to be hoped that in due course one will know just what is necessary.

Another cause for recent concern has been the introduction of the requirements for cavity barriers. The authors, in discussion with the Department of Environment, have established that the rules were not intended to apply to cavities in cored floor units. However, it is apparent that in interpretation of the regulations as written, such cavities would appear to be within the scope of the rules. Under the auspices of the Federation of Concrete Specialists, discussions are currently taking place to try and modify the wording to express the original intention more clearly.

BEARINGS FOR PRECAST UNITS

The maintenance of adequate bearings probably causes more contentious discussions than any other aspect of precast floors, and again the problem arises from the wording of C.P. 110 since anything less than 100 mm on masonry or brickwork and 75 mm on steel or concrete is viewed with disfavour. Section 2.3.4 of the 'Structural Joints in Precast Concrete' Manual (8) tries valiantly to bring some reason to bear on the problem and whilst appearing to add complication, does lead to a logical assessment of the requirements.

There are two plain and inescapable facts. The first is that however closely the matter is considered in the design office there will come a time when the tolerances all add up the wrong way and the minimum or net bearing width will not be achieved. The second is that however small the bearing, provided the bearings do not move apart the unit will in all probability carry the required load. This leads to a general feeling that small bearings on steelwork should not be very worrying, but could be more so on masonry, particularly where soft lightweight blocks are used for bearing walls.

The situation generally arising in domestic building is a series of spans over living and bedrooms, where it has often been thought possible to take a double bearing on an intermediate wall of 100 mm thickness. It has indeed been done so many times that one is almost persuaded that it is a practical construction, although this should not be considered a valid construction using simple bearings, but may be when used in conjunction with extended bearings. It is possible and economical to replace two spans with a single span using a lightweight division wall in place of the load bearing wall. Alternatively, the two spans may be made and erected as a single span, but designed to be supported intermediately by a

100 mm thick wall. Special shaping of ends has been used, as shown in Figure 2(f), to give the required simple bearing, but has caused manufacturing and planning complications, particularly where 'handed' units have been required.

Much trial and tribulations has been brought about by the wording of C.P. 110, viz. 'this bearing may be reduced at the discretion of the Engineer, taking into account relevant factors such as tolerances, loading, span, height of supports and the provision of continuity reinforcement'. It is hoped that the Manual (8) will go some way to identifying solutions which could be considered acceptable. The concept of extended bearings, Section 2.3.5 of the Manual (8), is welcome and can in fact make it entirely practicable to support a series of precast floor units on narrow walls in exactly the same way as an in-situ slab. Unfortunately the Manual then has a spate of 'adequates' and 'suitables' and goes on to suggest measures that the tests which the Manual suggests as 'suitable' for establishing the 'adequacy' of extended bearings show to be quite unnecessary. There is not space in this paper to report the tests in details but is is quite clear that extended bearings can be made simply as shown in Figure 3.

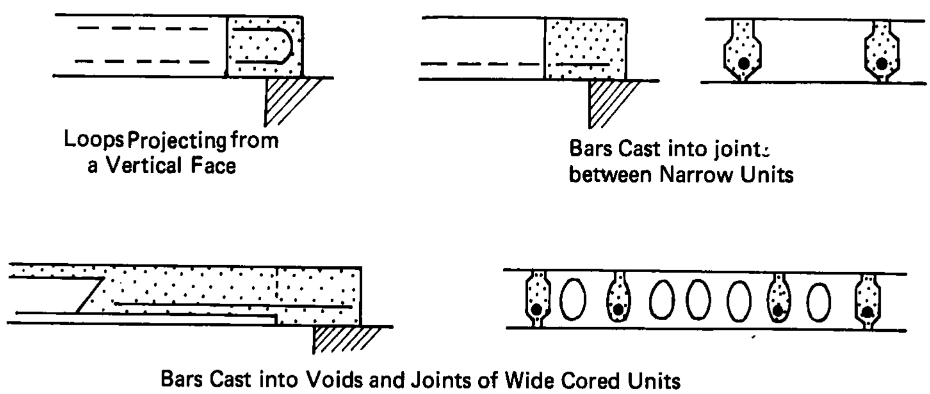


Figure 3 Methods of making Extended Bearings

It is necessary where narrow width bearings are used for erection purposes, but have provision for extension, to see that the erection is carried out on soft packings to ensure load is not concentrated near the edge of the bearing.

PROVISION FOR SERVICES

It is frequently argued that provision of service openings in precast flooring is more difficult than for in-situ construction and the authors would not disagree that the formation and trimming of a large opening at the last minute is indeed more easily accomplished, but would strongly maintain that there is more virtue and economy in having the requirements fully planned before construction is started, in which case there is usually little difficulty in finding a satisfactory precast solution. There are certainly advantages in cored precast flooring when numerous small holes can be easily and cheaply cut or drilled through areas where cores occur.

Over the years manufacturers of precast flooring have endeavoured, with varying success, to produce floors capable of receiving finishes direct to top and soffit.

While one can achieve a soffit which with a very minimum of improvement is suitable for decoration directly, however, there are so many advantages arising from the use of an in-situ top finish that the continued efforts of the precaster to do so does not make economic sense.

DIMENSIONAL DEVIATIONS

This subject is well covered in the Manual (8) but there is one aspect which should be mentioned here and this is camber of prestressed units. By now almost everybody realises that prestressed units will camber and many will prefer to see this to the sag of normal reinforced concrete slabs. However, it is the differences in camber that cause the trouble and it is easy to see why with the necessary tolerances for manufacture, and the inevitable variation in the properties of the concrete, some degree of difference between adjacent units is bound to occur; in the May 1977 revision of C.P. 110 there is some helpful advice on this matter.

For many years the authors strongly believed in the virtue of providing V joints between units, which can effectively disguise up to 6 mm of difference between adjacent units in an office with a relatively high ceiling, and will also make the crack that has a habit of forming between adjacent units less perceptible. However, the general non-acceptance by architects of lines on the bedroom ceilings of domestic buildings (unless of course they are symmetrically spaced), a general feeling that the cost of tidying up arrises which have become chipped for various reasons is more than the cost of flushing out joints, and the realisation that if the flushing out is done sensibly and the joints taped before decoration, then the cracks do not appear, has now convinced the authors that the V jointed soffit has no place in domestic construction work.

COST OF PRECAST FLOORS

In these days of inflation and at a time when British Steel is making losses of the order of £1 million per day it is difficult to make any true assessment of cost, but it is apparent that with today's prices a concrete floor can be provided in a dwelling house for about the same price as a traditional timber floor. Provided it is accepted that both floors are going to have a carpeted finish, then the concrete floor will have the advantage of better sound insulation, fire resistance and durability. It must be said, however, that to make the precast solution viable it will be necessary for a number of houses to be ready for flooring at the same time and this must generally preclude thoughts of using anything other than timber where houses are built one at a time, although the use of beam and block does sometimes offer competition in this circumstance. Table 1 gives some idea of the prices in September 1978 for typical construction.

GROUND FLOORS

After the summer of 1976 there was a great deal of concern regarding the troubles experienced with ground floors on sites subject to clay heave, although after two years of more normal weather the concern seems to be somewhat abated. However, it is obvious that when a ground floor requires a void under it for this particular condition, or indeed requires fill in excess of 600 mm it is not a case for seriously considering in-situ construction and a fully precast or composite floor not requiring any propping must be the sensible and economical solution.

Table 1 Typical Prices, £/m^2, for Precast Flooring, as at September 1978

(Prices are for site 20 mile radius from Iver, Nr Slough, Berks, with a total job of 1000 m^2. They do not include tie steel but Fixed price includes all grouting up and in-situ make-up strips. Prices for Plank include distribution steel as R6 @ 600 mm centres.)

PRODUCT	JOB					
	Flats (1)			Store (2)		
	5000x110 (3)	6500x150	7500x200	4000x110	5500x150	6500x200
Dry Cast 400 (4)						
S.W., kN/m^2	2.158	2.544	3.254	2.158	2.544	3.254
D.O.	9.42	10.56	11.23	9.35	10.56	11.84
Fixed	12.64	14.80	18.09 (9)	12.17	14.26	15.60
Dry Cast 1200 (5)						
S.W., kN/m^2	2.158	2.327	2.934	2.158	2.327	2.934
D.O.	9.48	10.09	11.16	9.48	10.09	11.16
Fixed	10.49	11.23	12.37	10.29	10.96	12.04
Wet Cast 400 (4)						
S.W., kN/m^2	2.017	2.364	2.758	2.077	2.364	2.758
D.O.	9.68	10.89	11.43	9.55	10.89	11.97
Fixed	12.98	15.00	18.36	12.24	14.46	15.80
Wet Cast 1200 (5)						
S.W., kN/m^2	1.992 (7)	2.292	2.677	1.992 (8)	2.292	2.677
D.O.	9.62	10.69	11.50	9.62	10.69	11.50
Fixed	10.69	11.90	12.78	10.42	11.63	12.51
Composite Plank Dry Cast 400 (6)						
Depth, mm	65+85	100+75	100+125	65+60	100+75	100+125
S.W., kN/mm^2	3.503	4.108	5.284	2.914	4.108	5.284
D.O.	7.26	10.22	10.22	7.40	10.02	10.02
Fixed	15.00	18.76	23.74	13.58	18.16	21.92

Note: (1) Masonry walling not exceeding 4 storeys; Loading: S.I.L. 1.5, Partitions 1.0, Finishes 1.5; Fire period 1 hr; 4 No. visits.
(2) Steel frame not exceeding 10 m lifts; Loading: S.I.L. 5.0, Finishes 1.5; Fire period 1 hr; 1 No. visit.
(3) Span x Depth.
(4) Assume <u>no</u> access for crane.
(5) Assume access for crane.
(6) Planks assumed propped.
(7) Span 4700.
(8) Span 3900.
(9) Includes extra labour due to length and weight of unit.

CONCLUSIONS

Firstly we conclude that there is for nearly every situation a fully precast, or composite precast and in-situ, floor construction which will offer not only saving in materials and cost but also construction time. This being so, designers could with advantage arrange plan forms to suit the product widths of precast products and achieve even more significant cost savings.

Secondly we believe that practising designers are now becoming overwhelmed by legislation, Codes of Practice, and publications by the Engineering Institutions which do carry considerable weight, and which may be used as mandatory requirements by checking authorities who often interpret incorrectly the intention of the authors of the documents. Frequently we feel that important design considerations which may basically affect the stability or durability of structures become neglected due to a pre-occupation with some absurd dispute over a design, or detail, which is deemed to be at variance with one of the publications.

Thirdly, the advent of computer aided design does, we consider, tend to be widening the undoubted gulf which exists between the academic and the practising engineer. On the one hand the academic engineer does tend to produce a complicated analysis of a simple portion of an idealised structure, whilst the practising man is working, usually with an imminent deadline, to produce simple practical construction details for complex structures.

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DEVELOPMENT OF A TWO-WAY SPANNING FLOORING SYSTEM USING PRECAST CONCRETE UNITS

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ABSTRACT Most of the techniques currently employed for precast floor construction make use of precast elements spanning in one direction only and supported over the two opposite edges. A new technique of precast floor construction in which it is possible to achieve a two-way spanning system has recently been developed by the Structural Engineering Research Centre (SERC), Madras. This has been successfully adopted in the construction of an experimental low-cost house at Madras, India. Tests conducted on floor slabs cast using the new technique have shown that they exhibit two-way plate action until failure, although some of them showed early crack formation at the joints of precast units. The salient features of design and construction of the new flooring system are described giving details of the experimental work carried out to study the flexural behaviour of the slabs cast using the system.

INTRODUCTION

In most precast floor construction, the precast units are designed to span in one direction only. Considerable savings in cost of construction can be achieved if the flooring system behaves as a two-way slab. In a majority of residential and office buildings, the size of the rooms, as well as the layout of the supporting walls or beams, generally offer sufficient scope for making the flooring system span in two directions. In the case of multi-storeyed buildings, the use of two-way spanning flooring schemes will further assist in reducing the size of the beams and columns and in achieving uniform distribution of load on the foundations below.

At present, two-way spanning floors are built in-situ using conventional centering at the time of construction. The flooring system that has been developed by the Structural Engineering Research Centre (SERC), and is described below, makes it possible to construct a two-way spanning floor using precast concrete components.

DESCRIPTION OF THE SYSTEM

The system essentially consists of precast battens or floor strips which initially span in one direction and are, if necessary, propped from below at the time of construction. Two-way action is achieved subsequently by casting in-situ transverse ribs in the perpendicular direction without the use of any centering by employing small precast concrete trough-shaped components, or filler blocks. The reinforcement required in the transverse direction is threaded through the openings left in the precast joists or floor strips before the transverse ribs are cast in-situ along with a layer of structural screed on the top.

In the first scheme developed at SERC, the precast reinforced concrete battens are arranged over walls or beams at regular intervals to span the shorter direction. Precast reinforced trough-shaped units are then arranged between them one behind the other in a row. The side profile of the trough units meeting each other is so designed as to form gaps between successive units along the transverse direction and perpendicular to the direction of the battens, thus providing the formwork necessary for casting the in-situ ribs and the screed concrete. The precast battens are provided with holes or small openings along their sides, at the time of casting, which coincide with the mid-point of the gaps formed by the trough units after the latter are placed over the battens during erection. The reinforcing bars required in the transverse direction are threaded through these holes from one end to the other. The gaps are then filled with in-situ concrete. A layer of in-situ screed concrete reinforced with a nominal wire mesh is then laid on the top and finished as required. The details of this flooring scheme are shown in Figure 1.

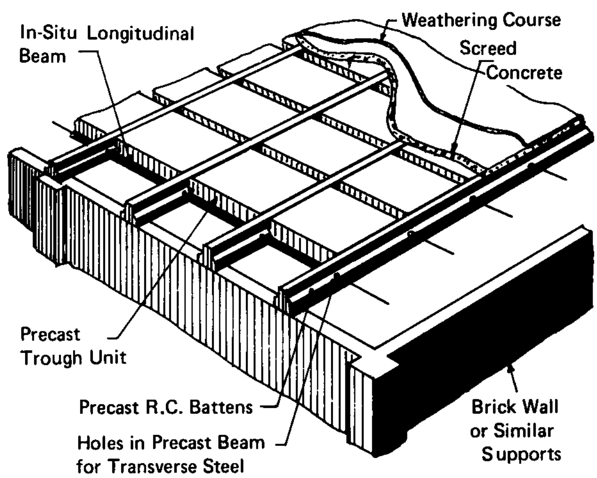

Figure 1 Two-way Slab, Scheme 1

In the other scheme, precast reinforced concrete floor strips in the form of an inverted channel and cast with lightweight, aerated concrete filler blocks, are used. The filler blocks are arranged with gaps between them which are later filled with in-situ concrete to form ribs in the transverse direction. As in the first scheme, the transverse reinforcement required for the cast in-situ ribs is placed in these gaps through the holes left on the sides of the precast floor strips. The details of the scheme are shown in Figure 2.

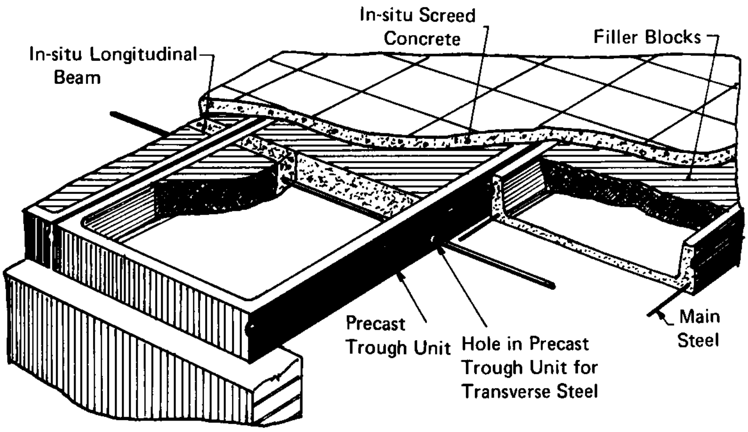

Figure 2 Two-way Slab, Scheme 2

EXPERIMENTAL INVESTIGATIONS

In order to study the application of the proposed flooring schemes to residential and similar construction, laboratory tests were carried out on two sets of two-way spanning floor slabs constructed using the schemes described above. A companion conventional cast in-situ two-way slab was also tested to compare its performance with that of the precast slabs. The test slabs were square in plan and were cast using concrete having a cube compressive strength of about 25 N/mm^2 and plain mild steel bars having a minimum yield strength of 260 N/mm^2. One of the slabs cast using precast channel floor strips (Slab 4) was not provided with the screed concrete in order to study the influence of the latter on the behaviour of the slab. The details of the test slabs are given in Table 1.

The precast battens were cast using timber moulds, while masonry moulds were used for casting the trough and channel units. The companion cast in-situ slab was cast using timber centering. The slabs were cured for a minimum period of 28 days before they were tested.

The slabs were tested under simply-supported conditions, the slabs being loaded on the top by means of a hydraulic jack. The load was distributed to sixteen points using a set of distribution beams and was measured using a calibrated dial-type indicator connected to the jack. Electrical strain gauges and 'Pfender' strain gauge points were used for measuring the compressive and tensile strains in concrete, respectively; the deflections were measured using dial gauges.

The slabs were initially loaded to 2.5 kN. The load was then released and the initial measurements taken. The slabs were then loaded gradually by increasing the load in steps of 2.5 kN, with the strain and deflection measurements being taken at every loading stage until the slabs started showing signs of yielding. The following measurements were taken during the tests:

i) Compressive strains in concrete at the top of the slabs in the middle portion;

ii) Compressive strains in concrete at the top of the central ribs at midspan;

iii) Tensile strains in concrete surrounding the steel at the bottom of the slabs in the central region (either in ribs or bottom flanges, as the case may be);

iv) Deflections at the centre and intersections of the ribs (or flanges and ribs) in the central region.

Table 1 Details of the Test Slabs

SLAB NUMBER	DETAILS OF RIBS		28 DAY STRENGTH N/mm^2
	Precast	In-Situ	
Slab 1 Scheme 1	100 mm x 125 mm with 1M10	100 mm x 125 mm with 1M10	22.5
Slab 2 Scheme 1	100 mm x 125 mm with 1M10	100 mm x 125 mm with 1M10	22.5
Slab 3 Scheme 2	100 mm x 105 mm with 2M6T, 2M8B	100 mm x 105 mm with 1M10	27.5
Slab 4 Scheme 2 (No screed)	100 mm x 80 mm with 2M6T, 2M8B	100 mm x 80 mm with 1M10	27.5
Slab 5 (Conventional in-situ)	-	80 mm x 125 mm with 1M10 (both directions)	30.0

Overall size:	Slabs 1 and 2: 2.2 m x 2.2 m Slabs 3, 4 and 5: 2.1 m x 2.1 m
Effective span:	Slabs 1 and 2: 2.1 m Slabs 3, 4 and 5: 2.05 m
Screed concrete:	25 mm thick, reinforced with 200 mm x 200 mm, 10 gauge welded mesh.

Observations were made on the initial formation of cracks, their widening and propagation, with the slabs being loaded to failure. The failure patterns of the slabs were critically examined. Figure 3 shows the failure pattern of one of the tested slabs, from which it can be seen that the formation of the yield lines at the bottom of the slab is identical to what is usually obtained in a square plate.

DISCUSSION OF TEST RESULTS

Cracking and Ultimate Loads

The cracking and ultimate loads of the five slabs tested are given in Table 2, together with the corresponding theoretical values.

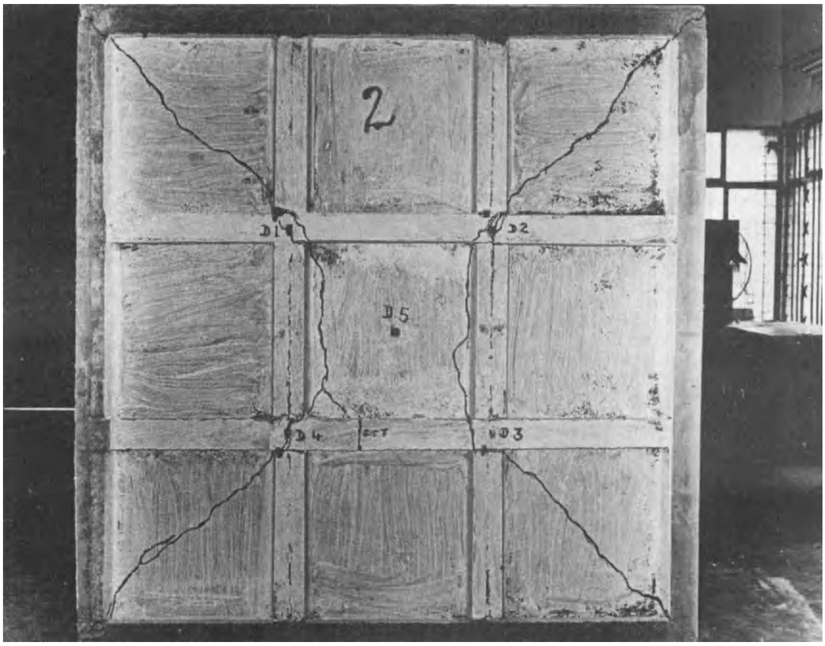

Figure 3 Failure Pattern in Slab 2

Table 2 Comparison of Cracking and Ultimate Loads

SLAB NUMBER	CRACKING LOAD, kN			ULTIMATE LOAD, kN	
	Theoretical	Experimental		Theoretical	Experimental
		Initially at Joints	At Formation of First Major Crack		
1	43.0	not noticeable	70.0	106.4	100.1
2	43.0	35.0	62.5	106.4	98.1
3	90.0	46.5	70.0	124.6	143.7
4	60.0	32.0	60.0	95.5	99.6
5	32.4	-	64.0	110.6	131.3

The theoretical cracking moment and load were calculated by assuming the modulus of rupture of concrete as $0.63\sqrt{f_c}$ N/mm^2, f_c being the cylinder strength of concrete, and by using Timoshenko's coefficients for plates and the gross moment of inertia of the uncracked section (1). The ultimate loads were computed by the yield line method, using the procedure suggested by Shukla (2).

It may be seen from Table 2 that the observed ultimate loads are, in general, in close agreement with the computed values. A lower ultimate load was registered for Slab 4 due to the absence of the screed concrete. The results have proved the two-way action of the slabs, up to failure in the case of slabs cast using the battens and trough units (Scheme 1), and up to about 70 per cent of the failure load in the case of slabs cast using the channel strips (Scheme 2). In the latter case, separation between the precast units along their joints was noticed even at the early stages of loading before the two-way action was established by the presence of the transverse steel across the joints. This scheme, therefore, requires further investigations before it could be recommended for large-scale application.

Strains

The strains at different loading stages at the top and bottom faces of precast and in-situ transverse ribs in one of the slabs tested (Scheme 1) are shown in Figure 4.

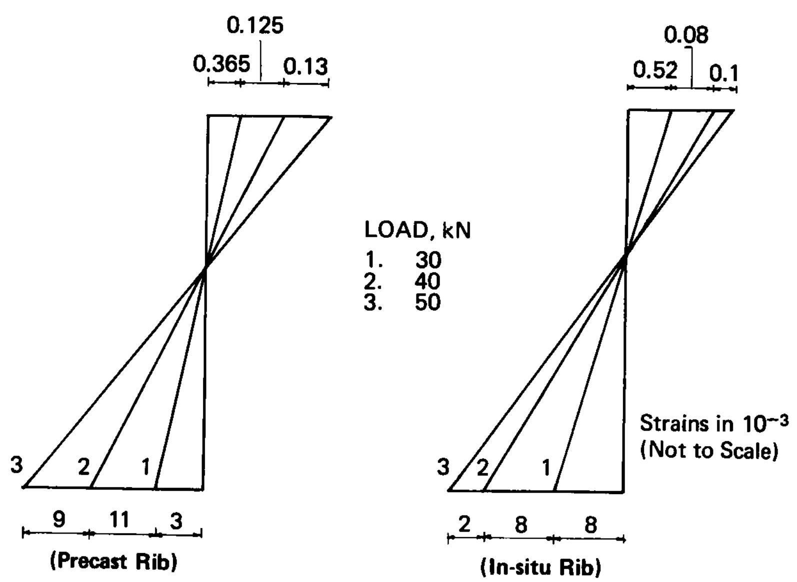

Figure 4 Strains in the Ribs

The initial development of tensile strain in the ribs was not the same. This was expected, since the joints between the precast elements opened up initially, before the reinforcing bars in the transverse ribs could take up the strain caused by the loads. At later stages of loading, the strains in the ribs developed at almost the same rate until the formation of the yield lines. This, as well as the strain profile shown in Figure 4, suggest that the load was shared by the ribs almost equally, thus confirming the two-way action of the slab using Scheme 1.

Deflections

As stated earlier, the deflections below the slab were measured at every loading stage in both directions. The relationship between load and central deflection for one of the slabs tested (Slab 2) is shown in Figure 5.

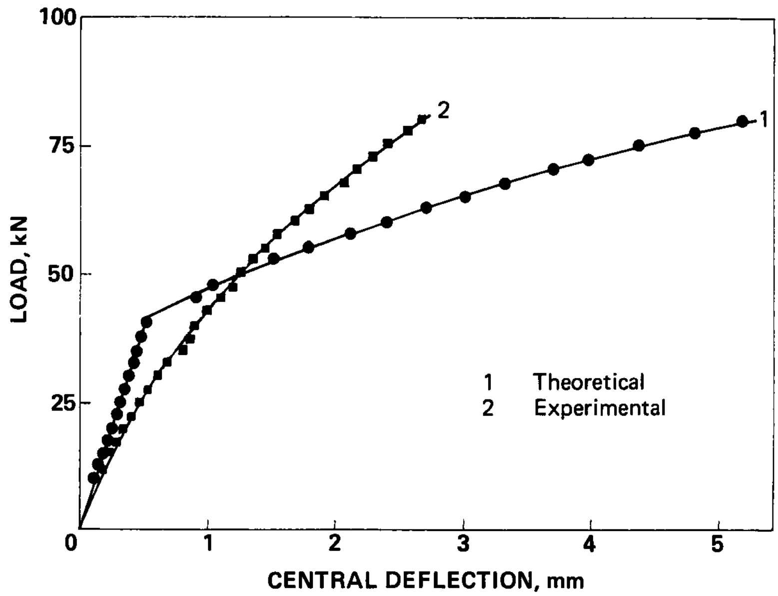

Figure 5 Load-Deflection Relationship for Slab 2

It was also observed during the tests that the deflections at identical places across the two centroidal axes of the slabs tested were more or less equal, thus confirming that the slabs deflected as a two-way plate.

The deflections measured at the centre of the slabs at different loading stages were compared with theoretical values computed in accordance with the procedure laid down in A.C.I. 318-77 (3) after taking into account the effective moment of inertia. A comparison of the observed and theoretical deflections at the centre of the slabs at various loads is given in Table 3.

The theoretical deflections after the initial cracking load were found to be more than the observed values, whereas they exhibited a reverse trend below this load. In any experimental investigation, the theoretical deflections can normally be expected to be more than the observed values. Hence, the post-cracking behaviour of the slabs can be considered as satisfactory and normal. The observed values of deflections before cracking were greater because of the formation of hair-line cracks at the joints of the precast units. However, this is not of much significance in view of the stiffening of the slab during the post-cracking stage due to the contribution of the in-situ ribs.

Table 3 Comparison of Observed and Calculated Deflections

LOAD kN	DEFLECTIONS AT CENTRE, mm			
	Slab 2	Slab 3	Slab 4	Slab 5
10	0.13 (0.13)	0.09 (0.09)	0.24 (0.25)	0.16 (0.16)
15	0.24 (0.19)	0.16 (0.14)	0.35 (0.38)	0.25 (0.24)
20	0.35 (0.25)	0.22 (0.19)	0.48 (0.50)	0.34 (0.32)
25	0.49 (0.31)	0.29 (0.24)	0.61 (0.63)	0.44 (0.40)
30	0.62 (0.38)	0.36 (0.29)	0.76 (0.75)	0.55 (0.48)
35	0.81 (0.44)	0.43 (0.33)	0.91 (0.88)	0.65 (0.68)
40	0.94 (0.50)	0.52 (0.38)	1.07 (1.00)	0.78 (1.05)
50	1.29 (1.28)	0.68 (0.47)	1.38 (1.26)	1.08 (2.05)
60	1.67 (2.35)	0.83 (0.57)	1.88 (1.51)	1.49 (3.28)
70	2.16 (3.73)	1.00 (0.66)	2.93 (2.58)	1.94 (4.57)

Note: Central deflections for Slab 1 were not measured.
( ) Values obtained from theoretical computations.

Experimental Construction

With a view to studying the long-term behaviour of the precast two-way spanning flooring system reported in this paper, a demonstration low-cost house having a plinth area of about 27 m^2 was built on an exhibition site in Madras during January 1977, Figure 6.

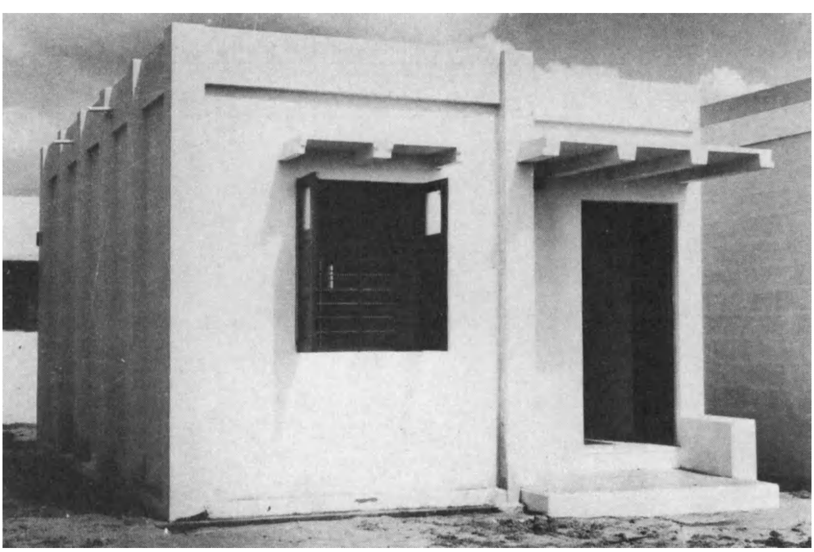

Figure 6 Low Cost House Built Using Scheme 1

Scheme 1, using precast concrete battens and trough units, was used in the construction of the roof of this house. Observations made so far have indicated that the roof has behaved satisfactorily and has not given rise to any cause for concern. The observations will be continued for a few more years before a final recommendation regarding the suitability of the scheme to large-scale construction is made.

CONCLUSION

An attempt has been made to develop a two-way spanning flooring system, using precast components, which eliminates the need to use conventional centering for its construction. Preliminary tests carried out on two sets of slabs incorporating two different schemes developed by the SERC have shown that the new technique helps to achieve two-way plate action almost up to the failure load. However, the formation of longitudinal hair-line cracks at the joints along the length of the precast units, even though not harmful from considerations of strength and serviceability, needs to be further investigated so that their occurrence is either eliminated or delayed by suitable means. Work is in progress on the development of a modified system of constructing a two-way spanning flooring system using small and medium-sized precast components.

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The paper is published with the permission of the Director, Structural Engineering Research Centre, Madras, India.

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DISCUSSION

Marion Chatterton. I was very interested in the presentation by Mr. Walsh about houses on heaving soils. We have experienced similar conditions in my practice in Rhodesia. I have some experience in this problem and if I may I would like to say a few words about it. We have experimented with many systems and I would like just to tell you about the ones which we found the most effective and not very expensive. We had similar conditions, extremely dry and extremely wet seasons, fluctuations in the soil water content and excessive heaving of the cotton soils and 'vlei' soils of southern Africa, which heave possibly more than does normal clay. We eventually came up with a technique for domestic construction with which we have now constructed up to 120 houses in three different areas, all on heaving soils. These buildings have been up now for eight to ten years, having passed through 16 to 20 cycles of dry and wet seasons. So far there has been absolutely no trouble. There were two solutions we used, the cheaper and the more expensive one.

In the cheaper solution, see Figure 1, we had all the walls supported on ground beams which were only about 350 mm deep, since they were designed for composite action between the concrete and the brick walls. Under these beams we had wax strengthened cardboard boxes, thus creating a void between beam soffits and soil. In these soils you can excavate neat to widths of about 250 mm, thus no formwork was needed for the beams. The cardboard voids were placed in the excavated trenches and the beams cast above them with very light reinforcement because of the composite action. The heaving soil was excavated out within the area of the house to the same depth as the bottom of the void and this area was then back filled with roughly 600 to 700 mm of well compacted stable fill. All the slabs were naturally completely divorced from the walls. Around the houses we put impervious pavings of about 1 m width, thereby more-or-less maintaining stable moisture conditions under the building itself. There were naturally very small fluctuations in dampness and because of that both our skirting-to-floor connection and the doors allowed for roughly 5 mm up or down movement of the slabs. The stub columns and bases supporting the beams, see Figure 1, were at about 3½ m centres, depending on the dimensions of the house, and they went down to roughly 1.6 to 2 m depth, where either better soil was found or the conditions of moisture were virtually stable.

A slightly more elaborate and more expensive system is shown in Figure 2, where instead of having the back fill supporting the floor slabs we had precast concrete planks suspended between the lines of walls with a void between the planks and the soil. The extra cost in 1968 to 1970 on a house of 1300 ft^2 (about 150 m^2) was compared with a house on conventional footings. In the first solution the

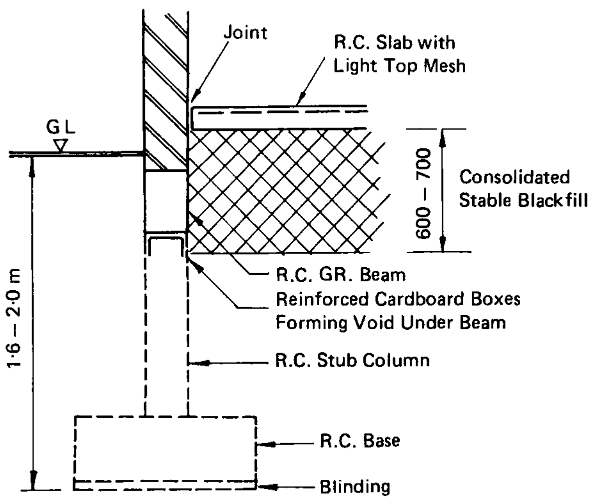

Figure 1 Slabs on Heaving Soils - Solution 1

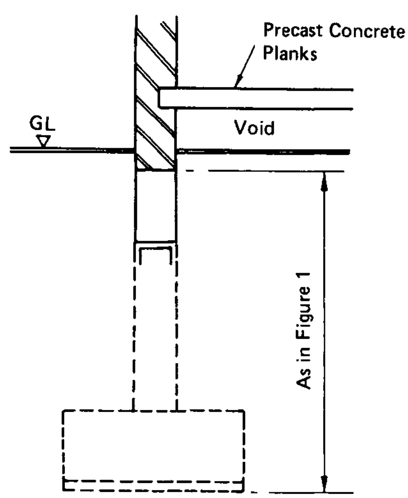

Figure 2 Slabs on Heaving Soils - Solution 2

difference per house was between £55 and £65 and in the second solution it was between £70 and £85, depending on the pattern of internal walls.

We had tried a few cases of raft foundations and had not very fortunate experience with them. We found that the load from the single storey building was too small, the movements of the soil were often severe and uneven because of deficiencies of drainage. Moisture fluctuations often affected one side of the building more than the other, and some of these rafts 'broke their back', i.e. developed a crack along the centre line.

Hans Gesund. I have some questions for Mr. Deacon. I may have been mistaken but it seemed to me that you did not use keyways between slabs and I was curious to know why not? Secondly, I notice you did use some reinforcement in the slabs, which I heartily applaud but which the Portland Cement Association of the United States condemns; they do not believe in reinforcing slabs on grade. I would like your reaction on both of these, please.

R. Colin Deacon. The first point on keyways: I utterly abhor keyways because I think they lead to a great risk of the two ears of the key being under-compacted which probably could be a cause of curling and possibly failure at the joint. We recommend a plain butt joint, in fact, tied together for load transference with a simple tie bar. This seems to work fairly well. Traditionally we used to have a square mesh that ran through the longitudinal joint; it required a split form, it was a messy construction detail and so we tried to simplify this. The only slight doubt in my mind is that occasionally we get a little bit of opening at some of the longitudinal joints, we may have a little less bond with the tie bar arrangement than the equivalent mesh steel that used to go through. As far as reinforcement is concerned, tests on highway pavings have shown that with a slab above about 170 mm in thickness, the performance is virtually identical whether it is unreinforced or reinforced. However, I must confess that I like to have a bit of reinforcement in it for safety. What we have done is to suggest that if you have the slab unreinforced, clearly you have got to have very closely spaced control joints. By putting reinforcement in, purely by observation we have suggested an upper limit of about 10 m between control joints. I should distinguish between a control joint, which is essentially tied with reinforcement, and a contraction joint, which is completely debonded. We have suggested a method of apportioning reinforcement based upon the thickness of the slab and the distance between debonded contraction joints; the thicker the slab and the further away you are from free contraction joints, the heavier the amount of steel you want in the longitudinal direction.

Hans Gesund. You have not found any problem of unequal settlement when running heavy fork-lift trucks across these longitudinal joints?

R. Colin Deacon. I have not personally found a lot of problem. We have a long stop by suggesting that if in fact you are going to have very heavy loadings then we think you should put dowel bars instead of the lighter tie bars to give full load transference. These situations are not common generally, it is what I would class as a special case that you should consider individually.

Robin T. Whittle. I would like to ask Mr. Deacon one simple question relating to Table 3 of his paper. He gives a table of mesh sizes for different thicknesses of slab. We have developed a reinforcement detailing manual and the question arose, is it really necessary to have all this variation in mesh sizes for different thicknesses of slab? Could I ask what he would favour if he was going to suggest an all purpose general mesh?

R. Colin Deacon. Well I think I gave the answer to your question in my answer to the previous one. You see, it is based upon the notion of the tension generated by restrained contraction of the slab; this is the only concept that we are taking into account, long-term drying shrinkage in effect. By observation we find that the early temperature effects which are so critical in an external pavement do not, in general, affect slabs which are constructed within the envelope of a building. I know of a number of cases, and I say this behind my guarded hand, where sawn contraction joints have not been put in for four weeks after the slab has been placed. If this was an external slab, within 12 to 24 hours you would get cracking. So we feel that temperature effects are unimportant either due to ambient conditions or heat of hydration effects in the normal sort of thicknesses that we encounter in industrial floors. However, long-term drying shrinkage is a factor to be considered and the concept is that the whole tied length will contract and will be restrained to some extent by sub-base friction. Now we have adopted, I say 'we', I should say I have, since nobody has done any work to prove me right or wrong although I think a number of other authorities, notably O'Brien in Australia, follow this concept too, a triangular lineal distribution of this frictional force from zero at the end to maximum at the mid point of the tied length of slab. Therefore your reinforcement is directly proportional to the length of the tied slab together with the thickness of the slab. Thus as your tied length goes up, as your slab thickness increases, so you must increase your mesh weight. If you do not do this then I think you are going to increase the risk of additional cracking.

Paul Bennison. As a practising engineer, I find it difficult to understand why structural engineers in the U.K. do not consider utilising some of the benefits of flowing concrete at the design stage. Everybody seems to wait until their project is on site before they consider using flowing concrete. I think there are advantages to the designer and I would like to see structural engineers utilise these benefits at the design stage. I wonder if Mr. Sweetland would like to comment on that?

David B. Sweetland. Currently flowing concrete tends to be used for the 'fire engine' jobs. I would prefer to see it planned into the project beforehand, which means that the optimum design can be decided upon and the opportunities for flowing concrete can be fully utilized.

William P. Liljestrom. A question for Mr. Deacon, concerning tie bars extending into the new slab: do you ever grease those bars, so that they might not form a bond, and so let the new slab slide as the load is transferred across the joint?

R. Colin Deacon. The answer is yes and no. Bearing in mind that we are controlling contraction in the length of the slab or the length of the strip, we want to put the major amount of steel in that direction and so for economy we have suggested in our tables, and in Table 3 of the paper, the use of standard meshes which are what we call long mesh, in other words the bars are closely spaced in the longitudinal direction with a limited number of transverse wires, so that in the transverse direction the slab is virtually unreinforced. Now we have a road design bible, which is called Road Note 29 issued by the Department of Environment, and this requires that for unreinforced slabs, every fourth joint should be a full contraction joint. For simplicity I have followed this concept and have suggested that when you are using long meshes, at every fourth longitudinal joint the tie bars should be debonded to allow that joint to act as a contraction joint. For all other joints the tie bars are not debonded. Now there is one alternative because clearly a slab does not know which way it is going, it is an inanimate object which does not know how it has been designed and will act just as it wishes. Clearly if it contracts in one direction it contracts in the other and if you construct it with undebonded tie bars at joints then you are going to generate a large number of adjacent strips all tied together. You must therefore look at the frictional effects in the transverse direction as well and as an alternative, if you want to you can put the same amount of steel in the transverse direction and use a square mesh, in which case there is no need to debond the tie bars.

Paul Poitevin. In Mr. Deacon's paper there is an example of slab reinforcement, the bars being 300 mm on centre. Such wide meshes are rather unusual. With such distances between bars, one is tempted to use ordinary deformed bars instead of welded fabric. So, do you think that only the section of steel reinforcement is important, or that welded fabrics are mandatory in slabs?

R. Colin Deacon. I think I follow you. In principle it is only the amount of steel which has to be equated to the tension due to the friction under the slab. Now for convenience, and for economy in most cases, we in the U.K. have standard welded meshes and so the Table in the paper relates to our standard meshes. In principle, however, you could put in individual bars if you so wished.

Christopher Coward. I have a question for Mr. Deacon on ground slab construction. We had from Mr. Liljestrom an insight into current American practice, using shrinkage-compensated cement which enables joints to be eliminated, whereas British practice, as outlined by Mr. Deacon in his paper on the subject, seems to be going the other way, putting in more and more joints in line, as he said, with road practice, as in Road Note 29, etc. One of the problems with these frequent joints is curling of the slabs, and surely the more joints that are put in the more of a problem curling is going to become, although it can be overcome to a certain extent by the use of permeable membranes as opposed to impermeable ones, a technique we have tried with a certain amount of success. The similarities that both Mr. Liljestrom and Mr. Deacon have suggested are the need to construct slabs under cover inside the buildings of which they are to be the floor. Now there are very often programme advantages in constructing slabs before erecting the building structure. What I would like to ask Mr. Deacon is why do we only consider concrete floor slabs, why do we terminate the road at the edge of the building, why do we not bring the road into the building and concentrate on flexible floor slabs as opposed to rigid floor slabs? I admit that we can use concrete as a base, maybe a lean rolled concrete base or a structural concrete base, but flexible slabs would seem to have a lot of advantages and perhaps this type of construction could be used for certain areas, say in aisles or areas where there is a lot of traffic. I would like Mr. Deacon's comments on this type of approach.

R. Colin Deacon. There is a great deal in this question which I shall try to cover point by point. I do not think our current systems utilise any more joints than the old. I would suggest that in fact they perhaps have reduced the number of joints. What I would say is that in the U.K. we seem to be very touchy on the subject of cracks. When I went to Denmark during my initial investigations into floors, I went to a big glass works that had been laid in long strips, about 80 m long, with paving machines. When I saw it, when the works was in operation, there were cracks in the floor opposite all the columns. I was walking round, not with an engineer but with the production director, and when I pointed out a crack in the floor he was totally unconcerned. The attitude on the Continent is that cracks are a fact of life. They will accept cracks and they do not worry about them unless the cracks really develop and cause some problems. I know of other cases where we have designed in this country for Continental clients who have come over here and expressed surprise at the number of joints we have used. Really if we accepted more cracks without getting upset about them, we would probably have less trouble with joints. The recommendations that we have given in our publications have in fact suggested that under appropriate circumstances you can choose either to saw the joints or to accept the odd crack. If we could do without having to make joints and get rid of the risk of cracking as well, as Mr. Liljestrom has indicated, this I think would be a marvellous step forward and I suggested in my address that we ought to be looking at continuous reinforcement as a possibility. If you lay your floor in the open like a road, you must design it as a road because it is going to be subjected, or is likely to be subjected, to early thermal problems and you must take these into account. You are going to have frequent fully debonded dowelled joints, not the simple cheaper tied ones that we have indicated can be used within certain limits. You can use a dry lean base with a bituminous topping, but whether it will fully satisfy all your requirements has to be evaluated in each case.

Maurice Levitt. I have two questions, the first to Dr. Gebauer. You state in your paper that the reported study is solely on the drying shrinkage of concrete at early ages due to the effect of evaporation of water. Would you say that there are basically three effects on concrete due to evaporation or loss of water? One is the overall contraction of the concrete in any dimension, due to the fact that the excess water is coming out of the mix, which could not result in cracking. The other one is a plastic settlement cracking due to the concrete consolidating inside the formwork which, if the reinforcement affects it, could cause a form of plastic settlement cracking observed over the reinforcement. The third type of cracking is the type you have observed. The point I am putting to you is that we have observed cracking on our construction sites and I am sure other contractors have also, well before your magic 2 or 2½ hours and in humidities approaching 95 per cent but with no wind at all. I put it to you that under these conditions concrete which has a void system latent within it will lose this water quite quickly, even at high humidities without any wind at all.

If I can switch now to Mr. Sweetland: we have argued, from both the point of view of academics and the point of view of practicalities, about using things like admixtures and superplasticizers in concrete. However, one particular side of the argument which seems to have been suppressed here, which surprises me, is the politics of the situation. Now the politics of the situation to my mind are that in Britain and in France something like 5 to 15 per cent of concrete contains admixture while in Japan and the United States somewhere between 60 and 80 per cent of concrete contains admixtures. One asks oneself the big question, why? There are two reasons: one, traditionalism and two, trade unions. Now if you can adopt a different philosophy, that any time you are taking on a job where superplasticizers are going to be used instead of doing this job with six men you are going to do it with four men, can you please tell me which way you are going to vote at the next election?

Juraj Gebauer. The results which I presented were for the laboratory conditions that I described, using test specimens 50 mm thick. It is possible that before 2 hours cracks appear in very thin sections or under very extreme conditions.

David B. Sweetland. There is no doubt that we in the United Kingdom mistrust admixtures, and we certainly do not like a concrete which is fluid, for the want of a better phrase, because it tends to frighten us since it goes against traditional thinking. I believe that it will take a long time to overcome this conservatism. Your comment on trade unions is in some ways relevant.

On three occasions we proved to the main contractor that flowing concrete was a convenient and also a profitable way of laying slabs. In each case the slab work had been contracted out to a sub contractor at agreed rates. When the main contractor suggested that flowing concrete could speed up the work as well as reduce the required labour content, the sub contractor adjusted his rates which negated all the benefits. I think that unless we overcome this attitude, we shall continue to incorporate only 5 per cent of admixtures in concrete rather than 60 per cent as in other countries.

John D.N. Shaw. Surely in that case you are switching to a superplasticized concrete which was not specified, otherwise the rate for the job would cover the saving in labour anyway and the unions would not have got involved at all. We have to get it written into the specification and not change to superplasticizers at a later stage.

David B. Sweetland. I quite agree, specification is the one thing which will be the life blood of materials like superplasticizers and flowing concrete. If we do not get into specifications and we do not have the active participation of the major contractors as well, I think superplasticizers will die a death quite frankly, which will be rather a shame.

Peter C. Hewlett. Can I restore the balance just a little. I think that people might be thinking that superplasticized concrete is flowing concrete, that is only one half of the concept. It is also possible to take advantage of the water reduction capability this workability enhancement also gives and perhaps go to an intermediate stage where you can make a 20 per cent water reduction and make concrete more easily compactable, if not flowing concrete, at water-cement ratios not exceeding 0.4. I think then one would not run into some of the problems described by Dr. Kral.

John M. Rolfe. I have two questions on flowing concrete. First of all our experience in southern Africa has been limited but a little bit disquieting because there seem to be indications, and I emphasize that this is based on very limited experience, that the action of superplasticizers is considerably modified by the choice of aggregate and the type of cement used. I would like to inquire whether this accords with local practice and if so what guidance can be given. The second matter which concerns me is that although improved workability and mobility are obviously beneficial in placing, could we be given some indication of the possibility of segregation due to vibration compaction?

David B. Sweetland. Quite frankly I think that is due really to the type of aggregate, particularly the shape of aggregate and its grading. We have found that we need to use more admixture to produce flowing concrete in Scotland with whinstone than we need to with rounded aggregates out of the West Midlands. It is a fact of life. I think the only guidance I am able to give you is that you need to do test mixes first to get an idea of how much superplasticizer you need, or indeed whatever admixture you are using, because I do not believe that there is a universal dosage. It will vary from aggregate to aggregate source. On the second point, I think the major danger of segregation due to vibration is when the shutters are full. When the concrete is being poured into the shutters it is on the move and it is difficult to over-vibrate it because it is moving past the vibrator. When the shutters are full, however, that is the time when it can be over-vibrated and you can get segregation. It is a matter of common sense and getting used to the material so it is very difficult to put guidelines on this.

John M. Rolfe. Could I ask also for an answer to the question of the effect of different cements?

David B. Sweetland. That is a good point. We have very little experience of anything other than common or garden O.P.C. We have done some work with sulphate resisting cement and if I remember correctly the dosages required were slightly higher for that than we would have expected for O.P.C., but that is the only experience we have.

John M. Rolfe. I was referring to O.P.C. from different factories.

David B. Sweetland. We have not had any experience that we would consider suggested that it would be necessary to change dosages to cope with cement from different sources. I think you are more likely to have to change the dosage rate because of the aggregate gradings and shape.

John F. Dixon. Mr. Sweetland, I wonder whether you would elaborate on your almost throw-away remark in your presentation to the effect that we can get flowing concrete with other materials rather more cheaply? What are those materials and how much cheaper are they?

David B. Sweetland. I am sure that the admixture suppliers here could tell you far better than I. I was referring to superplasticizers which cost round about £1.50 to £2.00 per cubic metre of concrete. Now this is not the pure cost of the superplasticizer, since it includes the extra cost of having to re-design your mix to start with, because for the common or garden flowing concrete you tend to start off with a 75 mm slump; this means that to get the strength you require you need to add extra cement so that adds to the cost. There are other materials, such as modified lignosulphonates, which will give partially flowing concrete for something like 40 to 60 pence worth of admixture per cubic metre of concrete. (If anybody would like to make a comment on that figure from the admixture side, I would be very pleased.) So you can make a form of flowing concrete a lot more cheaply with other materials, but I do not think it is as fully flowing.

Barry P. Hughes. Could Dr. Gebauer give some indication of the crack widths and the spacing of the cracks that he obtained, either now or in writing subsequently?

Juraj Gebauer. In our investigation we recorded only cracks, not microcracks, which could be seen with the naked eye, i.e. 0.1 mm or more.

Paul Bennison. If I could answer Mr. Dixon's question about the use of materials which are not true superplasticizers to produce flowing concrete. I have recently been doing some work on the development of materials which are mixtures of a true superplasticizer and modified lignosulphonates. These will produce fully flowing concrete, as specified by DIN 1048, for about 50 pence per cubic metre. You do not get all the associated benefits of a true superplasticizer, there is no acceleration in gain of strength and no higher ultimate strength, but the workability is as specified in DIN 1048.

David B. Sweetland. May I make a point there, Mr. Chairman. I think that if 50 pence per cubic metre is the admixture cost, there is still the extra cost of redesigning the mixes for whatever extra cement you require to get the strength that you need. It would probably be necessary to add on perhaps another 40 pence per cubic metre to get the full cost.

Alan R. Selby. In his paper Dr. Gebauer states that water loss and shrinkage at very early ages can produce beneficial effects on the quality of hardened concrete without danger of cracking. Could I ask whether he restricts this comment solely to his laboratory specimens or whether he intends that practising engineers can develop this technique in trying to improve their slabs. I ask this because I feel that where you have a significant mass of concrete you will not achieve evaporation from the centre of the mass due to the slowness of migration of the water and I would suggest that the overall effects may possibly be deleterious.

Juraj Gebauer. You cannot recommend this measure in every case but certainly it was shown in our tests that the strength is increased if in the first hour you allow evaporation of water from the concrete, especially on the type of concrete specimen used in the investigation. However, it is not valid for all concretes.

Paul Poitevin. In your paper you seem to affirm that the kind of cement is immaterial in relation to cracking at early ages. I think it is because you consider principally shrinkage cracking - the drying of cement paste - but thermal cracking seems more important. In France we fear high early strength cements and the laboratories used to evaluate the crackability of cements on cement paste rings. This test is now discarded, but the heat development at 12 hours is limited to 50 cal/gm for certain applications. So I would be pleased to hear your comment on the choice of cements.

Juraj Gebauer. The type of cement does not have an important effect on the early cracking. I did not mention it in my presentation through lack of time, but compared to other effects, such as wind velocity or humidity and temperature, the type of cement has only a small effect. However, I was talking only about shrinkage and cracking of concrete at very early ages due to evaporation. The cracking due to heat evolution appears later.

R. Colin Deacon. I would like to say how fascinating I found Mr. Rolfe's paper and I am sure we could discuss many aspects of it very profitably. I am sure that many of us here are very envious that he is able to get 50 N/mm^2 concrete with only 300 kg/m^3 of cement, I wish we could. I would just like to question him on two points, if I may. He said that he had trouble with sawn joints, perhaps he would elaborate on that, what form this problem gave him? The other point: he gave a detail on Figure 1 of expansion joints using a ground beam. I have always felt that this form of support to slabs is fraught with some difficulty. If you get settlement of the sub-base adjacent to the ground beam, there is always the risk that the slab itself would break its back; presumably he is able to give sufficient control on site to ensure full compaction so that this does not happen.

John M. Rolfe. Thank you for your kind words, Mr. Deacon. The first question concerns sawn joints. Lack of time prevented amplification. The sawn joint is normally about 25 per cent of the slab thickness. I try to get the reinforcement into the top of the slab and I have found the sawn joint very valuable in verifying that the reinforcement is in fact there after pouring concrete. If we have the Clerk of Works standing there watching the saw, when the sparks come up he stops it and measures the depth, and we did find this a very useful way of leaning on the contractor. There is no suggestion that the sawn joint does not, in fact, achieve its main purpose. What we have found is a point where wear was initiated because when the joint is sawn it is filled with a polyurethene filler, or similar plastic type filler, but shrinkage continues thereafter and shrinkage leads to a concavity of the joint. This leaves the sawn edges slightly exposed, traffic breaks them down, the adhesion at the top of the joint is impaired and we found that wear developed fairly rapidly from sawn joints whereas the other type of joint I showed, where we have a soft metal strip cast in, tends to provide support to the concrete more effectively than plastic filler. We have tried using very stiff formulations of plastic. What happens with those is that they adhere to one side and tear away on the other, which is worse. The question concerning the detail on Figure 1 - the sleeper slabs I am not terribly keen on. I prefer the dowel joint but I have used the sleeper slabs on certain occasions where the circumstances of the job suggested that it would be easier for staff management, causing less problems than putting in dowels, which I have always found are rather awkward to support and keep in place during concreting.

Frank R. Benson. During the course of his paper Mr. Mays asked why post-tensioned slabs have not been used so much in the United Kingdom. I think I can at least give one reason for that. The system was developed in the United States, as he says, in conjunction with the lift slab method of construction and the two methods have developed together very successfully there. In this country, although we do a lot of reinforced concrete lift slabs, we have done hardly any post-tensioned lift slab buildings. I think we can probably get a clue, but only a clue, from Figure 2 of Mr. Mays' paper where he illustrates the effect of column size, in fact this is due to B.E. Clark, I understand. You can see a large difference between the A.C.I. and C.E.B. codes and the Concrete Society's committee on post-tensioned slabs, labeled there as C.P.110. The difference between the two for the larger column sizes, which I suggest are in fact practical column sizes in multi-storey flat slab construction, is very considerable. There are, however, many other factors involved and I think it would be possible to show that C.P.110, or at least the Concrete Society's recommendations based on C.P.110, are very unduly conservative. Now this was touched on by Robin Whittle in his keynote address this morning and he did state during the course of his talk that C.P.114 or C.P.110 were deemed to satisfy in this country, but unfortunately that is not the case. C.P.114 is no longer deemed to satisfy so far as flat slab construction is concerned, which has been deleted from this code and so designers are in fact obliged to work to C.P.110. The main problem is the question of shear and I think again Mr. Whittle pointed out the problem which shear has become in flat slabs over the last few years since C.P.110 was introduced. In fact I think it can be shown that designs to C.P. 114, which has been in existence for several years, have load factors of less than one when viewed in the light of C.P.110 and yet Mr. Whittle says in his paper that there has been no evidence of any structural failures, or excessive cracking or anything of that sort. Mr. Whittle also says in his paper "we are now faced with a dilemma, we could persuade engineers to scrap the present simplified methods and in doing so we would, by implication, pronounce as unsafe innumerable existing, apparently satisfactory, structures." Well that, gentlemen, is what I believe the current British Standard Code of Practice has in fact done. Now Mr. Whittle offers a considerable amount of hope in the final concluding passage of his keynote speech. I was going to ask Mr. Mays whether in fact his studies bear out what I have been saying, but if Mr. Whittle is still here I would like to know whether those optimistic conclusions, that we do need more information and so on, can be realised and what the Code of Practice Committee intend to do about this in the way of amendment? How soon can we expect more realistic approaches to shear in post-tensioned flat slabs, because until that is done I do not see very much hope of seeing many post-tensioned slabs in this country, yet it is an elegant method of design, it produces one of the best types of functional buildings that one can have and I think it is a very great shame that we have not seen more of it.

Geoffrey C. Mays. I have read with interest the discussion which ensued from my paper on prestressed flat slabs, in particular Mr. Benson's comments on shear design. I should like to take the opportunity of comparing the difference in the American and British approaches by means of an example. I have chosen the example of a flat plate as given in the publication 'The Design of Post-Tensioned Slabs' by the Post Tensioning Institute. The transverse prestress is designed for the bay shown in Figure 3 having a 165 mm thick slab of Grade 35 concrete.

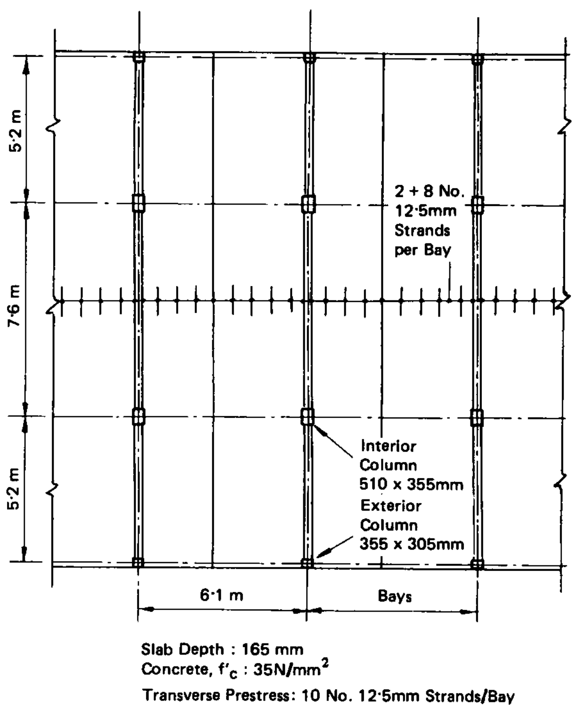

Figure 3 Flat Plate Apartment (from ref. 39)

Considering the shear design of a typical interior column then, by the American Regulations, shear design for 2 way action governs. At the critical section, $^d/_2$ from the column face, the total shear stress to be carried by the slab is 1.9 N/mm^2. The shear necessary to cause diagonal web cracking, v_{cw}, is well in excess of this when the vertical component of the tendon force crossing the critical section is taken into account. The value of v_{cw} is independent of the tendon distribution in either direction since f_{cp} is constant.

If the shear calculations for the same column are carried out using the U.K. method, and for uniformly distributed tendons in both directions, then the case of limiting the shear stress to v_c governs. At the critical section, 1.5 h from the column face, the direct shear stress is 0.89 N/mm^2 using partial safety factors for loads as given in C.P.110. The value of v_c, modified by the slab depth factor, based on an equivalent area of untensioned steel is 0.79 N/mm^2. The shear resistance is therefore inadequate and as shear reinforcement is not permitted in slabs less than 200 mm thick the slab thickness must be increased to approximately 200 mm before it is adequate. Furthermore, the design shear force in these calculations has not been increased to allow for non-symmetrical distribution of shear round the column as Clause 3.6.2 of C.P.110 requires for flat reinforced slabs. The situation may, however, be improved by banding over the columns, say, the tendons in the longitudinal direction thereby increasing the effective area of steel, A_s, and hence v_c.

This example, I hope, gives some idea of the magnitude of the difference between the two approaches, but I would reiterate the sentence from the conclusions in my paper: 'The Concrete Society recommendations are admittedly more conservative than the American, but are based on an understandable caution to avoid any possibility of sudden shear failures'.

Robin T. Whittle. I cannot add an awful lot at this stage in answer to Mr. Benson. For post-tensioned flat slabs the reasoning that seems to be going on in the Code Committee at the moment is to change the Code sufficiently, for instance, to allow lower strength prestressed concrete and, in obvious places, to ensure that it does not actually stop the use of post-tensioned flat slabs. However, at the last full meeting of the Code Committee, it was, I think, decided, or at least intimated, that at the moment reference will still be made to documents outside the code for the design of post-tensioned flat slabs. Coming back to the shear problem again, this is still very much in debate for ordinary reinforced concrete flat slabs. I have little to add to what I have already said, that there is a chance of changing the load pattern in the structure for slabs and there might be a chance of reducing the moment transfer in design for flat slabs.

John D. Peacock. I feel a bit inhibited talking about flat slabs because we are in the other business; we like to have a few beams around. I know that there are C.C.L. people here so I will say that my experience with post-tensioning was a long time ago, but I thought it might be worth mentioning that it always seemed to me that at the start of the job something went wrong: the jacks would not jack or the anchors would not anchor or the debonding did not debond or the cables would not stretch. Maybe life is easier now that people have had more practice and the equipment has become more reliable, but this kind of early experience may be contributory towards the slow acceptance of post-tensioning.

John M. Rolfe. Not on the subject of post-tensioned slabs, but on the subject of code anomalies, we had an event in Rhodesia recently, which has given no little delight to consulting engineers, where the municipal checking authorities decided that they would design their own seven storey building instead of putting it out. After a short period of occupation it was declared unfit and unsafe for human habitation, hastily evacuated and is still standing empty. The cause of concern was excessive deflection accompanied by fairly extensive cracking and the consulting engineers who were called in at this stage to carry out checks quite rightly initiated the checks on limit state principles. The building had been designed according to C.P.114 at about the time C.P.110 first came out and so it is no criticism of the designers that they did not use C.P.110; when they started work in 1972 it was not available. When a limit state design was carried out on this building it was found that practically every beam in the building had a factor of safety of less than one against shear, simply because of the variation in the shear requirements from C.P.114 to C.P.110. It really leaves one wondering whether every building designed to C.P.114 must now be considered as unsafe in shear.

Leslie A. Clark. If I could just comment on the comments that Mr. Peacock made about my comment yesterday. I would first like to say that I was not on the committee to which I referred, that is why I said I had a peripheral involvement. Secondly, I feel that it is important that there are people who are actually designing on code committees, the reason I feel this is that it is that sort of person who understands the interaction between clauses. If there were a lot of people on code committees who are actually designing, I feel that some of the anomalies that do get into Codes of Practice would not arise because they would be picked up very early on when the interaction of clauses was being looked at. The final point I would like to make is that the committee to which I refer did not have a single academic on it. It did have two people who were research workers with industrial concerns, but there were no academics at all.

John D. Peacock. Well, of course, I agree, we ought to have working designers on code committees but there is a problem: they are too valuable in the design office to let them go off to committee meetings. When we do convince ourselves that it is in our interest for this to happen, then it does mean that they have to devote a lot of time to it and many of them are just not suited for this type of work, they find it terribly tedious. I do not know what the answer is, but it is a real difficulty and we ought to try and do something about it. Dr. Clark must have been a bit more fortunate with the committees he has served on than I have, because I have always felt that there was a fair share of academics there.

Hans Gesund. I have a solution to the problem and that would be that since the various codes of practice become government regulation there is really no good reason why the government should not pay the people serving on the committees or reimburse their organisations for their salaries. If that were done I think you would have very little difficulty getting organisations to send some of their more experienced people into committee work and you might thereby ultimately save the country a great deal of money that is otherwise spent on unprofitable or unworkable regulations. This is not to mention the fact that most academics are salaried personnel from the government, so if instead of using some of their time on committee work, it were used in research or in teaching and the design firms replacing them on Committees were reimbursed for their costs, the government would still not be out of pocket. So I think this is something that perhaps needs to be encouraged. We have similar problems in the United States. I am on a number of American Society of Civil Engineers and American Concrete Institute committees and all these committees are full of academics or industry representatives and I mean thereby contractors or manufacturers' representatives, with the manufacturers' type of interest in some of these things, and there are very few practising engineers on the committee simply because they cannot afford to be; if the government would reimburse the firms I think the problem would be solved.

John D. Peacock. What a good idea. I only hope that we can take the transcript of your comment up to the B.S.I. and perhaps they will do something about it.

Walter Thorpe. I would like to pick up the point made by Mr. Gesund because this is something that quite a few of us are very well aware of. Codes of Practice and Standards are I think becoming more international and will continue to do so. We have at present got the big move towards the C.E.B.-F.I.P. model code of practice for concrete structures; I am told that there are other moves afoot at even higher levels in Brussels and we are all going to have to live with European design rules before very long. I think that when this happens a lot of contractors are going to jump up and shout and complain bitterly and in many cases I think they will only have themselves to blame. I do not think that in the U.K. (and I am sure it is equally true internationally) the Federation of Civil Engineering Contractors as a body pay sufficient attention to this. I think also that the Government should give much more support since the introduction of more international unified codes based on limit state methods is something that will happen. If I may make another quick point relative to flat slab design methods. There are two new design documents for flat slabs coming out, there is an international one from F.I.P. and there is a Concrete Society one. The international one relates to the model code, the Concrete Society one is based on C.P.110. I think some of the numbers from those documents will look a little better than the ones used in Mr. Mays' paper, which were related to the old Concrete Society documents, and I think we have got to a situation where we seem to be getting economical solutions out of the design. I am hoping that in the future things will become even better when this currently difficult problem of moment transfer to the columns has been cleared up.

John D. Peacock. And that C.E.B. code is going to be a lot heavier than C.P.110. I would like to raise a point on Mr. McDonald's contribution because we have been in the business of building hotels and high rise blocks of flats, although I am afraid those days have gone. It did seem to me that with those boxes you hardly needed in-situ columns. Could you perhaps comment on how you decided that you did need them?

A.D. McDonald. One of the reasons was to allow freedom at the first floor level, where commercial space rents at a much greater rate than the hotel rooms above. By using the columns, having the boxes spanning from column to column, you do not have to have a bearing wall penetrating down. It also serves the purpose of assisting in developing a fire rating, by creating a plenum space between boxes horizontally as well as vertically. Finally, we considered that the cast-in-place joints would help to eliminate some of the difficulties normally experienced with putting pre-cast pieces together.

Session 5

Surface Finishes

Chairman: R. Malcolm W. Horner

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Keynote Speaker: Roger D. Browne

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Authors

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D. Feldmann
M. E. Hodgson
R. G. Chaplin
E. S. Fairweather
M. J. Wallwork
C. Sankey
C. G. Price
S. G. Fearon-Wilson

Discussion

A. R. Cusens
R. D. Browne
J. D. N. Shaw
R. D. Browne
P. C. Hewlett
C. Strain
M. E. Hodgson
G. Barnbrook
D. Feldmann
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M. Levitt
S. G. Fearon-Wilson
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R. C. Deacon
C. Coward
E. S. Fairweather
C. Sankey

SURFACE FINISHES

Roger D. Browne

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INTRODUCTION

In our laboratories over many years we have had to deal with the recommendation of finishes for building and civil engineering structures. Further, we have also had the misfortune to deal with problems that arise in construction and in service, both immediately before acceptance by the client and after much longer terms of operation of the structures where failures have occurred.

Concrete floors, especially for industrial use, have for many years provided a significant proportion of our building defect problems. These are comparatively rarely due to settlement or other major structural causes. More commonly the problem is due to the performance of the surface finish, sometimes in association with weakness in the upper part of the concrete slab forming the substrate.

My contribution to this important session is concerned simply with trying to identify certain underlying problems in the subject and suggest possible global solutions.

COMMON DEFECTS

Among common defects the following appear prominently:

1. Crazing
2. Dusting
3. Abrasive wear
4. Surface break-up
5. Loss of adhesion to substrate
6. Polishing (i.e. loss of non-skid properties)
7. Chemical attack.

Of these, probably only crazing can be unequivocally put down to bad workmanship, i.e. ineffective curing. The remainder may be caused by either bad design, an inadequate specification or poor workmanship, or a combination of two, or all three of these factors.

CAUSES OF DEFECTS

The origins of many defects are to be found among the following weaknesses:

a) Inadequate specifications
b) End use of floor not known to specifier
c) Conflicting requirements
d) Contractual arrangements
e) Inadequate mixing
f) Inadequate long-term performance data.

Inadequate Specifications

A major problem in specification and application is the profusion of methods of finishing, types of materials and the glowing claims each supplier provides for his particular product, all contributing to the random and often inappropriate decisions made in material selection, particularly in relation to cost for the performance required. In the past specifiers have often relied on skilled specialist contractors to produce a satisfactory finish within the wide bounds of a sketchy specification. In the present climate of fierce competition among flooring contractors this is no longer good enough. Every floor finish must be specified in detail to achieve the performance needed, bearing in mind that contractors cannot afford to provide a better finish than the specification requires; to do so could be to lose the contract.

There is at present no generally recognised grading system for floor finishes; this normally means providing a specification detailing the method of substrate preparation, the materials and methods of mixing, application and curing. If a fully-bonded screed is required, the specification must say so, but the specifier must realise that this will require scabbling of the slab and be prepared for the additional cost. For example, on one site the finishing contract included provision of a sand-cement screed to a floor slab constructed under the main contract. The specification only required a 4 : 1 sand-cement screed and said nothing about substrate preparation, water-cement ratio or admixtures. After the screed had been laid the architect said that he now required removal and replacement of all hollow-sounding areas to provide a fully bonded screed. This would have required thorough scabbling of the matured slab.

End Use of Floor not Known to Specifier

In his interesting review of special floor finishes to be presented in this Session, Shaw propounds the necessity, in selecting a finish, to consider the precise service conditions to which the floor will be subjected. Unfortunately in many cases this is not possible. For example, many property companies develop industrial estates where factories are built for letting. Unless they are let before completion it is usual to provide a standard concrete floor with an inexpensive general purpose finish, such as power-floated concrete. If such a factory is subsequently let to a company engaged in dust-sensitive work, such as electronics or printing, complaints will result.

Conflicting Requirements

Some floor performance requirements are in direct conflict with other desirable properties. For example, skid resistance against ease of cleaning, appearance against durability, and floor finish cost against cost of lost production. The last-mentioned point is of particular importance in these days of high capital investment in production plant. In a production situation it is very desirable to choose a finish which is both durable and capable of being repaired within the limits of normal maintenance time, e.g. at a week-end or during the annual holidays. To be cheaper may prove to be expensive.

Contractual Arrangements

Most applied finishes are now laid by specialist sub-contractors. The sub-contract conditions should make it clear that, as specialists, their application of the floor finish signifies their approval of the condition and preparation of the substrate and their complete responsibility for the subsequent performance of the floor (other than for latent structural weaknesses). This of course implies that the required performance should be made clear in the specification. A better system is the supply-and-lay approach, where the responsibility for any failure can be more easily resolved.

Guarantee of finishes is still a major stumbling block, possibly because of the consequences of misuse and unsuitable maintenance. Whilst accepting that the end use may be ill specified, the need remains for some assurance of a modest performance life. The building fabric is expected to last for 20 to 50 years. For how long should the floor finish survive? There is a need to establish and define the potential life of finishes appropriate to the category of use. Life requirements may be as little as 5 years where frequent rearrangement of plant is expected, but must be properly assessed.

Inadequate Mixing

For cement based flooring systems, the performance of the small tilting-drum type mixer, traditionally used by floor-layers, is quite inadequate to produce the standard of mixing required for modern high-performance mixes. It appears that at present the only alternative is the use of ready-mixed material produced in high speed pan mixers of the Cumflow type by some ready-mixed depots. There appears to be a need for a more efficient small portable mixer.

Inadequate Long Term Performance Data

Frequently mechanical and chemical testing of finishes is carried out within a relatively short time-scale. The effects on the material properties (e.g. impact strength) of ageing, thermal cycling of surfaces, and long-term exposure to chemicals, traffic etc. are inadequately monitored.

Long-term tests carried out by us (1) on the bond of epoxy and polyester flooring to a concrete surface, showed the initially high bond strengths (1.5 to 2.5 N/mm^2) dropped within the first year and after 10 years eventually settled to 1 N/mm^2, adequate for the application. No similar performance data have been given in the papers for this Session and many other similar conferences.

CLASSIFICATION AND TESTING OF FLOORS

Many of the conditions which lead to floor finish problems could be avoided if a realistic grading system for floors (including floor finishes) could be produced and found generally acceptable by specifiers and the industry. Such a classification would necessarily be linked to standard empirical test methods. Each grade

Table 1 A.C.I. Floor Classifications

CLASS	USUAL TRAFFIC	USE	SPECIAL CONSIDERATION	CONCRETE FINISHING TECHNIQUE
1	Light Foot	Residential or tile	Grade for drainage make plane for tile	Medium steel trowel
2	Foot	Offices, Churches, Schools, Hospitals	Nonslip aggregate mix in surface	Steel trowel, special finish for nonslip
		Ornamental residential	Colour shake, special	Steel trowel, colour, exposed aggregate, wash if aggregate is to be exposed
3	Light Foot and pneumatic wheels	Drives, garage floors and sidewalks for residences	Crown; pitch joints; entrainment	Float, trowel and broom
4	Foot and pneumatic wheels*	Light industrial commercial	Careful curing	Hard steel trowel and brush for nonslip
5	Foot and wheels abrasive wear*	Single-course industrial, integral topping	Careful curing	Special hard aggregate, float and trowel
6	Foot and steel-tyre vehicles: severe abrasion	Bonded two-course heavy industrial	Base: Textured surface and bond	Surface leveled by screeding
			Topping: Special aggregate, and/or mineral or metallic surface treatment	Special power floats with repeated steel trowelling.
7	Classes 3,4,5,6	Unbonded toppings	Mesh reinforcing bond breaker on old concrete surface: minimum thickness 2½ in (nom.6.4 cm)	

*Under abrasive conditions on floor surface, the exposure will be much more severe and a higher quality surface will be required for Class 4 and 5 floors. Under these conditions a Class 6 two-course floor or a mineral or metallic aggregate monolithic surface treatment is recommended.

of floor would then, with experience, have an associated range of service conditions. In this connection the Schmidt hammer method of test described by Chaplin is of particular interest for abrasion resistance both in the laboratory and in the field. However, since many cementitious finishes for light use are not required to be fully bonded, one wonders how this would affect the results from the Schmidt hammer, which has sometimes been used as a method of detecting unbonded areas.

One system of classification of floors has been defined by the American Concrete Institute in their Standard 302-69, as given in Table 1. The breakdown is too detailed for general use and relates only to concrete finishes and not other products. An alternative, more general, classification might be as given in Table 2, to cover the range of end use on a more rational basis. Testing methods need to be defined for each category both for laboratory and site use.

Table 2 Suggested Floor Classification

1. Light Domestic	Houses; Flats
2. Heavy Domestic	Schools; Institutions
3. Commercial	Offices; Canteens; Assembly Halls; Shops
4. Light Industrial	Light Workshops and Stores
5. Medium Industrial	Machine Shops; Warehouses; Factories; Textile Mills
6. Heavy Industrial	Motor Vehicle Assembly; Ship Building; Railway Workshops; Steel Industry
7. Special	Dust Free Floors (Electronics, Computers, Printing); Chemical Resistant Floors (Chemical Factories, Dairies, Breweries); Sterilizable Floors (Operating Theatres); Radiation Resistant and Decontaminable Floors (Nuclear Power Stations)

Load Testing

Shaw's paper in this Session mentions wheel type and load as major factors in considering the floor finish. A few years ago we investigated the intensity of compressive stress produced by a steel wheel on a small hand operated fork truck travelling over a concrete floor. We were surprised to find that conventional elastic theory indicated a maximum stress of 380 N/mm^2. One can visualize that repetition of a wheel stress of this order would produce a rolling indentation in the floor surface leading eventually to dusting, abrasive wear and break-up due to a combination of the Poisson effect with intense shearing action at the sides of the wheel.

Could this perhaps suggest a method of test for classifying floors? A laden, small, hardened steel or cast iron wheel cycling to and fro would in time lead to the break-up of any normal concrete slab. The number of cycles to failure (given a precise definition of failure) would define the grade of floor. The 'Dynamic Indentation by Chaîr Leg' test referred to in Hogson's paper could be also of relevance here.

Wear Testing

Referring back to the list of defects given earlier, it appears that resistance to dusting, abrasion and surface break-up could all be tested by a single trafficking test such as that outlined above. Adhesion to substrate can be quickly tested by any existing standard pull-off test. This is largely a test of substrate preparation and can be easily and quickly carried out on site.

Polishing

There appears to be no standard test for resistance to polishing. Here also a very simple test for either laboratory or site use could easily be devised. In a recent paper Hall (2) has quoted a figure of 0.04 for the coefficient of friction required to ensure freedom from slipperiness. By abrading a floor sample under standard conditions, e.g. by counting the number of passes of a specified carborundum block under a range of specified loads to reduce the coefficient of friction to this or some other agreed figure, an immediate measure of resistance to polishing is obtained. Both abrasion and coefficient of friction measurement could be carried out on either a dry or wet surface as appropriate to the expected service conditions. The measurement of coefficient of friction is, of course, an extremely simple test operation.

Chemical Resistance

Chemical resistance is one of the most difficult floor properties to establish. The number of chemicals used in industry is enormous and their effect on floor finishes in every case depends on their concentration, temperature, velocity, contact time and mode of application. Where alternate drying and wetting occurs, concentrations of chemicals may build up over a period of time. Conversely, in some cases a dilute chemical may be more harmful than a higher concentration (for example we have found that 50 per cent sulphuric acid attacks an epoxy resin more vigorously than the 100 per cent acid). Even a neglected rainwater drip can do serious damage in time.

Cleaning

The cleaning regime may also have a significant effect and in fact, the chemicals used for cleaning and/or sterilizing purposes may themselves attack the floor finish. For these reasons the value of laboratory tests in determining the chemical resistance of floor finishes is limited unless service conditions can be precisely specified. There thus appears to be a good case for directing a substantial research effort into a review of the performance and useful life of floor finishes of all kinds in different industrial situations under actual service conditions.

CONCLUSION AND SOME SUGGESTIONS FOR BETTER FLOORS

At the present time there are in the U.K. no universally accepted standard test methods for all the essential performance requirements of concrete floors and floor finishes. Since standard tests are not available, it is at present impossible to define grades of floor and floor finishes or to specify by performance alone. It is therefore currently necessary to specify both the materials and methods of construction in detail including substrate preparation. A better alternative would be to specify a Grade 'X' floor on a Grade 'Y' substrate with a Grade 'Z' surface condition. The present organisation, the design by the engineer and the surface finish by the architect, can lead to a division of responsibility which may result in lack of co-ordination of design and specification requirements between elements, with resulting incompatibility.

Without a suitable classification system, detailed specifications are required concerning the following:

i) Tolerances in slab surface levels;
ii) Surface preparation of slab;
iii) Moisture content (or maturity) of slab at the time of application of the finish;
iv) Topping mix proportions;
v) Grading of aggregate;
vi) Water-cement ratio;
vii) Method of mixing;
viii) Method of consolidation;
ix) Method of surface finishing;
x) Tolerances in finished surface levels;
xi) Methods of curing;
xii) Protection from construction traffic;
xiii) Level of maturity required before putting into service.

For the future, it is suggested that a more rational method of specifying floor finishes is required which will involve the following steps:

1. Establish a standard classification of floors according to service condition.
2. For each classification, establish appropriate performance characteristics. This will require standard tests to be devised and agreed.
3. Carry out a comprehensive survey of the long-term performance of existing floors related to their service classification.

The survey mentioned above, to be useful, must be really comprehensive and include particularly situations involving common types of chemical aggression such as dairies, food manufacture, breweries, fertilizer factories, in addition to floors subject to various intensities of wheeled and pedestrian traffic. It will require the development of field testing methods to assess performance.

All this work will involve considerable effort and expense but cannot fail to be very well worthwhile in relation to the potential savings. The value of new floor finishes in the U.K. is in the region of £50 million annually and the cost of repairs two or three times this figure. Experience suggests that on average a significant proportion of the initial investment is spent on remedial work. In addition a substantial (often unrecorded) cost is incurred by architects, engineers and contractors in taking part in meetings to discuss remedial measures, writing specifications for remedial work and supervising repairs. Consequential costs in lost production or trading may be much higher.

A large proportion of these costs can be avoided if we are prepared to devote effort and resources to rationalizing our method of correctly classifying, testing and specifying floor finishes.

Finally, there is a need for one British Standard to provide guidance for engineers and architects covering the specification of all types of floor finishes, preferably based on the category system. Such a document has been produced for specifying concrete for the industry (B.S. 5528, 1976) where a sample specification proforma lists the range of strength grades, cement types and property requirements. The specifier marks the particular category for each aspect which the producer then meets by whatever means he considers appropriate.

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SPECIAL FINISHES FOR CONCRETE FLOORS

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ABSTRACT The paper reviews a great variety of floor finishes available to improve the performance of good quality concrete substrates. These finishes range from low cost penetrating 'in surface' seals to costly heavy-duty chemically resistant finishes capable of meeting the most exacting service requirements for floors in industrial or chemical plants.

INTRODUCTION

There are a bewildering number of special proprietary floor treatments available for the architect and engineer to consider and much of the brief technical literature describing them suggests that many products would appear to offer the same improved service at vastly differing costs. It is, therefore, not surprising that many specifiers are totally confused and tend to stick to the products they know rather than consider some of the novel products based on new technology which often offer distinct improvements over the materials traditionally used as floor finishes.

Before attempting to classify the various types of special finishes available it is important to consider the concrete substrate itself. By proper use of good mix designs and admixtures, with careful control of the water-cement ratio and careful attention to laying, finishing and curing techniques, concrete itself can serve as a highly durable floor finish under many industrial service conditions without the need for special separately applied finishes (1, 2). Properly laid concrete provides an abrasion resistant floor surface which has good resistance to attack by alkalis and reasonable resistance to attack by mineral and vegetable oils, although oils do cause some staining and impair appearance. Irrespective of how well it has been laid, however, concrete has poor resistance to acids and many other chemicals far too numerous to mention here; where spillage of such materials is envisaged, the concrete must be protected by the application of a special floor finish.

SELECTION OF REQUIRED FINISH

Before selecting a concrete floor finish it is imperative to consider carefully the precise service conditions to which the floor will be subjected. Conditions that must be considered include:

i) Service temperature;

ii) Rate of change of temperature, as rapid temperature changes can cause some heavy duty finishes to crack up due to the high stresses developed by thermal shock;

iii) Nature and concentrations of any materials likely to come into contact with the floor;

iv) Accuracy of laying the concrete subfloor to levels to allow spillages to run away reliably to drains;

v) Grade of concrete laid for the subfloor;

vi) Maximum loads and type of wheels on vehicles using the floor.

Without such precise information, and on occasions even with it, inappropriate floor finishes are all too often used resulting in rapid breakdown of the floor.

REQUIREMENTS OF CONCRETE SUBSTRATE

If at the specifying stage it is decided that the service conditions for the concrete floor do require special finishes to be applied subsequently, then it is imperative that the contractor laying the concrete is fully aware of this factor and does not use a conventional spray applied resin solution curing membrane as this could seriously affect the adhesion of any special finish to be applied subsequently, being difficult to remove uniformly and reliably (3). In these circumstances, and also for industrial buildings where the eventual use of the floor is not known but may, therefore, require a special finish, overlapping polythene sheets or other efficient curing method which does not affect the adhesion of any subsequently applied finishes should be used.

It is important to examine carefully the surface of the concrete substrate prior to applying the special finish. Although the concrete laid by the contractor may indeed have cube strengths well in excess of that specified, it is still possible for the concrete slab to have a very weak surface due to over trowelling, for example. It is the top few microns of the concrete to which the special finish will be applied and it is essential that any weakness in the surface is removed by a technique which is appropriate to the type of finish to be applied (4). Most specialist flooring contractors have sufficient experience to assess the quality of concrete surface without site testing; however, if there is any doubt the surface strength of the concrete should be tested using a simple pull off tester or other appropriate means (5, 6).

As a general rule any concrete base which will be subsequently treated with a special floor finish must not be subject to rising damp and thus any ground supported slabs must incorporate an efficient damp proof course. If there is any doubt the concrete should be tested using a direct reading concrete moisture meter (maximum 6 per cent moisture) or an Edney hygrometer (reading not exceeding 75 per cent relative humidity after 4 hr). It should be stressed that these figures are based on the practical experience of a number of specialist flooring contractors and serve only as a guide. Other factors such as the depth of the slab, time elapsed since placing, and degree of weather protection all have an influence on the moisture content within the concrete substrate.

Finally, before considering finishes, mention should be made of the application of the right joint filler in all movement joints (7). Far too often with a carefully laid concrete floor for industrial service, no detailed attention to joint filling is given and this results in unfilled or wrongly filled joints rapidly spalling at the edges under heavy loads of rigid or semi-rigid wheel traffic, resulting in expensive repairs. Apart from preventing spalling at the floor edges the right joint filler will also improve cleanliness, help smoother running and prolong wheel life of forklift trucks, for example, and contribute to safety. The selection of the right joint filler is a difficult problem, however, and, in general, it is true to say that there is no one single ideal material for floor joints since it is impossible to combine all the performance characteristics ideally required in one product.

SPECIAL FINISHES

In this paper the author has attempted to classify the various types of finishes available for concrete floors in terms of increasing applied costs.

Sodium Silicate and Silico Fluoride Solutions as Concrete Surface Hardeners

Both sodium silicate and silico fluoride solutions are applied to clean, dry, sound concrete floors as dilute aqueous solutions (10 to 15 per cent solids) in 2 to 3 applications, taking care to ensure that all material penetrates and is absorbed into the concrete surface. The silicate or silico fluoride reacts with the small amount of free lime in the concrete to form glassy inert materials on the surface and the successful application of both materials depends upon sealing the micropores in the surface of good quality concrete, leaving its surface appearance and non-skid characteristics virtually unchanged. The main difference between the two types are that the reaction products of the silico fluoride types are somewhat less soluble in water and are also harder, which may give slightly better in-service performance, but at a slightly higher material cost. However, with recent developments in floor laying techniques the concrete substrates for industrial floors are laid with much more dense low porosity surfaces so that neither silicate nor silico fluoride treatments are as effective as they used to be, say ten years ago, when the concrete used had a slightly more open finish and hence was more receptive to these treatments.

It is important to stress that neither sodium silicate nor silico fluoride will improve the performance of a poor, low strength, dusty concrete floor, and if the surface is too porous there is no way that all the material applied can react with the relatively small quantity of free lime in the concrete surface. All that would happen is that the pores would be filled with unreacted powder, producing a most unpleasant alkaline dust which can be very irritating to the skin and eyes when the floor is put into service.

Finally it is important to note that while sodium silicate or silico fluoride treatments properly applied to clean and sound concrete floors can improve their performance, both wear resistance and resistance to mild aqueous chemicals and oils, at a relatively low cost, they are not the answer to all industrial flooring problems as many specifiers appear to believe.

Low Viscosity Resin Based Penetrating In-Surface Finishes

Liquid resin based systems which, like the chemical surface hardeners, penetrate into the surface of the screed and protect the acid susceptible cement matrix from attack and at the same time strengthen the surface of the concrete, are now being increasingly used. These in-surface seals leave the appearance and slip resistance of the concrete floor virtually unchanged, but the treated floors are easier to clean and more durable.

Non-Reactive and Semi-Reactive Resin Solutions

Resin solution penetrating sealers are now available which, for very large warehouse floors, are comparable in applied costs with the concrete surface hardeners and are now being increasingly specified. Experience indicates that certain acrylic resin solutions are proving more durable and offer better protection to chemical and oil spillages than concrete surface hardeners. Other resin solutions, in white spirit or stronger solvent blends, used as penetrating floor sealers include: air drying alkyds (similar to the resins in conventional gloss paints) styrene butadiene resins, urethane oils and styrene acrylates. All such resin solutions are based on flammable solvents and are becoming increasingly less acceptable on health and safety grounds and there is, therefore, increased interest in water-based polymer dispersion floor sealers.

Polymer Dispersions

Polyvinyl acetate (PVA), acrylic and other polymer dispersions have been widely used as anti-dust treatments for concrete floors for many years. In general, the polymer dispersions used have been similar to those used in the manufacture of emulsion paints and until recently have tended to be based on dispersions of relatively large polymer particles (particle size 0.15 to 0.25 μm) and therefore do not penetrate significantly into the surface of good quality concrete. However, much finer particle size polymer dispersions (particle size 0.03 to 0.05 μm) are now becoming available which offer superior performance as floor sealers. The chemical and water resistance of the various polymer dispersions which have been used in the past varies considerably from the PVA types, which are rapidly softened and eventually washed out by water, to acrylic types which exhibit excellent resistance to a wide range of chemicals. It should be noted that there are now available fine particle size polymer dispersions with chemical groups on the polymer chains which will, like the concrete surface hardeners, react with free lime in the surface of the concrete. These types are, however, not yet proven as concrete floor sealers but merit further consideration.

Reactive Resin Solutions

The two-pack low molecular weight epoxy resin systems in volatile solvents have proved most effective for improving the wear and chemical resistance of both good and poor quality concrete floors. The epoxy resin solutions (approximately 20 per cent solids) are high strength systems, very similar to those used in heavy duty chemically resistant trowelled epoxy floors, and, depending on the concrete, can penetrate 100 to 200 μm into the surface of the concrete, similar to the fine particle size polymer dispersions, but in this case the solvent rapidly evaporates and the resin crosslinks to form a tough, chemically resistant resin with a compressive strength up to 30 N/mm^2, thus reinforcing the concrete surface very significantly.

One-pack low viscosity resin solutions based on moisture curing polyurethane systems, which are based on higher molecular weight polymers than epoxy resins and may therefore not penetrate quite so far into the concrete, are also available. However, in practical terms the fact that they can be applied direct from the can without any mixing is a most important factor and they are now widely used (3).

Floor Paints

Floor paints in a wide range of colours and based on a number of different binder systems are used extensively for concrete floors in light industrial applications.

Chlorinated Rubber Paints

Chlorinated rubber floor paints are probably the most common of the lower cost floor paints on the market. They produce tough and chemically resistant coatings, but their adhesion to concrete is not always good, they tend to wear off in patches and cannot be considered as a durable floor finish except under light traffic conditions. However, recoating is a simple job and floors can be easily repainted over weekend shut downs, for example. Similar paints based on other resins such as acrylics, vinyls and styrene butadiene are also used.

Polyurethane Floor Paints and Multi-Coat Finishes

Solvent containing moisture cured or two-pack polyurethane resin paints are also used extensively. They combine excellent abrasion resistance with good chemical resistance and are normally applied in two coats to give a coating thickness of 0.10 to 0.15 mm. In addition, moisture cured polyurethane resin solutions are used for quite thick durable decorative floor finishes. Several coats of resin are applied to the prepared substrate at approximately 4 to 6 hr intervals, with one or more coats being dressed with coloured paint flakes which are sealed in by the next coat and then lightly sanded. This type of floor finish was widely marketed in the U.K. about ten years ago but in the main they were considered unsatisfactory due to rapid discolouration of the floor because of the lack of ultra-violet stability of the urethane resins used, which rapidly turned yellow-brown and looked dirty. However, ultra-violet stable urethane resins which do not suffer this discolouration are now available and this type of durable decorative floor finish is gaining re-acceptance, for example, for kitchens, toilets and reception areas.

Epoxy Resin High-Build Floor Paints

Solvent free high-build floor paints are available which can readily be applied by brush, roller or spray to a prepared concrete substrate to give a thickness of 0.10 to 0.20 mm per coat. Normally two coats are applied and the first coat is often lightly dressed with fine sand or carborundum to give a non-slip, chemically-resistant and durable coloured floor under light industrial traffic conditions, for example, rubber shod wheels (5).

Self Levelling Epoxy or Polyester Resin Systems

Like the high-build epoxy paints, these are solvent free low viscosity systems which are readily applied onto a prepared level substrate to provide a jointless thin (thickness approximately 1.5 mm) chemically resistant floor in a single application (5). The term 'self levelling' by which they are commonly described is somewhat of a misnomer as they require spreading out to a near level finish with a squeegee or the edge of a steel trowel and by themselves then flow out to give a smooth finish. Perhaps a better description is 'self smoothing'. Before the system is cured the surface is normally lightly dressed with fine carborundum; without a non-slip dressing there is a tendency to produce a slippery, very glossy surface which shows every scratch mark. This can be overcome to some extent by careful formulation and also by the application of a slip inhibiting industrial floor polish on a regular basis when the floor is in service.

Similar polyester resin based systems are also available but polyester resins have a tendency to shrinkage during and after curing, and formulation and application of this type of flooring is very critical. At present there are only one or two systems being used in the U.K. Several pharmaceutical companies specify polyester resin floors in many areas because they combine excellent chemical resistance with ease of cleaning to the near sterile conditions required.

Heavy Duty Floor Finishes

A considerable range of different toppings are available for heavy duty service. The correct selection of the most appropriate topping on a cost performance basis can only be made if the service conditions are very clearly defined. In general, heavy duty toppings require a sound (preferably 25 to 30 N/mm^2 strength) concrete substrate.

Granolithic Toppings

In effect granolithic toppings are just a method of producing a high cement content concrete wearing surface on a concrete substrate. The application of separately laid granolithic toppings is always fraught with the danger of debonding and curling, and therefore monolithic grano-toppings are generally essential. However, for many industrial floors where good resistance to abrasion under heavy traffic is required and where a monolithic grano-topping is specified, a suitable floor could be achieved more economically by direct finishing of a high cement content, high strength (55 to 60 N/mm^2) concrete. This was borne out by recent work carried out by Chaplin (8), who found the abrasion resistance measured by a number of different methods to be directly related to the compressive strength of the concrete. This work also showed that the abrasion resistance and compressive strength could also be related to Schmidt hammer test results, which could be of considerable interest for the future non-destructive site testing of concrete floors.

Bitumen Emulsion Modified Cementitious Floors

The use of specially formulated bitumen emulsions as the gauging liquid for graded aggregate/sand/cement screeds can produce a dustless, self healing, jointless surface for industrial areas subject to heavy wheeled traffic under normally dry conditions. This type of topping is normally laid approximately 12 mm thick and has been used very successfully for more than thirty years, particularly in warehouses. The bitumen modified cementitious floor topping is less hard underfoot than concrete and has proved a very popular improvement to warehousemen. However, with recent trends towards high rise stacking, heavier forklift trucks and narrow aisles between racks the topping tends to indent or shove and the truck forks become misaligned with the pallets stacked on the higher shelves of the racks so that it is not possible to get the goods down. Loading levels above about 8 N/mm^2 for short-term and 4 N/mm^2 for an indefinite period, are therefore not recommended for bitumen emulsion modified cementitious floors.

Mastic Asphalt Floors

Hot applied mastic asphalt floors have been used for many years in industrial environments where a good degree of chemical resistance under normally wet conditions is required. Properly laid mastic floors are totally impervious to a wide range of chemicals, but not solvents. In terms of mechanical performance mastic asphalt floors are similar to the bitumen modified cementitious floors but they are generally laid at a minimum of 25 mm thickness and tend to shove and corrugate in service under heavy loads. Mastic floors are not very commonly used now, except where the floor is essentially tanked, such as car park decks over shopping precincts.

Polymer Emulsion Cementitious Floor Toppings

Polymer modified cementitious floor toppings are normally laid approximately 12 mm thick. The incorporation of the polymer latex enables graded sand/aggregate/cement (typically 2/1/1) screeds 3 to 4 mm thick to be readily laid at low water-cement ratios. The polymer addition markedly improves the chemical resistance, tensile strength and bond strength of the toppings and also acts, to a considerable extent, as an integral curing membrane. Polymer modified toppings based on special acrylic latices are widely used in food processing industries particularly meat processing. The toppings can be laid in a Friday-Monday weekend shut down and are reported to be capable of withstanding full service conditions forty-eight hours after laying. They are easily cleaned with typical steam cleaning techniques without the problems of thermal shock breakdown, which has been observed with other heavy duty polymer toppings. By careful selection of the polymer latex, toppings can be produced with excellent mechanical properties and resistance to many chemicals encountered in the food and printing industries, but being based on an acid sensitive cement matrix, their resistance to organic or mineral acids is limited and resin bound mortar floor surfacings should accordingly be used.

Epoxy Resin Mortar Floors

Trowelled epoxy resin floors approximately 6 mm thick are used extensively where a combination of excellent chemical resistance and good mechanical properties are required, in particular abrasion and impact resistance and resistance to very heavy rolling loads. Epoxy toppings are available with compressive strengths up to 100 N/mm^2 and tensile strengths up to 30 N/mm^2. This is achieved by careful formulation of the binder and the incorporation of high strength blended fillers (5). When formulating a system for optimum abrasion resistance, both the epoxy resin/hardener binder system and the filler blends used appear to have an influence. The simulation of abrasive service loads on industrial floor toppings in a laboratory is not simple and numerous wear test machines have been devised. Correlation between different wear test machines is not always good, although most laboratory tests on abrasion resistance do give an indication of the floor's likely performance in service in a qualitative rather than quantitative manner (9, 10).

In one series of laboratory tests carried out to find the optimum wear resistance of heavy duty epoxy resin flooring compositions, a number of different abrasion resistant materials were evaluated using B.S. 416, employing three different epoxy resin binders which themselves had significantly differing chemical compositions and mechanical properties. The results of this work, which was carried out under dry conditions, are given in Table 1. As can be seen from the Table the selection of the abrasion resistant material and the resin matrix both influence the abrasion resistance of the system, although the abrasive material incorporated appears to play a more crucial role.

In wet abrasive conditions, which often occur with heavy duty industrial flooring, a small quantity of abrasion resistant material tends to be carried on the wheels of trucks and produces a grinding paste between the heavy duty wheel and the surface. Since the abrasion resistant material in the surface is generally harder than any sand or grit carried into the factory on wheels the grinding paste tends to become more abrasive as the binder is worn away. Abrasion resistance tests

Table 1 Abrasion Values of Trowelled Epoxy Resin Flooring Compositions, using B.S. 416

AGGREGATE	EPOXY BINDER COMPOSITIONS	MASS LOSS AFTER ABRASION gm
Graded Sand, Zone 1	A	4.10
Graded Sand, Zone 1	B	2.75
Graded Sand, Zone 1	C	2.85
Graded Sand, Zone 2	A	5.5
Graded Sand, Zone 3	A	5.7
Gritstone	A	9.95
Granite	A	1.45
Calcined bauxite	A	0.95
Basalt	A	1.5
Cast Iron Grit	A	0.45
Copper Slag	A	2.25
Sand (Zone 1) Gritstone (50/50 by mass)	A	1.35

under wet grinding paste conditions, however, do indicate a similar order of resistance although the binder appears to play a more significant part. In applications where the flooring is flooded with water for long periods, the resin binder plays a more important part since the strength of the adhesive bond between the particles of abrasion resistant materials can, if the wrong resin binder system is used, drop markedly under prolonged wet conditions. In formulating resins for heavy duty floors it would appear that the adhesive properties of the resin binder used to bond the resistant particles firmly together is the most important factor when selecting a resin system. In the selection of systems for large scale applications, costs must also be considered and on this basis bauxite calcined under defined temperature conditions has been most commonly used as the abrasion resistant material.

Another aspect of epoxy resin mortar floorings which needs careful attention is that their coefficients of thermal expansion are approximately three times that of concrete. This, coupled with the relative low thermal conductivity of epoxy mortar, can cause stresses to be induced at the resin mortar/concrete interface under conditions of thermal shock (e.g. steam cleaning) resulting in break up of the floorings due to initial failure in the concrete. Two approaches have been tried to overcome this problem.

a) Using a lower modulus epoxy resin mortar and applying the topping at a thickness of 3 to 4 mm.

b) Applying a stress distributing flexible epoxy layer 1 to 2 mm thick between the rigid epoxy topping and the concrete.

Both approaches have been used with some success but in a) the lower modulus topping also tends to have lower chemical resistance which can be a problem, while the technique b) is significantly more costly in terms of both materials and labour (9).

Polyester Resin Mortars

Polyester resin floor toppings, similar in performance to the epoxy toppings, have been used but as indicated earlier polyester systems tend to shrink and without careful formulation and laying, shrinkage stresses with polyester resin systems can develop at the interface between the topping and the concrete substrate. Coupled with the additional stresses due to the differences in their coefficients of thermal expansion, this can cause failure at the surface of the concrete substrate (10, 11). Several years ago one U.K. company had considerable success with a carefully formulated polyester mortar topping specifically designed to minimise these stresses, but found that unless it was laid with meticulous care failures could occur.

Polyester resin mortars, however, cure within two hours of placing to give greater strength than concrete and are widely used for the rapid repair of small areas of damaged concrete floors, and with the use of 'igloos' even in cold stores in service. Another polyester resin based heavy duty topping which has proved very satisfactory in service is based on a unique approach. It comprises a blend of treated Portland cement and a dispersion of a special water soluble catalyst system in an unsaturated polyester resin. This blend is mixed on site with graded aggregates and a measured quantity of water. The water addition dissolves the catalyst and in the presence of free alkali from the cement releases free radicals which trigger the curing of the unsaturated polyester resin. The cured product gives a tough floor topping which over the past ten years has been widely used in abbatoirs, dairies and other food processing plants (10, 12).

Polyurethane mortar flooring systems based on somewhat similar technology to this special polyester system have also been used in chemical plants and have given excellent service. The basic urethane polymer is more elastomeric than either epoxy and polyester resins and as such is reported to have excellent thermal properties up to at least 140°C and good resistance to thermal shock. The adhesion of the urethane systems to damp concrete is to some extent suspect and dry substrates are therefore normally essential, although systems with improved adhesion to damp substrates are becoming available (10, 13).

Industrial Tile Finishes

There are industrial flooring situations where the service requirements or the time allowed for laying do not permit the use of jointless floor toppings. For such applications a wide range of industrial tiles are available which will meet most requirements, in terms of either mechanical properties or chemical resistance. When tiles are used in very aggressive chemical environments the main problem is grouting between the tiles with a grout having adequate resistance. Grout systems based on specially formulated furane resins (which in particular resist very strong acids) and epoxy resins are available for this purpose, and tiles laid in very fast setting mortar bedding and properly grouted can be installed and returned to service in under 48 hours. A typical application for tiles is in dairies, many of which operate 364 days a year. By using a fast setting fondue based mortar bed bonded to the underlying substrate with an epoxy adhesive one day and then laying quarry tiles bonded and grouted with an epoxy resin system the following day, it is possible to repair a completely broken down and impossible to clean floor to a good standard with no interruptions to production.

COMPARATIVE APPLIED COSTS

It is difficult to give precise costs of floor treatments as size of total area, areas to be coated at any one time, degree of surface preparation required and other factors all influence costs. The following is a rough guide to comparative applied costs in the U.K., based on November 1978 prices.

FLOOR TREATMENT	COMPARATIVE COST INDEX
Concrete Surface Hardeners, 2 to 3 coats	1 to 1.8
In-Surface Seals:	
Resin Solution Non Reactive, 2 coats	1.2 to 2.5
Polymer Dispersions, 2 coats	1.5 to 2.5
Reactive Resin Solutions, 2 coats	2 to 3
Paints:	
Chlorinated Rubber, 1 to 2 coats	1.7 to 3.2
Polyurethane, 2 coats	1.8 to 4
High Build Epoxy, 1 to 2 coats	3.5 to 7
Multicoat Polyurethane Flake, 4+ coats	5 to 12
Epoxy Self Levelling, 1 to 2 mm	10 to 16
Polyester, 2 to 3 mm	9 to 12
Bitumen Modified Cementitious, 12 to 16 mm	7 to 10
Mastic Asphalt, 25 mm	9 to 13
Polymer Modified Cementitious, 12 mm	8 to 15
Epoxy Trowelled, 6 mm	18 to 24
Polyester, 6 to 9 mm	15 to 20
Industrial Tiles, various	15 to 30

CONCLUSIONS

The range of special floor finishes available is very wide and to many extremely confusing. All that most specifiers can hope to do is to carefully list all the service requirements for the floor finish and approach specialist suppliers for their recommendations for suitable products for the service indicated together with detailed instructions on how they should be applied including concrete substrate preparation.

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CEMENT BASED SMOOTHING AND SCREEDING COMPOUNDS

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ABSTRACT The paper deals with the current state of development, with particular reference to the properties and applications, of cement-based smoothing and screeding compounds, which can be used in thick as well as thin layers for a variety of purposes. Different types of product are available with different application properties, such as thixotropy, flow, self-levelling and self-smoothing, or with the ability to incorporate various grades of aggregate.

INTRODUCTION

In some countries the Standards clearly stipulate the permitted tolerances allowed on surface finishes for floors, walls and ceilings. To fulfill these rather stringent requirements the use of smoothing and screeding compounds on various types of substrate is often necessary.

A substrate which has been improved with the aid of smoothing and screeding compounds often has applied to it different coverings or coatings, for example, on floors, asphalt tiles, PVC based floor coverings, carpets, parquet, tiles of ceramics or natural or artificial stones and terrazo can be laid. Thus, it is important that these smoothing and screeding compounds possess properties which are compatible with those of the substrate, in particular with regard to strength, hardness, wear resistance and resistance to the penetration of water. Furthermore, they must have good adhesion on different substrates, they should not develop any internal tension which would develop shearing stress and thus, in some cases, lead to their being separated from the substrate even when the adhesion is good, but when the surface of the substrate is weak.

All these requirements can be fulfilled with cement-based products, which are discussed in this paper.

DEVELOPMENT OF THE CEMENT-BASED COMPOUNDS

After the second world war, new floor coverings, which were thinner than those previously used, were developed. Therefore, they needed a more smoothly finished substrate, to avoid unsightly markings arising on the surface from a rough substrate containing trowel marks, pores, coarse particles, cracks or scratches, for example. Furthermore, there was often a need for making the substrate level when there was a difference, even if very small, in the height of different areas. Thus, a product was required which could be laid in different thicknesses and could smooth the surface.

The first smoothing and screeding compounds were based on plaster, but plaster, as a bonding agent, does not develop sufficient hardness when used as a screeding compound. So it was necessary to use some additives which would give the desired ultimate hardness. The most common additive is lignin sulphonate which is used very often with the addition of sodium dichromate. The lignin sulphonate is a wetting and fluidizing agent which reduces the water requirement of the plaster-based screeding compounds but, as it also reduces the hardness and the resistance against water, addition of sodium dichromate is necessary to lead to a polycondensation of the lignin derivative, thus obtaining a better hardness and resistance against water. Nevertheless the plaster-based smoothing and screeding compounds are not water resistant, so they cannot be used on moist surfaces or beneath water-based adhesives. Also, the hardness is dependent upon the thickness of the layer, so that normally a thickness of more than 1.5 mm has to be used to get sufficient hardness and in addition these smoothing and screeding compounds do not have any flow or levelling properties so that they develop a rough porous surface.

Thus the plaster-based smoothing and screeding compounds did not fulfill all the requirements which are needed. This led to the development of other products, namely, the casein-based smoothing and screeding compounds. These compounds have some major advantages in comparison to the plaster-based compounds, namely, they have good flow and self-levelling properties, their hardness in thin layers is very good, and the surface of the hard layer is smooth with no large pores.

The casein-based smoothing and screeding compounds are products with casein as the main binding agent, combined with hydrated lime and Portland cement, in which the casein reacts with the hydrated lime and the cement to form a calcium-caseinate which develops the ultimate hardness during drying out. Besides these binders, some filling agents and aggregates are used which help the development of hardness in the different layers. Since casein-based compounds harden by drying out, they are not water-resistant, the calcium-caseinate becoming soft to a degree depending on the time of exposure to water, although there is a certain amount of resistance against short exposure to water. Hardening by drying out also means that only thin layers can be applied, because in thick layers a skin is formed before the inner part has hardened and so the skin cracks. If there is a necessity for thick layers, then the casein-based screed has to be laid in separate thin layers applied one on top of the other. The casein-based products form a good bond on different substrates because casein has good adhesive properties. There is, however, one other disadvantage: the development of internal tension which can produce cracks or a rupture in the surface of the substrate if it is not of very good hardness and tensile strength. As mentioned previously, the casein-based screeds contain cement, but the cement does not hydrate because the gelation of the casein disturbs the normal hydration processes. The cement will, however, bind some of the mixing water used to produce a paste, so that not all the mixing water is lost by evaporation.

At the same time as the plaster-based and casein-based screeds were marketed, latex-based compounds were also being developed. The plaster and casein-based products became more widely used on the Continent, whereas the latex-based compounds were mainly used in the U.K. and U.S.A. The latex-based compounds, in which the binding agents are latex from natural or synthetic rubber and cement, can be used in different layer thicknesses as they harden through hydration. The hardened products are resistant against water, but they have no flow or self-levelling properties, and consequently they cannot be used in thin layers. In addition, these products have to be mixed on site from two different components, namely the cement-based powder has to be mixed into the water-based latex emulsion. Sometimes, an addition of more water is required and this very often can lead to bad results when the mixing ratio is not in the correct proportions. The latex-based compounds, however, have some advantages which are not present in the plaster or casein-based compounds, but there are some disadvantages also, such as their inability to flow or self-level and their lack of smoothness and freedom of pores.

During the period 1965 to 1970 there was a development of new products, based on special cements combined with casein. These powder-based products have only to be mixed with water on site and can be used in thin or thick layers. They have very good flow and self-levelling properties. The special cements used are produced in a way so that when hydration occurs, although the casein constituent has a retarding effect, hydration is achieved by the quick hardening and quick setting properties of the cements. Thus, these products have properties which make them widely used; they are suitable for thin and thick layers, they are self-levelling, will give smooth and non-porous surfaces and will be resistant against water.

There is, however, one property which limits their field of application: the development of internal tension. It is well known that the shrinkage of cement-based products is dependent on the size of the aggregate used. As the cement-based surfacing and smoothing compounds only contain fine fillers and aggregates, the magnitude of the tension is not surprising. Therefore, these products have to be mixed with coarse aggregates if they are to be used in thick layers. This technique is used very often with good results. The quick hardening and the quick drying out of these cement-based screeds is very helpful in different thicknesses, but sometimes the surface of the finished substrate is not very hard, or has some impurities. In these cases, the compound and the substrate may delaminate, depending on the degree of hydration and drying out. These limitations meant there was still a demand for cement-based products with no or only very small shrinkage, or internal tension.

During the last three years, research has been undertaken aimed at developing products which would fulfill most of the desired properties, and the new cement-based smoothing and screeding compounds developed have many advantages in comparison to the previous ones. They are based on quick-setting cements or regulated set cements with low tension, so that they can be used in all thicknesses required without forming cracks or disturbing the adhesion to the substrate. Their fast setting time and fast hardening allows the application of the final floor covering very soon after screeding. They have excellent flow or self-levelling properties and give a very good adhesion with the substrate, a smooth surface without any big pores and a very good adhesion to all floor coverings. The special screeds can also have adjusted thixotropy if it is required to apply them on walls or to produce thick layers with an incorporation of various grades of aggregates for reducing cost. Thus, it could be expected that these new cement-based screeding compounds, Table 1, will capture a large part of the market.

TESTING SPECIAL PROPERTIES

It is appropriate to discuss the properties the different screeds should have and how these properties can be examined or tested. The screeds should have a very smooth surface, they should close the pores of the substrate and have no large pores themselves. It is normally required that any adhesive which is applied on these screeding compounds has good adhesion and that the quantity of the adhesive which has to be used is not very large. The absorption capacity should be high enough to give a very good key to the adhesive but not too high so that most of the adhesive is absorbed. The absorption capacity of smoothing and screeding compounds can be measured as the time taken to absorb a drop of water placed on the finished surface. However, for compounds containing a water repellent agent, namely latex-based compounds, this measurement can lead to misleading results, and the test

Table 1 Surfacing and Screeding Compounds for Floors

MAIN BINDING AGENT	POSSIBLE THICKNESS OF LAYER, mm	SHRINKAGE VALUE DURING HARDENING AND DRYING, %	NORMAL DRYING TIME REQUIRED FOR 3 mm, hr
Plaster	1.5 to 3	0 to 0.1	6
Casein	0 to 3	0.8	24
Special Cements and Casein	0 to 10	0.4	6
Regulated-Set Cement and Organic Resins	0 to 25	0.05	3

should be performed using an organic solvent instead of water.

Levelling of the substrate can only be achieved with good self-levelling properties in the smoothing compound, and this is particularly important when thin floor coverings are to be used. The self-levelling property is dependent upon the thickness at which the compound is laid, and, therefore, can be measured only as a relative value between different products having the same thickness; the commonly used method employs a comb with teeth at a distance of more than 3 mm.

The hardness of the surface should be improved by the cement-based screeding compound. This surface hardness can be tested by scratching the surface and observing the depth and width of the scratch, although this is not a very satisfactory method because width and depth of the scratch are dependent upon the filler and/or aggregate being incorporated in the screeding compound. New testing techniques are being developed for this purpose.

The inner hardness of the screeding compound should be comparable to that of the substrate. This inner hardness can be tested by the ball pressure hardness or ball impact hardness method, in which a ball is impacted on the surface and the diameter of the indentation is measured.

The resistance against walking over the surface or against castors is also very important. This resistance can be tested by means of a special heavily loaded rolling chair; such equipment is used for testing carpets. This property cannot be achieved with a very hard binding agent, such as cement. Therefore, some more expensive raw materials, as synthetic resins, dispersed in water, have to be added.

The screeding compounds in the hardened state are required to have sufficient resistance against water so that water coming from the substrate, or from water-based adhesives will not adversely affect their performance in service. A hardness test measuring the time taken for water to affect the performance of the screeding compounds is considered to be suitable for this purpose.

The hardened screeding compounds are also required to be resistant against the different adhesives or paints which may be applied subsequently. Therefore, the compatibility of the screeding compounds with different coating products should be established. Testing for compatibility with the substrate is clearly most important as the screeding compounds must form good adhesion with them. This adhesion can be tested by means of a tensile strength test, using a metallic anchor which is bonded on the surface of the screeding compound with the aid of a two-component adhesive (e.g. an epoxy resin); the tensile strength is then measured by pulling off the metallic anchor.

There are some other properties which are required for special applications. For these properties some special test methods have been developed, although in most cases the testing methods are related to those which are used for the coatings used on the screeding compounds.

SUBSTRATE TO BE COATED

The substrate on which a cement-based smoothing and screeding compound is to be applied has to be examined to determine if any preparation is necessary before the screeding compound can be applied. It is important that damp-proofing is done prior to screeding, because the cement-based compounds are water-resistant but not waterproof. Thus water coming from the concrete or other substrate can go through the cement-based screed and influence the adhesion of paint or other layers which are applied to the screeding compound and weaken or disturb them.

The substrate has to be sound and firm enough to have sufficient wearing capacity for the screeding compound and for the floor tiles or other decorative covering used. If the strength is not sufficient the substrate has to be abraded until a firm surface is reached. In some cases a primer will help hardening but one should be aware that priming will only harden the surface and cannot make good a thick defective layer. All impurities which are on the surface of the substrate should be removed because they can become a separating layer preventing the screed from adhering properly to the substrate. A smooth surface such as natural or polished stoneware, terrazzo, etc. has to be primed with special primers. Very often a polychloroprene primer will be helpful but, as this type of primer always incorporates organic solvents, one has to be careful during application. A special emulsion of organic resins can also build up a key between the substrate and the screed but when this key or primer has to be resistant against water, then one should use epoxy-based, two-component products.

The screeding and surfacing compounds can improve the properties of the surface of the substrate and they should be used for this reason alone. It is impossible for them to improve the whole thickness of a concrete or other substrate on which they are applied.

WORKING PROPERTIES

The working properties of screeding compounds are influenced by the different fillers or aggregates used with them so that these materials have to be chosen carefully. The mixing of cement based surfacing and screeding compounds and their application is normally very simple, so that it is not necessary to make special mention of anything. It is no doubt apparent that these cement-based products are sensitive to excess water and, therefore, the mixing ratio which is indicated by the producer should be used carefully. Normally, the powders are wetted quickly and easily with the water and the mixing is very simple, it can be done with a stick, although preferably a slow speed stirrer should be used. Some of the screeding compounds need a few minutes maturing time but the newer types are ready for application directly after mixing. Application should be carried out in such a way that the screeding compound is pressed into the substrate to improve adhesion. After this, the required thickness of layer can be spread.

Since most of the screeding compounds are formulated in such a way that there is no risk from too rapid evaporation of water, it is normally not necessary to keep them damp; the layers need to be protected only when there are high temperatures combined with strong draughts.

Grinding or polishing of these screeds is possible directly after setting. It should be done while they are still wet when their hardness is not too great. If it is necessary to lay a second layer on the first one it should also be done directly after the first layer has set. When this is not possible, the first layer should be primed or damped down to ensure that the next screed will adhere firmly.

THE USE OF WATER DISPERSED POLYMERS IN CEMENT MIXES FOR FLOOR TOPPINGS AND CONCRETE REPAIRS

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ABSTRACT Water dispersed polymers, also known as emulsion or latex polymers, are added to cement mixes used for floor toppings, patches and repairs to damaged concrete. The polymer imparts vastly improved abrasion resistance, adhesion and toughness, gives increased tensile strength, and, if required, the mix can be made completely waterproof. Polymer addition promotes good cement curing, even in very thin sections as when a patch is feathered out at the edges. This paper reviews polyvinyl acetate emulsions, widely used over more than two decades, and the recently developed epoxy resin emulsions.

INTRODUCTION

Cement mixes are commonly applied as relatively thin toppings or screeds on a concrete base as the final smoothing and surfacing operation in floor construction. Plain cement screeds, unless laid monolithically on fresh concrete, must generally be at least 50 mm thick and the finished surface, unless given some form of special treatment, has poor abrasion resistance resulting in dusting and wear problems. Under severely wet conditions, this kind of surface can quite rapidly wash away down to, and even below, the large aggregate particles. Polymer dispersions can be used in cement mixes to improve their tensile strength, resilience and corrosion and oil resistance and give greater resistance to dusting, abrasion and impact. The improved bond to the underlying concrete and the better curing of thin sections means that very thin screeds can be laid which are nevertheless durable and firmly bonded to the base.

The earliest systematic work in this area seems to be that of Geist et al (1) on the addition of polyvinyl acetate emulsions to cement mortars. The considerable amount of work carried out since then on a variety of materials has been reported mainly at the RILEM Symposium in 1967 (2) and at the Concrete Society congress in 1975 (3). Interest has centered on polyvinyl acetate (PVA) emulsions, copolymers of PVA, and other vinyl esters, on styrene butadiene rubber latex and on acrylic and epoxy systems. While the author has previously briefly reviewed the available information (4), this paper is mainly devoted to PVA and epoxy emulsion systems, which are used in materials for concrete floor repairs and toppings.

POLYVINYL ACETATE EMULSIONS (PVA)

Polyvinyl acetate is a clear, brittle polymer which finds wide application in adhesive formulations. Production is by an emulsion polymerisation process, the end result of which is a product consisting of a great many small particles of polyvinyl acetate dispersed in water. The size of these particles is around 1 to 2 μm so that the dispersion is a creamy liquid having the appearance of milk or white emulsion paint. For construction applications, the polymer particles are required to be somewhat rubbery, which is achieved by adding a plasticising material which dissolves in the polymer and softens it.

When the emulsion dries out very considerable surface tension forces are developed which force the PVA particles into contact with each other and with the substrate. The particles fuse with each other to give a solid material, and also form a strong bond with the substrate. In this state PVA adhesives will give good bond, as strong as the materials being joined, to a wide variety of materials including brick, smooth concrete, glazed tiles, wood, paper, plaster and plaster-board.

When the correct quantity of PVA is added to cement mixes the physical properties of the resulting screed are greatly improved, particularly in the case of thin floor toppings cured under normal site conditions. Geist et al (1) found that the effects of polyvinyl acetate on Portland cement-sand mortars were dependent on the amount added. Increasing the PVA proportion in a mix up to a polymer solids-cement ratio of 0.2 improves its properties generally; at this ratio the polymer fills voids in the cement matrix and appears to produce improved adhesion between particles. At higher polymer-cement ratios the polymer tends to become the continuous phase and obstructs bonding in the cement gel. This results in a reduction in the physical properties of the concrete or mortar as excessive amounts of polymer are added. At the optimum level of polymer addition, polymer-cement ratio of 0.2, Geist et al found that the properties of cement mortars were changed as shown in Table 1.

All the tests were performed on samples made with a constant water-cement ratio. Air entrainment was minimized by careful mixing, but was always a little higher when the polymer was incorporated. The samples were cured at 50 or 100 per cent relative humidity for 28 days and were tested whilst still in the same state of full or partial saturation. The results in Table 1 are shown plotted in Figures 1 and 2.

Curing under 100 per cent relative humidity for 28 days represents the best possible conditions which can be attained in the field. Under these conditions addition of PVA to cement reduces the tensile and compressive strengths of cement mortars, although abrasion and impact resistance are improved. However, if these saturated samples are allowed to dry out before testing then their tensile and compressive strengths increase, with tensile strength being higher than for plain mortars. Thus, even under ideal curing conditions polymer addition increases the strength of mortars and concretes used under normal dry conditions. The loss in strength when wet does, however, severely limit the use of PVA in load bearing structures which may become soaked in water.

Under more practical curing conditions, i.e. 50 per cent relative humidity, the addition of PVA produces marked increases in strength and bond. These improvements are particularly valuable in screeds and repair work which are normally cured under far from ideal conditions. Under such curing conditions, the polymer dispersion appears to retain water in the mix. Consequently the cement is able to hydrate properly and develop strength. The workability of cement based mixes is greatly increased by the addition of PVA dispersions which offers the possibility of reducing the water content in a mix, obtaining further increases in the strength

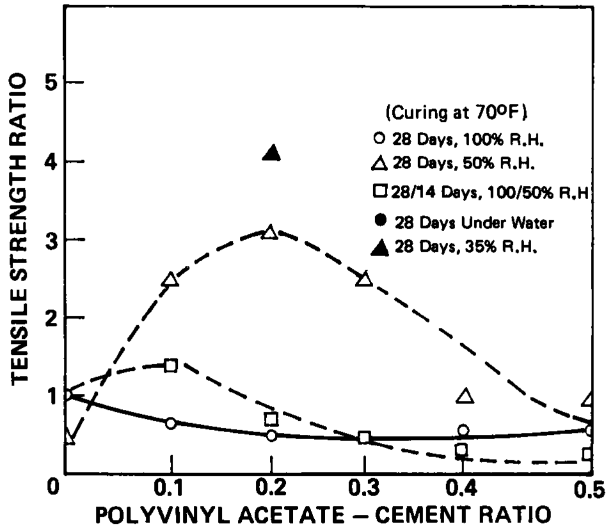

Figure 1 Relative Tensile Strengths of Cement Mortars

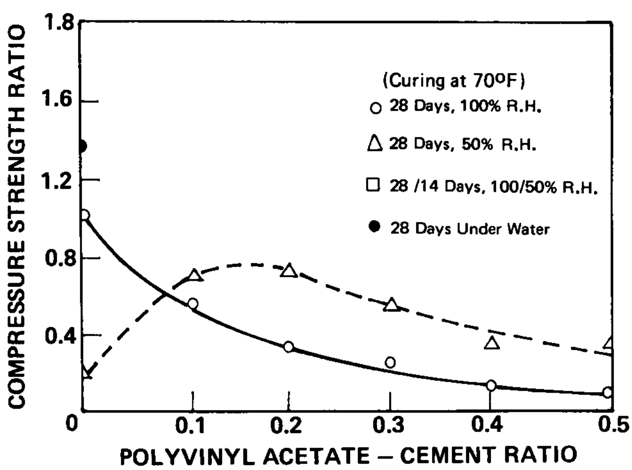

Figure 2 Relative Compressive Strengths of Cement Mortars

Table 1 Properties of PVA Modified Mortars

PROPERTY	RELATIVE CHANGE*			
	Cured at 50% R.H.		Cured at 100% R.H.	
	P/C**=0	0.2	0	0.2
Tensile Strength	0.40	3.00	1.00	0.80
Tensile Modulus of Elasticity	0.20	0.30	1.00	0.04
Tensile Elongation at Rupture	1.00	30.00	1.00	45.00
Compressive Strength	0.20	0.75	1.00	0.30
Compressive Modulus of Elasticity	0.35	0.20	1.00	0.10
Compressive Deformation at Rupture	2.00	5.00	1.00	4.50
Abrasion Resistance	-	-	1.00	10.00
Impact Resistance	-	-	1.00	1.20
Corrosion Resistance	-	-	1.00	> 1.00
Adhesion	0.50	5.00	1.00	1.00
Thermal Expansion	1.00	1.00	1.00	1.00

*Change in properties relative to non-polymer modified mortar cured and tested at 100 per cent relative humidity.
**P/C is polymer solids-cement ratio.

and durability of the final product. The effect of reducing the water content on cement mortars containing polymer is shown in Figure 3.

The plain mix used in this study gave a maximum tensile strength at a water-cement ratio of 0.43, reducing at lower ratios as decreased workability resulted in poor compaction. When PVA dispersion was added, it was possible to decrease the water content of the mix and obtain higher strength even at a water-cement ratio of 0.32. The improvement obtained was found to be of the order of 100 per cent.

The resilience of PVA modified mortar has been tested in another study (5), using a 3 to 1 sand-cement mix containing 7 per cent PVA to cast thin slabs which were tested by repeated flexing. There was no failure of the PVA modified slabs when flexed 7200 times, while the control slabs, without PVA, cracked after being flexed only 2700 times.

A long history of PVA use has served to confirm these early results, with later work, notably that reported at the RILEM Symposium in 1967, being generally in agreement (2). Cherkinsky (6) and Ghosh and Pant (7) in particular reported relevant work demonstrating the benefits obtained. Desov (8) reported test results obtained at high humidity and underwater immersion confirming that there is no advantage to be gained by the addition of PVA to concrete used under these conditions.

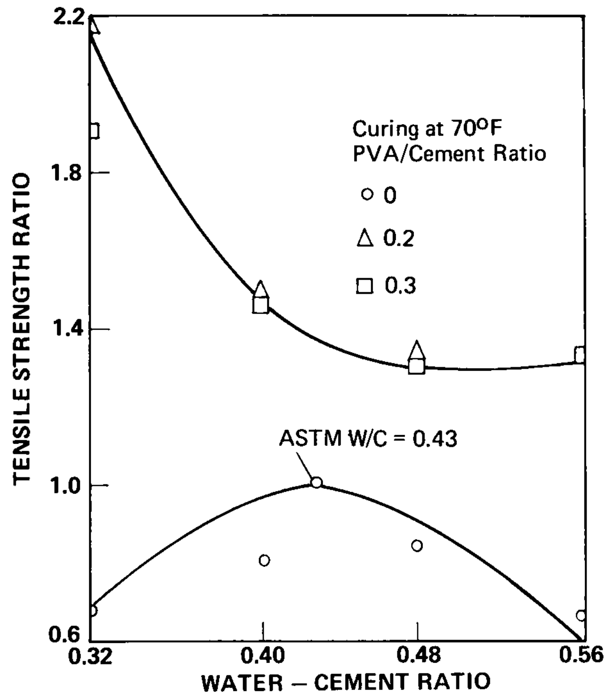

Figure 3 Effect of Water-Cement Ratio on 7 Day Tensile Strengths

Cherkinsky also studied the effect on PVA modified mortars of hydrolysis, to which it is known that polyvinyl acetate is susceptible. This work confirmed that hydrolysis did seem to occur under wet conditions but found no consequent change in the strength of the mortar. It may be that the polymer dispersion exerts its major influence during the cement curing process, and subsequent changes to the polymer then produce only minor effects.

The polyvinyl modified mortars, therefore, can be used as heavy duty, wear and impact resistant floor toppings and as concrete repair mortars, but they should be limited to dry service conditions. For wet service conditions, the use of other polymers, such as the water based epoxy resin systems described in the following section, should be considered.

EPOXY RESIN EMULSIONS

Floor topping and repair mortars based on epoxy resins have been available for some years. These consist usually of a liquid epoxy resin incorporating various fillers and are high strength heavy duty materials, giving excellent adhesion to a variety of substrates. High materials cost and some practical difficulties in dealing with these mortars have tended previously to restrict their application. In recent years, however, epoxy resin materials have been introduced which are dispersed in water and can be incorporated in cement mixes in a manner similar to polyvinyl acetate emulsions. These products consist of two components, resin emulsion and hardener emulsion, the user mixing the two together and adding them to the concrete mix. Two pack water dispersed systems are formulated so that equal volumes of resin and hardener are mixed. This reduces the possibility of errors in use, particularly when non-standard size mixes are required. Epoxy resin dispersions are not quite so convenient to use as are the non-reactive PVA dispersions; being reactive there is a definite working life, after the mixing of resin and hardener, which must not be exceeded.

The use of epoxy resins, however, is justified by the superior properties which these resins impart to concrete and mortars. The improved properties relate particularly to damp and wet service conditions where the chemical and water resistance of epoxy based materials are particularly advantageous. For example, it is a simple matter to produce a strong concrete to concrete bond using PVA. After immersion in water, however, the PVA bond will readily fail in the bond line, whereas a similar epoxy bond will not fail in the bond line but rather in the concrete. The bonding layer of resin can also be used as a waterproofing membrane.

In general, epoxy resin dispersions in water are used as bonding aids and as cement admixtures in much the same way as PVA dispersions. They may be applied to damp surfaces to act as bonding aids or added to concrete and mortar mixes, in which optimum properties are generally developed at a level of addition equal to 20 per cent of the cement content. A marked plasticising effect is obtained in cement mixes, allowing the use of low water contents, as low as 0.35 water-cement ratio, further aiding the increase in mortar quality which ensues from their use.

Due to the water retaining properties of such mixes, the cement will cure well even in thin floor toppings, the epoxy content itself curing rapidly and allowing early use of the floor. Figure 4 illustrates the increase in tensile, compressive and flexural strengths obtained when an epoxy dispersion was added to sand-cement mixes which were cured in thin (6 mm) sections (9). This Figure demonstrates the large increase in strength obtained if an epoxy resin dispersion is added to cement mixes cured under practical, non-ideal conditions. In this respect there are similarities with the data reported by Geist et al using PVA dispersions as described previously. The difference between the two materials, however, is that whereas PVA mixes tend to lose their increased strength under damp conditions, epoxy mixes retain their properties.

Tests were carried out (10) on a commercially available product, HydrEpoxy 260, by an independent laboratory, using a standard mortar mix containing epoxy emulsion, ordinary Portland cement and sharp washed sand in the proportions of 1:1:3 by mass respectively. The results of tests carried out on air cured samples of this mix are described below.

Resistance to Abrasion

Taber Method

The samples were tested at ambient temperature on a Taber abrasion machine, using Calibrade H-18 (non-resilient) abrading wheels. Unfortunately no results were obtained from this test as the HydrEpoxy 260 wore away the abrasion wheel after only 100 cycles, while very little abrasion of the material took place during this time.

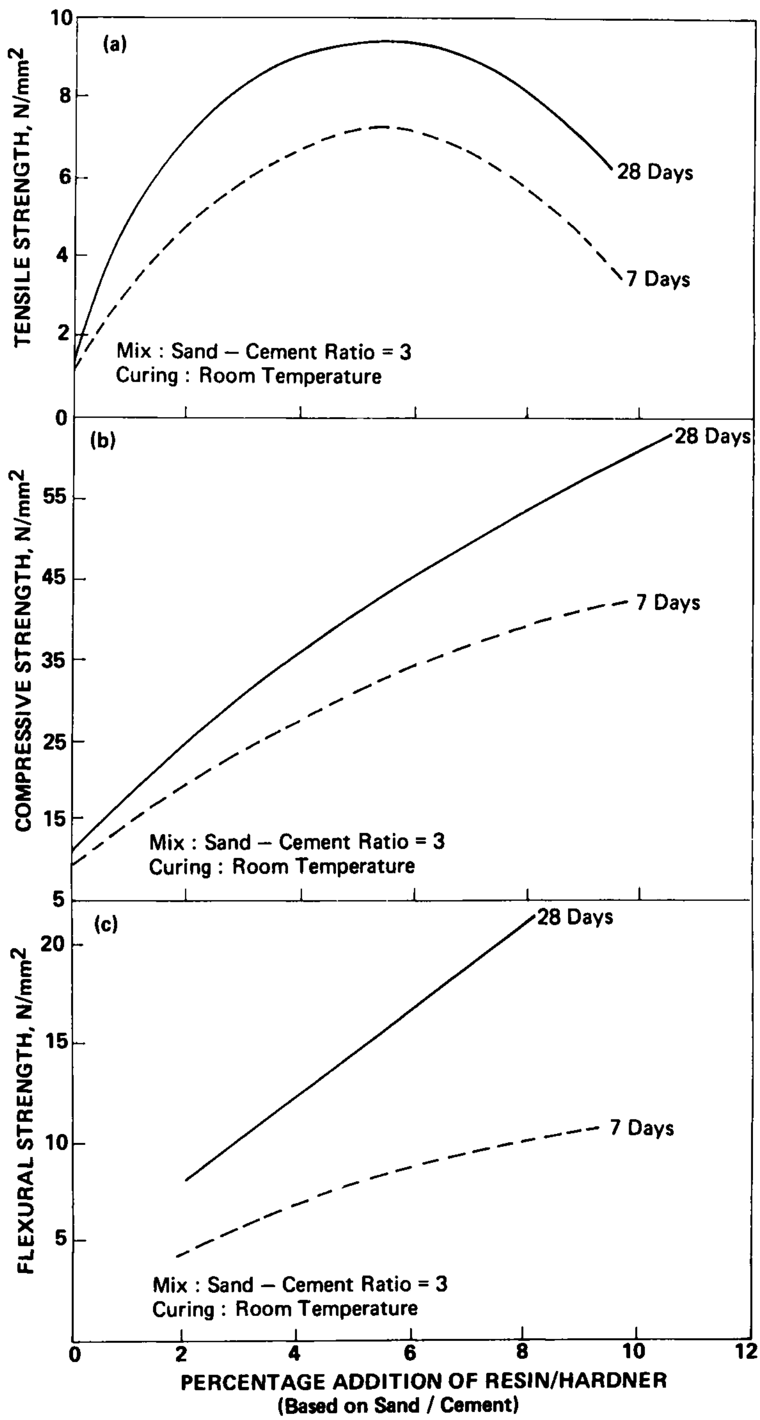

Figure 4 Influence of Epoxy Resins on Strength of Sand/Cement Mixes
(a) Tensile, (b) Compressive and (c) Flexural Strength

A'Court Method

The samples were tested on an A'Court abrasion machine as described in B.S.784:1953. The samples were weighed initially and after periods of ¼, ½, 1, 2 and 4 hr and the loss in mass recorded, see Table 2.

Table 2 Abrasion Tests

SAMPLE	TOTAL MASS LOSS, g					
	¼ hr	½ hr	1 hr	2 hr	3 hr	4 hr
1	0.0	0.1	1.1	3.2	-	6.4
2	0.1	0.3	1.8	5.8	-	12.2
Reference A*	-	-	0.9	2.5	4.0	5.6
Reference B*	-	-	1.3	2.9	-	6.2
Reference C*	-	-	0.6	1.5	-	2.4
Control Specimens						
Reference D**	-	-	12.3	16.3	20.3	24.0
Reference E**	-	-	12.2	17.4	21.0	25.3

*Cladding material of reconstituted slate with resin binder and glass reinforcement; results obtained from existing data.

**Sand/cement mortar (3:1 mix by mass sand:Portland cement); results obtained from existing data.

Resistance to Damage Caused by Loading

Dynamic Loading

This test is described in the UEAtc Common Directive on Thin Flooring, part 2:232, 'Dynamic Indentation by Chair Leg'. The apparatus consists of a movable frame provided with two legs 300 mm apart tipped with cylindrical brass ferrules 20 mm diameter, with their edges smoothed but not rounded, and having weights added to make a total load of 85 kg. Drive rods allow the frame to pivot freely about the line of support of the legs, which passed from the vertical position to an inclination of 20° to the vertical at a rate of 26 oscillations per minute. A total of 100 cycles were made before the samples were inspected for damage, the only effect noted being a slight indentation of the surface caused by the leading edge of the moving feet. No cracking or other deformation of the HydrEpoxy 260 layer was observed.

Static Loading

This test is also described in the UEAtc Common Directive on Thin Flooring, in this case part 2:234, 'Static Indentation by Furniture Leg'. The rig consists of a metal plate fitted with three legs at the apices of an equilateral triangle with sides of 370 mm. The legs terminate in cylindrical brass ferrules each of 10 mm diameter, the lower edge being smoothed but not rounded. The rig weighs 19.2 kg when placed on the sample and the indentation caused by this mass was measured by means of a comparator fixed at the centre of the triangle of contact. The load was then increased to a total of 300 kg and maintained at this level for three days, the indentation being noted as a function of time. The extra load was then removed and any permanent indentation noted. No indentation was observed either before or after loading of the test sample.

Resistance to Chisel Impact

The sample was subjected to 40 impacts from a 1 kg chisel dropped from a height of 200 mm. The chisel head is 20 mm wide and has a 90° tip angle. The damage caused by each impact was recorded with reference to the scale shown below. Typical results obtained at a temperature of 20°C gave a mean value (of 40 impacts) equal to scale category 2, with no impacts in categories 4 or 5; the same results were found for tests carried out at 0°C.

Scale of damage,
0 : No damage,
1 : Surface mark,
2 : Slight indentation,
3 : Severe indentation,
4 : Puncture, part penetration of the chisel head,
5 : Puncture, complete penetration of the chisel head.

Resistance to Steel Ball Impact

Samples retained from the abrasion test (A'Court Method) were used for this test. The samples were subjected to a series of impacts from a 1 kg steel ball dropped from heights of 0.5, 1 and 2 m, giving impact energies of 5, 10 and 20 J respectively. The only damage caused by testing at all heights up to the maximum of 2 m was slight indentation at the point of impact; no cracking or flaking occurred in either sample even at a test temperature of 0°C.

CONCLUSIONS

As the above data shows, the addition of epoxy resin emulsions to cement mixes produces marked improvements in their wear properties. The bulk properties of carefully cured mortars are less affected while the compressive strength is not increased and may even be reduced slightly if air entrainment is increased due to use of an emulsion which does not include a defoamer; tensile strength is increased by approximately 47 per cent.

Cost naturally rules out the use of epoxy resins simply for the increased tensile strength imparted; except in specialized applications it is cheaper simply to use an extra 50 per cent thickness of concrete. For flooring and other work requiring wear resistant surfaces, however, the epoxy emulsion materials give very useful properties at a competitive cost. At current prices it is possible to lay a 6 mm floor topping for a materials cost of £4.50 per m^2, with a low labour cost since water dispersed materials are relatively very easy to work with.

For patching and repair work, it is essential to add a material of the type described to the mortar to obtain adhesion to old concrete, and to obtain proper curing in the thin sections of mortar used.

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ABRASION RESISTANT CONCRETE FLOORS

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ABSTRACT Concrete in abrasion resistant floors must be of a high grade, well compacted, properly finished and adequately cured. Most failures occur because recommended specifications and construction practices are not applied. The settlement of disputes over floor failures would be helped by the development of a classification system and of a simple test for assessing abrasion resistance. Research is showing that the Schmidt rebound hammer is sensitive to factors known to affect the abrasion resistance and may, therefore, be useful for this purpose.

INTRODUCTION

The abrasion resistance of industrial concrete floors is of great importance because failures cause considerable problems for the user and may involve very high costs in repair or replacement of the floor. Conversely, over specification may incur needless expense. Of the many enquiries on floors dealt with by the Cement and Concrete Association, a large proportion relate to abrasion resistance and other associated problems. Often these problems lead to disputes between the user, the specifier and the contractor which are difficult to resolve satisfactorily. Many of these problems could be avoided by the correct application of existing recommended specifications and construction techniques and by a better understanding of the factors influencing abrasion resistance. One of the difficulties in resolving disputes is that there is no simple in-situ test for assessing abrasion resistance.

The purpose of this paper is to discuss briefly the factors influencing abrasion resistance which have been established by previous research and to describe the author's current research programme aimed at developing a simple in-situ test for assessing abrasion resistance. A classification system for abrasion resistant floors is also proposed.

CAUSES OF ABRASION

Severe abrasion of concrete floors is caused by heavy objects being rolled or dragged over, or impacted onto the surface. Steel and hard plastic wheels of heavily laden trolleys are particularly damaging and considerable abrasion may be caused by steel pallet legs being pushed across the surface. Foot traffic, rubber or soft plastic wheels and pneumatic tyres cause little wear unless an abrasive medium is present, but they may cause dusting, loss of surface texture and polishing.

FACTORS INFLUENCING ABRASION RESISTANCE

A previous survey (1) has shown that there has been much research on abrasion resistance in the U.S.A., some in Europe, but little in the U.K. All research programmes have used accelerated wear tests involving impact loading, shot blasting or abrasion by means such as steel ball bearings, discs and dressing wheels. These have produced results which are, in some instances, highly reliable and have enabled certain basic factors to be related to abrasion resistance.

Concrete Strength

Concrete strength is generally the most significant factor influencing abrasion resistance. There is considerable evidence of a good correlation between concrete strength and abrasion resistance; Figure 1 illustrates the relationship derived by Witte and Backstrom (2). Other workers have studied the effect of varying factors which influence the strength such as water-cement ratio, cement content and quality of curing. Figures 2, 3 and 4 show the relationships between the abrasion of concrete surfaces and water-cement ratio (3), cement content (4) and curing (5) respectively. Further evidence is gained from British experience of using granolithic toppings for floors subject to severe abrasion. Recommended mix proportions are 1:1:2 by mass of cement:sand:10 mm aggregate; such mixes have a 28 day cube strength of 60 to 80 N/mm^2.

Coarse Aggregate

It has been shown (3, 6) that there is no correlation between the hardness or abrasion resistance of good quality aggregates and the abrasion resistance of concrete of high strength. Soft aggregates such as limestone and sandstone do reduce abrasion resistance and quality of aggregate is important in concretes in the range of 20 to 35 N/mm^2 cube strength. It has also been demonstrated (7) that malleable iron aggregates, which tend to smear over the surface matrix can give up to 300 per cent increase in abrasion resistance compared with similar concrete containing natural aggregates.

Surface Finishing

Bleeding and segregation of concrete after compaction often lead to the formation of 'laitence' on the surface which if left to harden would very easily be abraded away. The traditional finishing technique with plain and granolithic floors involves floating and trowelling to flatten and compact the surface mortar, reducing the effective water-cement ratio at the surface to produce a hard wearing concrete. The correct procedures for carrying out these operations are described in detail in a number of publications (8, 9, 10, 11) and are not discussed in this paper. The effect of trowelling on abrasion resistance is illustrated by the results obtained by Fentress (5) shown in Figure 5.

The timing of trowelling operations is dependent upon the surface condition of the plastic concrete and long delays, to allow for evaporation of bleed water, often cause quality control problems as well as increase in cost. Vacuum dewatering is a proprietary process which is used to enable trowelling to begin sooner after placing than would otherwise be possible. This process significantly increases

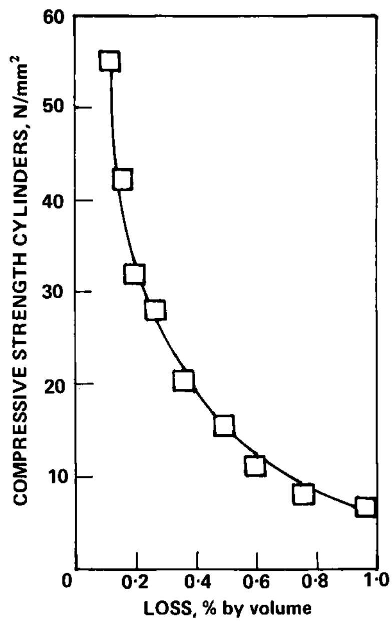

Figure 1 Relationship between Strength and Abrasion (Witte and Backstrom)

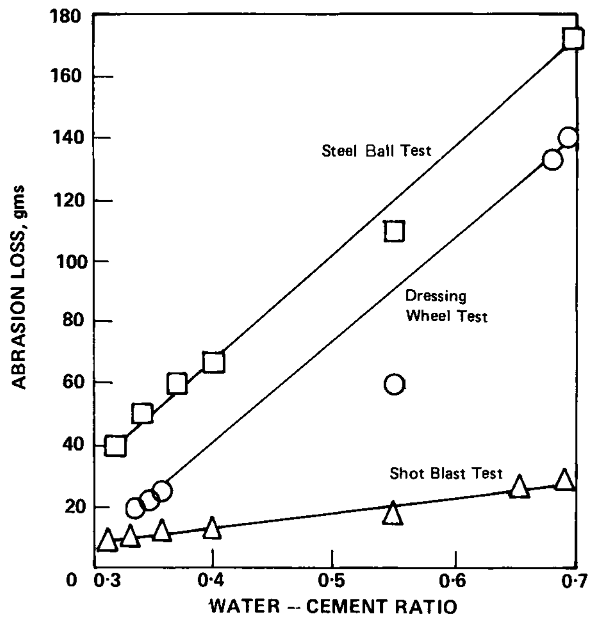

Figure 2 Relationship between Water/Cement Ratio and Abrasion (Smith)

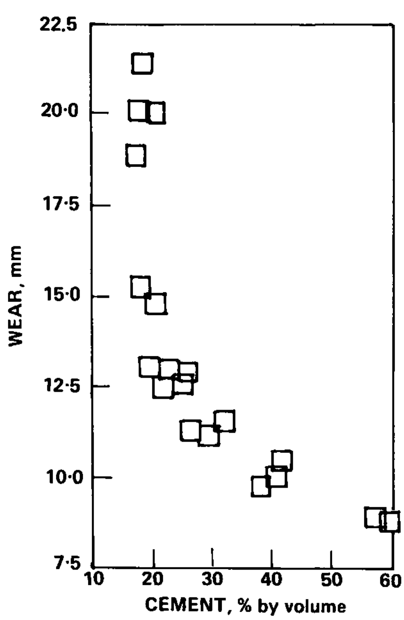

Figure 3 Relationship between Cement Content and Abrasion (Abrams)

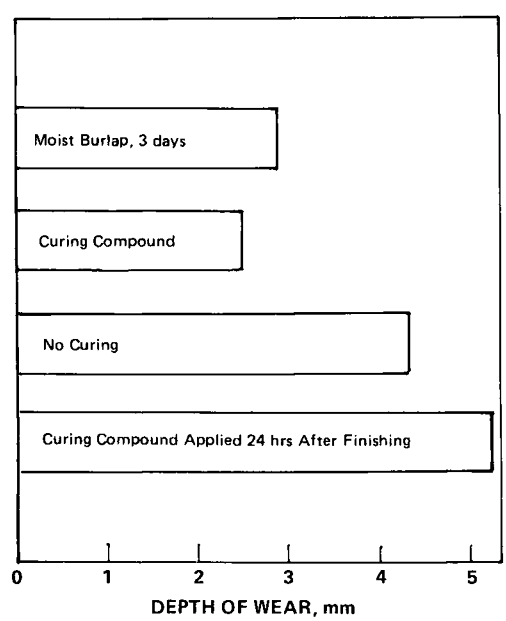

Figure 4 Relationship between Curing and Abrasion (Fentress)

abrasion resistance (12, 13) by reducing the water-cement ratio of the concrete near the surface.

The trowelling process may be replaced by early grinding at 36 to 48 hours after placing; this removes the surface laitence to expose the harder concrete underneath. Indirect evidence from the rebound hammer work, described later in this paper, suggests that early ground surfaces may be of similar hardness to trowelled surfaces at the high strengths, though not at lower strengths. Other special finishing processes exist but are not discussed in this paper.

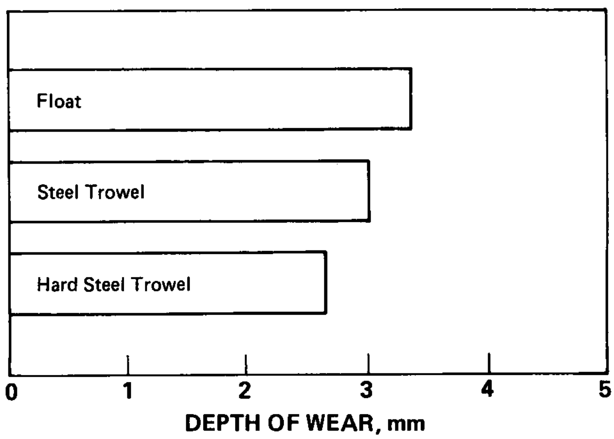

Figure 5 Relationship between Type of Finish and Abrasion

DEVELOPMENT OF AN IN-SITU TEST FOR ASSESSING ABRASION RESISTANCE

Although accelerated wear tests produce useful results, they do not relate to real wearing conditions and are difficult to apply in-situ. It is suggested that any apparatus for in-situ testing should be simple, robust, easily portable, quick to use, sensitive to factors influencing abrasion resistance and should not damage the floor surface. An investigation (14) of the abrasion resistance of vacuum processed concrete which involved the use of an N type Schmidt rebound hammer, in addition to accelerated wear tests, prompted a further study of the device in this context by the author. The results of this were encouraging and have led to the current research programme aimed at establishing the use of the Schmidt rebound hammer as a means of assessing abrasion resistance. Additional work has been carried out by Plimmer (15).

Description of the Equipment

The Schmidt rebound hammer, shown in Figure 6, was developed in 1948 and is normally used for assessing the strength of in-situ and precast concrete. The device is a sclerometer, or surface hardness meter, and consists of a spring-loaded hammer mass which impacts on the surface and rebounds to give a reading on a scale. A number of readings are taken in the test area and the mean rebound index calculated. This is converted, using a calibration chart, to an estimate of concrete strength.

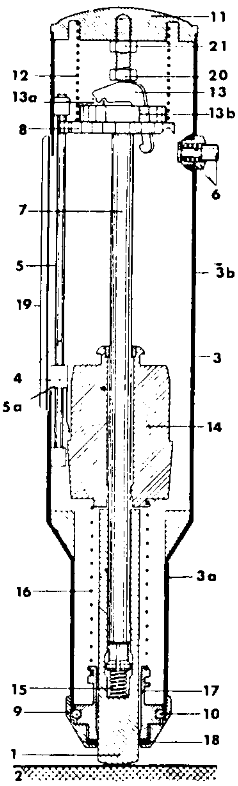

Figure 6 N Type Schmidt Rebound Hammer, (left) Diagrammatic View and (right) Readings being taken through a Positioning Template

Suitability for Assessment of Abrasion Resistance

The Schmidt rebound hammer enables concrete strength to be estimated within 20 per cent and it is evident that there must be a relation between rebound index and abrasion resistance as both increase with increasing compressive strength. The causes of the errors in measurement of strength have been attributed (16, 17) mainly to variations in surface texture, surface density, carbonation of the surface, moisture content and coarse aggregate type.

The potential suitability of the Schmidt rebound hammer is based on the knowledge that it is precisely these factors which lead to high or low abrasion resistance as indicated by accelerated wear tests in past research studies. It seems logical that there is a relation between the hardness of the concrete surface and its abrasion resistance and it is possible that mean rebound index may correlate better with abrasion resistance than with strength. The object of the current investigation is to establish the validity of this assertion.

Current Research

Test Method

The rebound hammer is calibrated on a special steel anvil and used on surfaces which are dry and free from dirt and grit. Variations in readings occur because of the presence of large aggregate particles or voids immediately under the impact points. To avoid bias in selecting impact points, a metal template is used to take a set of 12 readings on a rectangular grid at 25 mm centres, as shown in Figure 6. From each set, the highest and lowest readings are neglected. Three sets of readings are taken on each slab and the mean calculated to obtain a mean rebound index.

Experimental Technique

Forty test slabs, 1 m x 1 m x 150 mm, were cast in the laboratory. A standard concrete with zero slump was used for the bottom 110 mm and compacted with a poker vibrator. This was topped monolithically with concrete batched and mixed under laboratory conditions and compacted with a short steel beam vibrator. Most surfaces were floated and trowelled by hand and cured under polythene sheeting, but some were finished by early grinding and some surfaces were left uncured. The surfaces cured under polythene sheeting were uncovered at 21 days and allowed to dry before being tested with the rebound hammer. Three cubes were made from each topping mix and tested at 28 days in accordance with B.S. 1881:1970.

Objective

The objective is to establish the influence of factors known to affect abrasion resistance on the mean rebound index obtained from slabs finished by floating and trowelling and properly cured. Variables being examined are cement content, water-cement ratio, coarse aggregate type, quality of trowelling, quality of curing, cement-sand ratio and sand grading and type. To date, only some of these variables have been examined and much further work is required. For this reason the conclusions drawn are tentative.

Discussion of Results

Compressive Strength

The relationship between the cube strength of the concrete and mean rebound index of the trowelled floor surfaces at 28 days is shown in Figure 7. It is evident that cube strength correlates well with mean rebound index even though there are variations of up to eight points in rebound index for concretes of similar strength. It is probable that these variations are largely the result of differences in aggregate content and grading, variations in finishing, especially trowelling, and variations in ambient temperature and humidity under the polythene sheeting used for curing. There is some difficulty in maintaining similar conditions for all tests.

Cement and Sand Content

Figure 8 shows graphically the effect of changes in the cement-sand ratio of a series of mixes in which the total percentage of fine material is constant. It is clear from this that a very definite relationship exists between the quality of the matrix and the rebound index and this will be the subject of further study.

Aggregate Type and Grading

The effect of aggregate grading has not yet been investigated and only limited

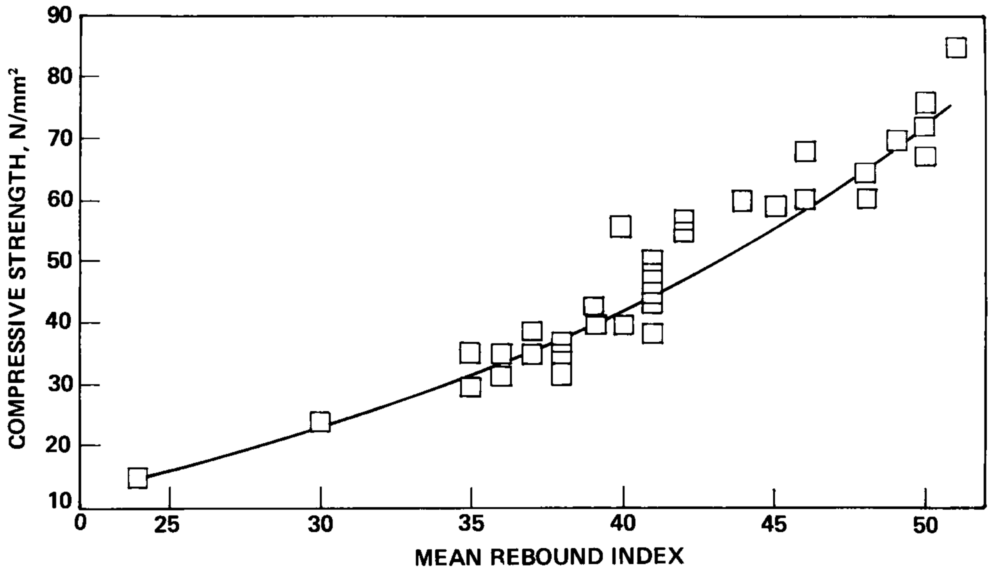

Figure 7 Relationship between Cube Strength and Mean Rebound Index of Floor Slabs at 28 days

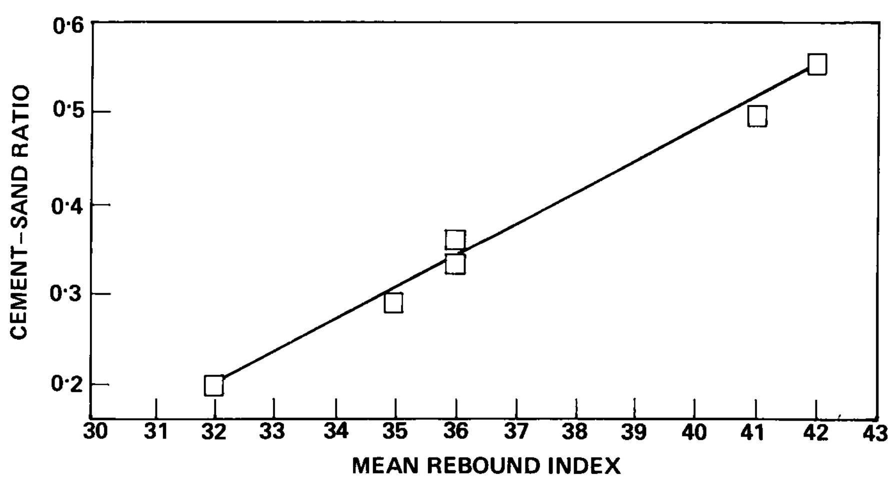

Figure 8 Relationship between Cement-Sand Ratio and Mean Rebound Index of Floor Slabs at 28 days

information has been obtained on the influence of the type of coarse aggregate. The results of a series of tests using four different coarse aggregates are given in Table 1.

Table 1 Effect of Different Types of Coarse Aggregate on Cube Strength and Mean Rebound Index

COARSE AGGREGATE TYPE	MIX A CEMENT CONTENT, 18%		MIX B CEMENT CONTENT, 13%		MIX C CEMENT CONTENT, 11%	
	Mean Cube Strength N/mm^2	Mean Rebound Index	Mean Cube Strength N/mm^2	Mean Rebound Index	Mean Cube Strength N/mm^2	Mean Rebound Index
Mount Sorrel Granite	85	50.8	43	40.8	33	35.9
Clee Hill Basalt	76	49.8	41	40.2	35	34.7
Thames Valley Gravel	70	47.7	38	40.5	36	38.0
Moorcroft Limestone	62	48.5	37	37.8	33	37.7
95% Confidence Limits		±0.8		±1.8		±2.5

In each set of mixes the mix proportions were kept constant. The differences in cube strength and rebound index of the lower cement content mixes are small although some of the differences in mean rebound index are statistically significant. In the high cement content set, the cube strengths vary considerably and it is possible that these represent the upper limits of strength for concrete containing these different coarse aggregates. The differences between the mean rebound indices in this set are all statistically significant but are more consistent than the cube strengths. This suggests that the rebound index is reflecting variations in the matrix rather than overall strength of the concrete. It should be noted that in a separate investigation Plimmer (15) concluded that rebound index was not significantly affected by coarse aggregate type. Again, this aspect will require further study.

Surface Finishing

The significance of trowelling is illustrated by two series of tests. Figure 9 indicates the difference between surfaces which have been floated, early ground and trowelled.

It may be seen that trowelling causes a considerable increase in the rebound index especially at the higher strength levels. Table 2 gives the results of differences in trowelling on slabs made from Grade 30 concrete and shows that an increase of up to 10 points in rebound index may be achieved by an additional two trowellings at hourly intervals.

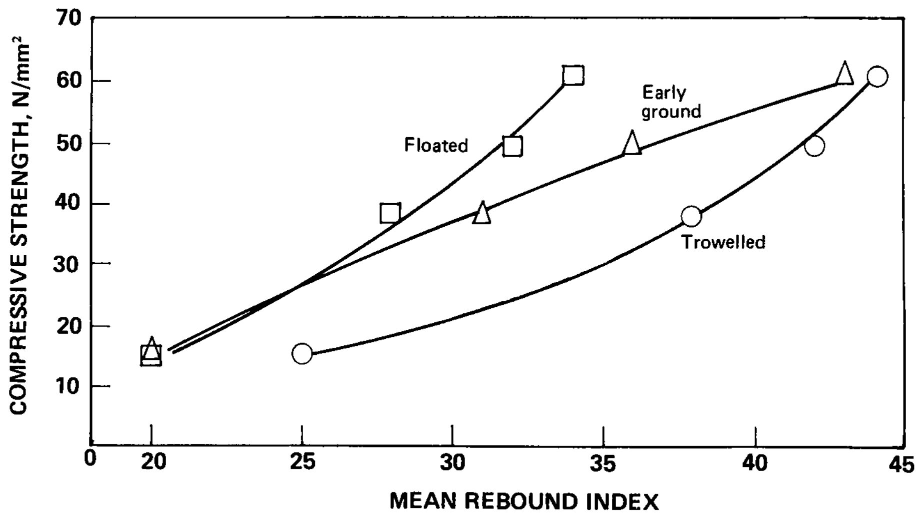

Figure 9 Comparison of Mean Rebound Indices of Surfaces Finished by Floating, Early Grinding and Trowelling

Table 2 Effect of Trowelling on Rebound Index

NUMBER OF TROWELLINGS	MEAN REBOUND INDEX
1	33
2	37
3	43
1 Delayed	39

Curing

Figure 10 shows the reduction in rebound index which results from a complete lack of curing after the first 12 hr; this agrees with Plimmer's conclusion that curing has a significant effect on abrasion resistance.

PROPOSED CLASSIFICATION FOR ABRASION RESISTANT FLOORS

A proposed classification system for abrasion resistant floors in industrial and commercial buildings with suggested grades of concrete, cement contents and finishing processes is given in Table 3. The limits for mean rebound index are derived from the strength relationship shown in Figure 7. The values given are based on the research to date and are tentative and liable to amendment.

Table 3 Proposed Classification for Abrasion Resistant Floors

CLASS	ABRASION RESISTANCE	MEAN 28 DAY REBOUND INDEX	GRADE OF CONCRETE	CEMENT CONTENT kg/m^3	COARSE AGGREGATE TYPE	FINISHING PROCESS	DUTY	APPLICATION
Special	Very high	Probably not applicable	60 +	-	Metallic or granolithic	Hand trowel or special process	Severe abrasion and impact	Heavy engineering
1	High	45 - 50	60 +	1:1:2 by mass	Granolithic	Trowel	High abrasion; steel wheel traffic	Heavy and medium industrial
2	Good	40 - 45	40 - 50	400 +	Good quality gravels and crushed rock	Trowel or grind at high strength	Moderate abrasion; dust-free conditions	Medium industrial; warehouses
3	Nominal	35 - 40	30 - 40	330 +	Any, other than soft limestone and sandstone	Trowel or grind	Foot traffic; pneumatic tyres	Light industrial; warehouses; commercial
4	Negligible	< 35	20	280	Any	Float, trowel or grind	May be surfaced with other materials	Offices; shops; schools; hospitals etc.

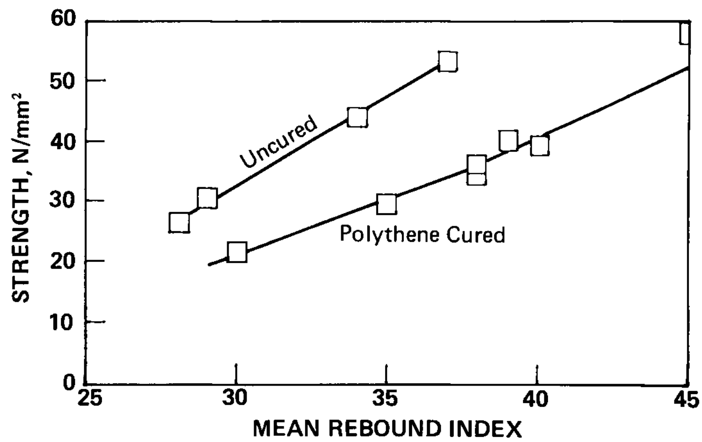

Figure 10 Comparison of Mean Rebound Indices of Surfaces Cured under Polythene Sheeting and Uncured Surfaces

CONCLUSIONS

The research work so far indicates that the Schmidt rebound hammer test is sensitive to changes in concrete parameters which are known to influence abrasion resistance. This is promising and suggests that the surface hardness may be more closely correlated to abrasion resistance than to any other measurable property and may well form the basis of a useful in-situ test. Much further work is required to confirm this conclusion and it may be desirable to make direct comparisons with some form of accelerated wear test.

The current state of knowledge indicates that compressive strength is the most significant factor influencing abrasion resistance but there are strong indications that it is the quality of the matrix at the surface which is of over-riding importance. The properties of the coarse aggregate are of lesser significance.

The application of current recommendations on specification and workmanship would remove many of the problems which relate to abrasion resistance of concrete floors.

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DIRECT FINISHING OF CONCRETE SLABS USING THE EARLY AGE POWER GRINDING TECHNIQUE

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ABSTRACT The present primary finishing methods used on industrial and residential floor slabs are reviewed. The disadvantage in durability, cost effectiveness and time taken to carry out the procedures are considered. Comparisons are made between the early age power grinding technique and the more conventional methods to demonstrate the improved quality and cost effectiveness which can be achieved, and the structural savings which can be effected.

INTRODUCTION

It is unfortunately true that over the years considerable confusion and many misconceptions have grown up regarding the behaviour of concrete ground floor slabs. Accordingly many designs and specifications are unnecessarily complex and restrictive and consequently in-built faults and premature failures are not infrequent. Nowhere is this more true than in the surface finishing of concrete floors. Since the most used section of any building is the floor, in fact it is the only part of the structure that is used for traffic, it is vital that a good durable finish is achieved. Unfortunately the indiscriminate specification and use of toppings and screeds, together with general malpractice and bad workmanship, have on many occasions been the cause of very disappointing floor performance.

The purpose of early age power grinding is the direct finishing of concrete floors to give a good durable and virtually dust free surface. Using this technique floors can be laid to very tight tolerances at the lowest possible cost. In this paper the method of finishing floors by early age power grinding is discussed in detail and comparison with conventional methods of finishing are made to demonstrate the advantages of power grinding.

METHODS OF FLOOR LAYING

Until recently it was normal practice in the United Kingdom to lay concrete floors using the chequer board method, usually in squares 12 ft x 12 ft (3.7 m x 3.7 m). One of the major disadvantages of the chequer board method was the large number of joints, both along the length and width of the slab, and failures at these joints frequently took place. It was also extremely difficult to maintain a perfect level between the various squares of the chequer board.

In recent years there has been a marked change to laying slabs in long strips, a method which has resulted in considerable benefits in economy. In addition the standard of finish which can be achieved is considerably higher, due to the elimination of transverse joints, and this method of laying is particularly suitable for the early age power grinding technique of finishing.

CONVENTIONAL SURFACE FINISHING OF FLOOR SLABS

Sand and Cement or Granolithic Screeds

It has been normal practice over the years to finish industrial floors with a granolithic screed to give a hard wearing level surface. Industrial floors which will not be subject to heavy traffic and floors in multi-storey residential buildings have usually been finished by a sand and cement levelling screed. The use of such screeds, when the preparation and laying work has not been carried out by an expert, is the cause of very many of the floor failures that take place. Ideally the floor should be laid monolithically but very often the work programme does not permit this; when separate construction takes place then unless the base course has been properly prepared there is always a grave danger of the screed curling and lifting and subsequently cracking and failing. In addition the screeds add considerable weight to the structure and since the normal screed is about 50 mm in thickness, on a multi-storey building they can add considerably to the height of the building. For these reasons the use of screeds can add considerably to the cost of the structural work.

Toppings

Although toppings can suffer the same disadvantages as screeds and should generally be omitted in favour of direct finishing, even with a good direct finish floor toppings will be needed, and should be specified, in conditions of severe usage. Floors liable to attack by chemicals, or which will suffer severe abrasion or impacts, should always be treated with specialised toppings.

Direct Finishing - Trowelling

The use of directly finished floors has been increasing in recent years, the normal method used being that of power floating or power trowelling. While this is quite a good procedure when used properly, it has some inherent dangers and always suffers from certain disadvantages.

Disadvantages

The power floating of concrete floors can only be undertaken when the initial set has taken place in the concrete but before the concrete is set hard (a rule of thumb method for deciding when one can power trowel is when the concrete is hard enough that a man standing on it can just get the impression of his heel into the concrete). If the work is done too early the concrete is disturbed. If the work is done too late then the power trowel has no effect whatsoever because the concrete is completely set.

Timing is therefore essential and it is usual to lay concrete only during the morning and to stop concreting some time between 12.00 and 14.00 hr in the afternoon. The operators must then wait until the concrete has gone off sufficiently before beginning to power trowel, and this can involve a wait of anything between

2 and 3 and up to 10 hr depending upon weather conditions.

The first disadvantage, therefore, is that during this period no concrete laying can take place and there is considerable cost involved in the equipment which is standing idle waiting for the next day's pour. The second big disadvantage comes from timing of the power trowelling operation which commonly has to continue, and may not even start until, after normal working hours. The operators therefore have to be paid overtime, possibly whilst waiting to get on the slab and also whilst carrying out the trowelling. It will be obvious therefore that this method of finishing is both costly in wages and costly in the loss of plant utilization.

In addition, due to the time involved in this method of finishing, the operators are often tired, frequently the supervisors have left, and very often unsatisfactory work is produced.

Dangers

The primary danger of power trowelling is that the operation will be started whilst the slab is still in too plastic a condition. As a result, and due to the rotating action of the blades of the power trowel, certain movement of the mass takes place. This movement results in a condition that unfortunately is frequently found in floors, viz. a medium or long wave undulation along the length of the slab.

EARLY AGE POWER GRINDING (ERT-GRIND)

Introduction

This is a technique for direct finishing concrete floor slabs to provide a hard wearing surface at very competitive cost. It is possible to provide a smooth level surface for the direct application of floor coverings, even very thin vinyl tiles, without the need for costly underlayment, or purely by the addition of cement to the concrete mix (i.e. increasing the minimum cement content), floors can be finished to give an extremely good, hard wearing and durable surface for heavy industrial purposes without the need for an additional topping.

The procedure used in the early age power grinding system removes the laitence from the surface so that a virtually dust free floor is achieved. An additional advantage is that the final surface, although very level and smooth, has a very fine sand paper texture, thus making the floor non-slip.

Construction Technique using Early Age Power Grinding

In the early stages, construction of the floor slabs follows normal practice. The concrete is placed between side forms, laid to line and level, and compacted initially with poker vibrators at the edges before overall compaction with a twin beam vibrating screed. Following compaction, fine levelling and smoothing of the surface should be carried out using an easy float or bull float. The object of this operation is to bring the surface of the slab to a smooth level, free of ridges and blemishes, and it is essential to the success of power grinding, which is intended to remove only the thin layer of laitence from the surface, not to remove gross irregularities. After floating the slab is left to harden and cure under sheeting as normal.

Grinding the hardened concrete must be carried out dry with a slow speed special purpose concrete grinding machine, if necessary, with dust extraction equipment, see Figure 1. The grinding is carried out as soon as concrete is hard enough to withstand the forces applied by the grinders without tearing the surface. This is normally between 2 and 5 days after casting depending on the nominal strength of the concrete and the curing environment maintained, although with very strong concrete (grade 40 or better), grinding frequently may be started within 24 hr of placing. Since the actual time of grinding is not critical the operation can be carried out whenever convenient within normal working hours, and the equipment does not require skilled or specialist operatives.

Once a smooth even surface with a glass-paper texture has been obtained over the whole slab, the grinding operation is complete and the normal curing procedures can be continued as required.

Figure 1 Concrete Grinding Machine with Dust-Extraction Equipment

Surface Accuracy using Early Age Power Grinding

The normal tolerances for in-situ floor finishes and for accuracy in building have been laid down by the British Standards Institute in publication C.P.204 (1970) and P.D. 6440 (1969). In each case the normal tolerances quoted are ± 3 mm under a 3 m straight edge and this tolerance is readily achieved using the early age power grinding system.

The technique of early age power grinding has been extensively used in Switzerland, where Swiss Standard S.I.A.134 calls for a surface exactitude of concrete of ± 2 mm in 2 m. This very exacting standard has been achieved without any difficulty using the ERT-Grind technique in place of traditional sand/cement screed, often at far lower cost and with the advantage of monolithic floor construction.

The Economics of Early Age Power Grinding

In addition to the economy of achieving simply and easily an extremely good floor to very tight tolerances, several other economies must also be considered:

1. There is no interrupted working. The concrete layers can operate all day, thus having full utilization of all site plant, while all finishing is carried out whenever convenient during normal working hours and under full supervision, thus eliminating all overtime costs for finishing.
2. Where thin floor coverings are to be used, levelling compounds are either eliminated or kept to an absolute minimum, and where hard wearing surfaces are required the addition of cement to the mix can largely eliminate the need for costly granolithic or other toppings.
3. On a multi-storey building an average of 50 mm (screed) can be saved per floor so that on a 15 storey building the height of the building can be reduced by 750 mm and this, together with lower floor weight, permits considerable economies in the overall structural design.
4. All the items of equipment used for the early age power grinding technique are low capital cost and can be used for other purposes when not in use for grinding thus giving it economies of plant utilization.

COMPARISON OF EARLY AGE POWER GRINDING WITH CONVENTIONAL DIRECT FINISHING

Comparisons are given below between power trowelling and early age grinding. No comparisons are given with hand laying as for any large area this is becoming a most unusual procedure. Likewise no comparisons are given with vacuum de-watering of concrete, a system which is excellent if it is really essential to have the concrete fully solidified within a very short period to enable services to travel over it, but which is only a specialised part of a finishing system. To achieve the rapid hardening of the surface the system requires the expenditure of a large capital sum on the vacuum de-watering equipment, mats, etc. which are costly in themselves and which have no other uses. The system automatically, from its nature, requires a lot more labour since one has to provide all the labour to lay and remove the mats to de-water the concrete and after that has been done the normal processes for power trowelling to finish the concrete still apply. Vacuum de-watering speeds up the process between the laying of the concrete and the initial power floating operation. However, once the initial power floating has been carried out, all the other items and delays associated with power trowel finishing still apply.

The economies of early age power grinding compared to power trowelling are achieved in three ways.

1. Working hours. To obtain a really good finish with power trowels it is normally necessary to make a minimum of 3 passes. Firstly there is a floating pass, followed by a waiting period of between 30 minutes and 2 hours until the first finishing pass can be made, then a further waiting period of up to one hour for the

final finishing pass. In fact, in many cases to obtain the required standard of finish more than 2 finishing passes are made, with additional waiting periods between. In addition to the time lost in waiting the actual trowelling operation is doing well if the operator can produce 15 m^2/hr.

With early age grinding, which requires less skilled operators than trowelling, it is possible to average between 40 to 50 m^2/hr; higher outputs have been, and are being, achieved. This means that the actual labour content of grinding is less than one third of that of trowelling, see Table 1.

Table 1 Comparison of Finishing Costs for Two Contracts each of 1000 m^2

ITEM	FIRST CONTRACT		SECOND CONTRACT	
	ERT-Grind	Power Trowel	ERT-Grind	Power Trowel
Machine Costs, £	799	476	-	-
Replacement Costs for Stones or Blades, £	240	460	240	460
Output, m^2/hr	50	15	50	15
Total Hours	200	666	200	666
Labour Costs, £	350	1332	350	1332
Total Cost, £	1389	2268	590	1792

Note i) Comparison assumes machine costs completely written-off against first contract, therefore no machine costs for second contract. This has been done for comparison purposes only, to show the increased savings accruing once machine costs have been written off.

ii) Labour costs have taken as £1.75/hr (semi-skilled) for ERT-Grind and £2.00/hr (skilled) for power trowel.

iii) No excess for the cost of overtime has been taken into account in the calculation of labour costs for power trowel.

2. Cost of replacement tools. The cost of replacing trowel blades compared to that of replacing grinding stones is very difficult to establish as rates of wear vary from concrete to concrete. On average, however, the cost of stones needed for power grinding works out to less than the cost of replacement float and finish blades, see Table 1.

3. Other savings. Mention has already been made of the structural savings which can be achieved by eliminating screeds and thus reducing the height and/or weight of a building.

In addition, the entire cost of a screed including the material and the labour are entirely eliminated. A further serious saving to be considered is that the avoidance of a screed means that a wet trade can be kept off site completely with all the advantages this gives to work planning and, for example, the avoidance of overloading of hoists, cranes.

CONCLUSIONS

The use of early age power grinding for the direct finishing of concrete floors makes possible the monolithic construction of high quality floors to very close tolerances whilst, at the same time, producing a safer and virtually dust-free concrete floor. The use of early age power grinding as the finishing technique for the final surface of the concrete results in reduced direct floor finishing costs. Other very considerable savings are achieved, including savings in the structural cost and in plant and man-power utilisation, which can give a significant cost advantage to the client and additional profit to the contractor. It is also notable that at a time when it is becoming more and more difficult to obtain good skilled labour, and more and more costly to pay such labour, the ERT-Grind System gives first class results without the need for the high cost of skilled labour — even if that skilled labour can be obtained.

THE APPLICATION OF VACUUM DE-WATERING TO IN SITU SLAB PRODUCTION

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ABSTRACT The use of vacuum de-watering is now a well established practice in the U.K. and Scandinavia during the construction of in-situ ground floor slabs, particularly in conjunction with the long strip method. The paper attempts to provide a resumé of current site techniques based on experience gained during several years as the principal suppliers of de-watering equipment and advising contractors on its use. Additionally, qualitative aspects of concrete subjected to vacuum and associated mechanical treatment during the processes of compaction, levelling and surface finishing are considered.

INTRODUCTION

The equipment designed by K.P. Billner for site application of surface vacuum to concrete slabs was first introduced commercially in the U.S.A. during the mid 1930's. In response to the recognised need for a reduction in the water-cement ratio of the mix on completion of forming processes for which a high degree of workability is desired, this de-watering process was found to be capable of achieving a significant improvement in compressive strength with consequent benefit in terms of surface hardness and durability. Although quite widely used for approximately 20 years, the practical limitations and cost of utilising the unwieldy rigid covers under which the vacuum was created in the Billner patented process led to a gradual decline of the technique in commercial practice.

Subsequent development in the late 1960's of simpler and less expensive equipment pioneered in Sweden arose from attention being given to the more practical consideration of saving time on site. Specifically the objective was to reduce the time required for comparatively high slump mixes to become workable with surface finishing tools such as the increasingly popular mechanical power trowel. In ambient conditions of low temperature and/or high humidity the delay between completion of compaction and levelling and commencement of trowelling may amount to several hours and with increasingly high labour charges, this was becoming a significant cost factor on site.

THE WORKING PRINCIPAL

Billner's process is based on the creation of a vacuum between the top surface of the fresh concrete and a cover which provides an airtight seal around the area of concrete to be treated. In order that water in the concrete can be drawn off, a means has to be provided for the formation of channels leading to a point outside the surface cover. This is achieved by introducing a mesh combined with a fine filter medium between the concrete surface and the cover. At the centre of the cover a connection is made to a vacuum source by means of a semi-flexible hose which also conveys water away from the slab to a suitable point of disposal.

Billner's process utilises semi-rigid preformed plates to provide the surface cover, whereas the more successful commercial systems in use today are based upon the use of flexible mats. Not only are these mats lighter and more adaptable, but they have made a significant contribution to the cost effectiveness of the vacuum de-watering process.

The action of creating a vacuum beneath the impermeable top cover results in the transmission of atmospheric pressure on to the surface of the concrete, with water (and some air) being forced to the point of least resistance, i.e. in the area of vacuum, and so movement takes place. In practice it is found that only approximately 20 per cent of the water contained in the mix is removed; the limitation is believed to be imposed by the progressive closure of capillary ducts along which water moves through the concrete. It will be evident that not only is water removed by the vacuum process but additional compaction also takes place. The inclusion of a filter material between the concrete and mesh prevents cement and fine aggregate particles being removed with the water from the surface.

THE EQUIPMENT

Three component elements are employed: i) a vacuum generator, ii) an airtight top cover and iii) a combined mesh and filter cloth.

Vacuum Generators

Three basic designs, with variable features, are to be found in use: i) generator with single combined vacuum and water storage tank, ii) pump with twin tanks and iii) continuous discharge units.

Single Tank Units

A vacuum is generated within the water receiving vessel either by a power driven pump or by means of a venturi linked to an independent source of compressed air. Water is drawn into the tank along the pipe connected to the top cover and when full, extraction ceases (if not previously terminated by the capillary contraction described above). In practice the size of tank is selected so that sufficient capacity is available to accommodate extracted water from a range of mat sizes covering up to approximately 35 m^2 laid on a slab up to 250 mm thick. Units of this type were the first to be introduced commercially. Although imposing limitations on the area or depth of concrete to be processed and being somewhat bulky, nevertheless, they are still popular. The type employing a venturi have the advantages of low capital cost and virtually no wearing parts.

Twin Tank Units

Models of this construction were introduced a few years ago in order to overcome the limitations imposed by the tank size on the earlier types. They featured intermittent discharge of water while maintaining vacuum in the remaining part of the system. Although achieving the objective of allowing larger areas of concrete to be treated, their engineering complexity, comparatively high cost and weight militated against general adoption on site and their use is mainly confined to specialists with a high rate of utilisation and who are more prepared to devote the necessary attention to essential cleaning and maintenance routines.

Continuous Discharge Units

The evident disadvantages of the twin tank models soon led to the introduction of a third generation based on a vacuum pump of the liquid ring type. This pump will handle both air and water effectively, so eliminating the need for the separation required when conventional vacuum pumps are employed. It also made possible the construction of a much more compact and portable unit. Operational experience with this type of unit is still rather limited, but they are expected to become increasingly popular with the more experienced and regular practitioner. The expensive and sophisticated liquid ring pump will undoubtedly require care in its maintenance and here again acceptance will greatly depend upon the willingness of users to give proper attention to cleaning and servicing.

The Top Cover

A flexible mat of heavy gauge reinforced plastic material is generally favoured for its light weight, ease of handling and adaptability to variable dimensional demands. In situations such as precast production, where there is a regular demand for predetermined areas to be treated, a rigid form manufactured in glass reinforced plastic or a similar material may be equally effective.

The creation of a vacuum beneath the cover results in atmospheric pressure sealing the edges down onto the wet surface of the concrete, so preventing air being drawn in. It is, however, equally important to ensure that air is prevented from being pulled in from any point below the concrete surface and to this end sound edge forms, well bedded onto the sub-base, should be used. Careful attention should also be given to good compaction of the concrete, particularly along the edges of the slab by use of an immersion vibrator. (A practice also essential to the formation of good joints).

Generally treatment will be applied to continuous areas of concrete. However, provision for service ducts and other unconcreted areas can be accommodated within the mat area by temporary masking with P.V.C. sheeting, or a similar material, to prevent air being drawn in at these points.

Combined Mesh and Filter (Sieve Cloths)

Before rolling out the top mat or positioning the rigid cover, it is necessary to place a series of sieve cloths directly onto the wet concrete, these cloths laid so that the filter element is in immediate contact with the concrete. As already described, these filters prevent withdrawal of fine cement particles with the water. A plastic mesh provides the necessary waterways on the surface so that movement can take place towards the central collecting area of the top mat to which the vacuum hose is attached. The size and number of sieve cloths to be used is determined by the dimension of the top mat and the need to leave an uncovered area around the perimeter on to which the mat can make direct contact with the wet surface of the concrete, in order to create the required airtight seal.

OPERATIONAL PROCEDURE

Contrary to the impression that may have been given by the necessity for a detailed exposition, the operations associated with use of the equipment on site are matters of only a few minutes work for an experienced team. As soon as an effective vacuum has been generated, the operators can return to other associated operations such as laying additional concrete or trowelling the previously treated area, since the unit will operate unattended.

It is not possible to be specific about the length of time required for water extraction, the degree of vacuum actually achieved, (between 500 mm and 700 mm Hg being operable levels), slab thickness and cleanliness of the sieve cloths being the principal determinants. As a guide, however, it may be taken that approximately 3 minutes should be allowed for each 25 mm thickness of concrete being treated. It has been shown that slabs of up to a little over 300 mm thick can be effectively de-watered.

For reasons stated earlier, there is a limit to the amount of water which can be extracted in practice. Consequently, there can be no risk of excessive de-watering. It should not be thought that sufficient water will have been removed for all subsequent surface treatment operations to be completed without further delay. The initial action of floating can commence immediately following cessation of the vacuum treatment, but sufficient time must then be allowed for further hardening to take place before final trowelling operations. The time required for the concrete to achieve this state will be dependent upon the rate of heat generation in the concrete and again ambient conditions will be influential as will the constituents of the mix.

CONCRETE MIX DESIGN

It is not usually necessary to make any change in the design mix to accommodate vacuum de-watering. In practice it has been found that any mix containing well graded aggregates can be treated successfully. In almost all cases where difficulties have been experienced both during and after the de-watering period, it has been found that an excess of undesirable fines in the mix is the main cause.

ASSOCIATED TREATMENTS

In order that the benefits of the process are optimised in both economic and technical terms, it is necessary to consider it in the context of the overall programme of slab construction. De-watering has to be viewed as one in a sequence of processes to which the concrete is subjected as a part of the good practice advocated by leading specifiers and advisory bodies such as the Cement and Concrete Association, whose recent publications (1-3) broadly deal with the vacuum de-watering system. From these and manufacturers' publications it will be seen that de-watering fits logically into a sequence comprising compaction, surface levelling (with further compaction),floating and trowelling. The high quality mechanical plant available today is capable of ensuring that these associated processes can effectively and economically complement de-watering. The desired end product, a high quality and durable floor, can be achieved from the structural concrete and forward looking contractors are also realising the benefits to be gained when compared with more traditional attitudes.

ECONOMICS

For the reason that de-watering should be viewed in the broader context referred to, no attempt has been made here at an economic evaluation in isolation. Such an exercise would have to take account of the relationship between plant capital cost and utilisation rate or hire rates, together with running and maintenance costs, all of which tend to be specific to the individual operator. Similarly the economies in labour utilisation made possible would show considerable variation according to site and operational circumstances, all of which would have to be precisely defined for accurate reporting.

CONCRETE QUALITY

The general effect of changes in water-cement ratio on the compressive strength of hardened concrete is well understood and requires no comment in this paper. However, investigations into the results achieved by vacuum de-watering and associated processes may be less familiar and the reader is directed to a number of publications (4-9). Drawing on these publications and the authors' experience gained during the preparation of a commissioned report from the Faculty of Construction Technology and Design at the Polytechnic of the South Bank, London (10), the following information and comments may be helpful.

Water-Cement Ratios in a Test Slab

Mass of Slab	3275	kg
Mix Proportions		
Aggregate	1245	kg
Sand	665	kg
Cement	330	kg
Water	175	kg
Initial Water in Slab	237	kg
Water Removed (18.4 per cent)	43.5	kg
Water Remaining	193.5	kg
Final Overall Water-Cement Ratio [(193.5 x 2415)/(330 x 3275)]	0.43	
Initial Water-Cement Ratio	0.53	

Effects of Transportation and Compaction

During the transportation stage between initial mixing and placing the wet concrete has a tendency to segregate and during subsequent vibration in the compaction operations further separation is unavoidable. The coarse aggregate moves towards the lower part of the slab while fines, cement and water move towards the top face. Consequently the upper part of the slab has a higher water-cement ratio than the interior and the lower section contains more coarse aggregate and has a lower water-cement ratio. Thus, the compressive strength at the top is less than at the bottom, the reverse of the desired state. The mixture of fines and water at the surface, if left untreated will inevitably leave a layer of weak laitance on the floor and have the familiar tendency to dust.

However, advantage can be taken of the cement enrichment at the surface consequent upon removal of the water using vacuum. The effect may be illustrated diagrammatically, Figure 1.

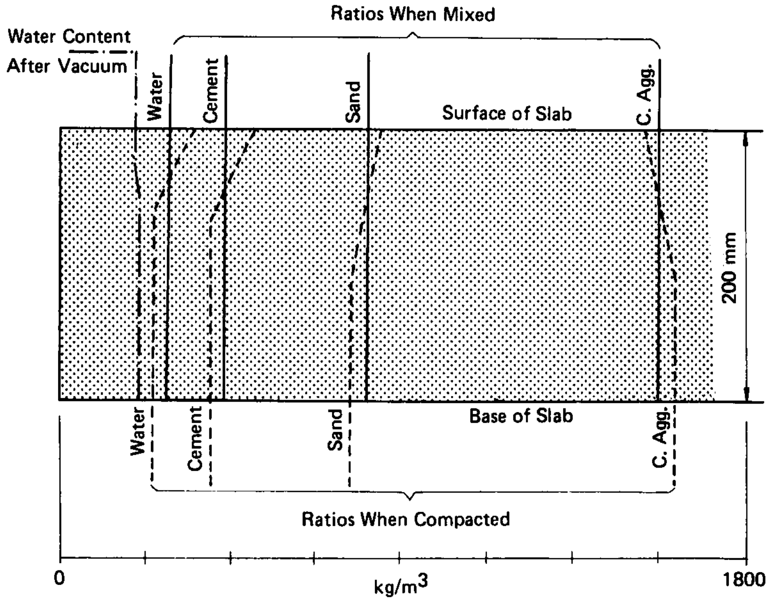

Figure 1 Change in Concrete Mix Proportions due to Vacuum Treatment

Surface Treatment after De-Watering

The subsequent operations of floating and trowelling on the de-watered surface, particularly when carried out with a light weight machine employing a disc for the initial treatment, have the effect of closing up the surface with the cement enriched paste produced by the earlier processing of the concrete. The benefits so derived can be demonstrated in terms of improved abrasion resistance and reduced permeability, both of which have been reported elsewhere (10).

CONCLUSIONS

Vacuum de-watering now has an established and proven place in the design specifications and construction of concrete floor slabs intended for industrial, commercial and warehouse applications. Associated with compaction, levelling and surface finishing tools, it enables the full benefits of long strip construction and direct finishing to be realised.

For the client there is the long term advantage of a quality floor slab capable of adaptation to a wide range of uses and treatment.

For the designer there is the greater surety of good concrete, well finished.

For the contractor there is the opportunity to fulfill the requirements of the specification with more certainty, meet timetables more precisely and make economies in labour utilisation.

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PLANT HIRE IN CONCRETE TREATMENT, PREPARATION AND MAINTENANCE

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ABSTRACT With the increasing shortage of skilled labour in the construction and maintenance industries, more and more firms are turning to the plant hire company as a means of ensuring output while reducing the effect of ever rising plant costs. The paper briefly outlines the small plant items available on non-operated hire which can be effectively utilised on concrete slab construction, examines the role of the plant hire company and discusses trends in the plant hire business.

INTRODUCTION

Over the last 20 years there have been relatively few innovations in the actual craft of building and general construction. What has happened is that the old skills have been mechanised, for example, instead of volume batching and mixing on a board, quality assured ready mixed concrete is now delivered to site and instead of hand trowelling slabs they are finished by power trowels. This change is in part due to the normal development of technology, but is also due to the drift of skilled workers away from the industry to better paid and more secure employment. This drift away from the industry is always worse during a downturn in construction activity, which is also the time when there is least money available for investment in plant to help overcome the shortage of skilled workers. In this climate it is becoming increasingly common for companies to turn to plant hire as a means of resolving these difficulties.

Plant hire is a growing branch of the construction industry; it is now possible to hire virtually anything from an offshore drill rig to a step-ladder. The plant hire industry can be divided in various ways: one is into the two categories of operated and non-operated plant, and it is with one subdivision of this latter category, the field of small non-operated plant, that this paper is concerned. The paper deals firstly with the items of plant which are presently available for use in slab construction and attempts to outline new developments in plant which are possible or desirable. Having described the plant it then goes on to briefly review methods of plant acquisition and to discuss the present and future role of the plant hire company in the construction industry.

PLANT FOR USE IN SLAB CONSTRUCTION

The equipment described below is not intended to be an exhaustive list of all the many items of plant which are available for the preparation and maintenance of concrete floors. It is intended only to highlight items of plant for basic operations which can thus be effectively carried out by relatively unskilled or semi-skilled operatives, more quickly and at lower cost than is possible by traditional means.

An indication of the increasing use being made of some of these items of plant is given in Figure 1, which shows the growth of expenditure by the author's company to enable it to meet hire demand for three of the most popular items in comparison to overall expenditure and turnover.

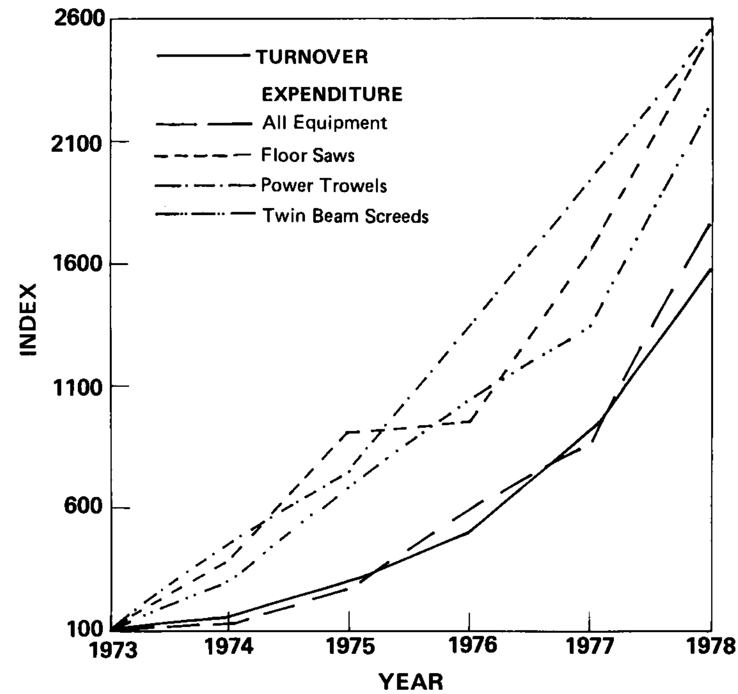


Figure 1 Scale of Turnover and Expenditure on Capital Equipment (Index Base (1973) = 100)

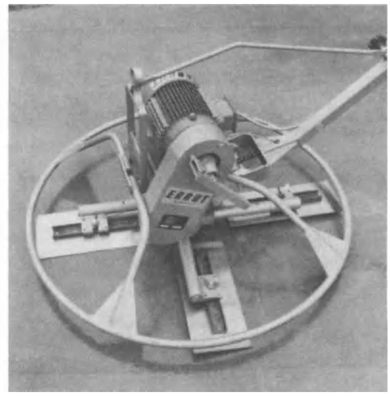
Initial Construction

Small plant items can make a significant contribution to the efficiency of slab construction from initial laying through to final finishing.

The twin-beam vibrating screed, Figure 2(a), mounted on aluminium beams replaces the conventional hand tamping beam, and performs the triple function of strike-off, compaction and levelling more rapidly and with less labour. The twin beam screed is light and self supporting and can be easily operated by one man.

This is also the case with the rotary power trowelling equipment shown in Figure 2(b). Interchangeable blades permit both initial floating and trowel finishing,

(a) Twin Beam Vibrating Screed



(b) Rotary Power Trowel

(c) Concrete Grinder

(d) Floor Saw

(e) Scabbler and Crack Cutter

Figure 2 Small Plant Items for Concrete Slab Construction enabling the very much slower, more skilled and thus more expensive method of hand trowelling to be dispensed with except for limited areas at the edges of the slabs.

As an alternative to power trowelling, concrete slabs can be finished using the early age power grinding equipment shown in Figure 2(c). Like the power trowel this requires only a semi-skilled operator and can produce a relatively dust-free surface which is sufficiently smooth and even to permit the direct laying of thin thermoplastic tiles or any other form of desired decorative finish.

One aspect of slab construction which has traditionally been both troublesome and expensive in terms of time and labour requirement is the forming of expansion joints. The concrete floor saw, Figure 2(d), can cut a clean and accurate expansion joint in one pass, and on some equipement the cutting blade can be replaced by a rotary wire brush to clean out the joint before sealing. In addition to cutting expansion joints the floor saw can obviously be used for any job that requires a neat groove to be cut in a concrete floor, for example, in preparation for underfloor heating or for the installation of electrical conduit. With this and the equipment available for drilling concrete, either rotary-percussion or diamond tipped core drills, it is now a relatively quick and simple operation to prepare a finished concrete floor for fixing down machinery or any other purpose.

Repair and Maintenance

It is probably true to say that from the moment concrete is laid a maintenance situation exists. Expenditure on repair and maintenance is steadily increasing, see Figure 3, and a significant proportion of this expenditure is on concrete floor slabs.

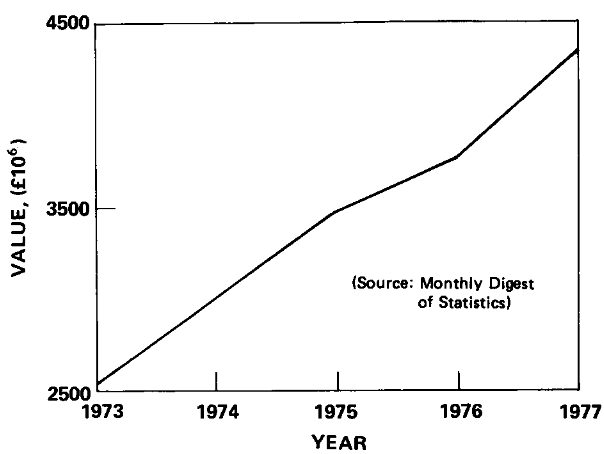


Figure 3 Trends in Value of Repair and Maintenance

The development of simple items of plant for scabbling surfaces to remove unsound material and provide a good key for the repair or to chase-out random cracks before filling, Figure 2(e), have helped to cut the time required and the cost involved in floor repair. Similar benefits can be achieved with concrete planers and retexturing equipment used to clean concrete surfaces and maintain the required finish, either smooth or skid-resistant.

New Trends in Plant

Over the last few years there have been relatively few new machines introduced in the category of small plant relevant to concrete slab construction or maintenance. Instead most of the development work undertaken has been in terms of modifications and improvements to existing equipment to meet customer demands for higher output and greater reliability. An example of this is the screed pump shown in Figure 4 which is capable of placing sand/cement screed at distances of 150 m horizontally or vertically. Originally manufactured with an integral compressor, it now uses an external compressor and has shown a marked improvement in reliability.

Figure 4 Sand/Cement Screed Pump

It is the author's opinion, however, that there is still a need for improvements in some aspects of equipment design, especially in respect of handling. Many items of plant are awkward and heavy to lift and lifting rings or handles are all too often absent. In contrast, sharp corners and other projections are too often present, probably because they cut production costs as well as hands. Much could also be done, and with the presence of the Health and Safety Executive may have to be done, to cut noise and vibration for the user. Too much equipment is made with the profit, not the operator, in mind. There are signs of improvement in this direction also, however: one manufacturer is now supplying concrete grinders with integral vacuum equipment to remove the dust at source, thus providing a cleaner working environment for the operator and incidentally increasing the efficiency of the operation since it removes the need for a second operation to clean up the dust.

Some trends are also apparent in the usage of plant items. One of the most notable is the increasing use of diamond tipped core bits for drilling holes as against percussion drills or rotary hammers, in many instances at the clients insistence due to the reduction in dust, noise and vibration. Paralleling the rise in popularity of diamond tipped core bits is the increasing change from abrasive to diamond tipped blades in floor saws. The large self-propelled diamond blade floor saw is rapidly gaining favour and today much more concrete is cut rather than chiselled when repair, maintenance or alterations are required.

The popularity of sawing concrete rather than chiselling it is likely to increase still further with the introduction of one new and novel item of plant, see Figure 5. Called the 'wall saw', and developed to facilitate the cutting of openings in reinforced concrete walls up to 375 mm thick, this machine once set up can operate unattended until the cut is made. Its application to cutting openings or grooves in floors is obvious as is the attraction of its ability to operate unattended.

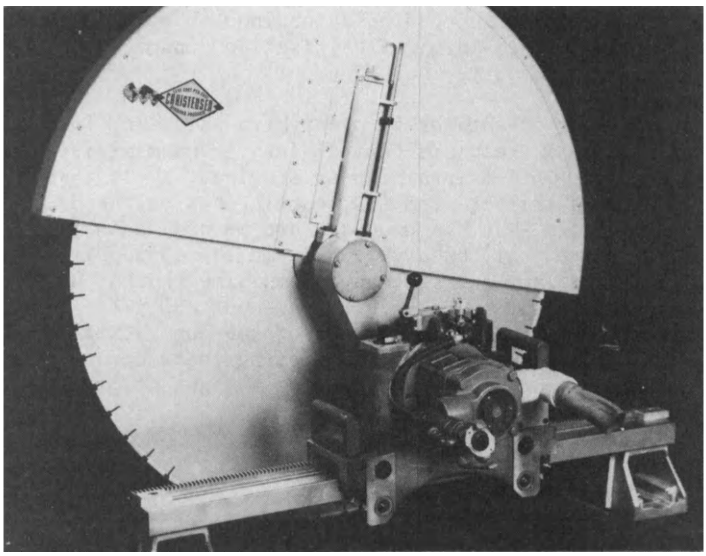


Figure 5 Wall Saw

THE ROLE OF PLANT HIRE COMPANIES

General

Before looking in detail at the role of the plant hire companies it is worthwhile to briefly outline the methods of plant acquisition available to an operator. Basically his choice lies with one of four options. Firstly he may choose outright purchase, the simplest option, which is a matter of obtaining the equipment at the lowest possible price commensurate with reliable after-sales servicing. The second option, hire purchase, is again a straightforward matter of paying for the equipment over an agreed number of years. Leasing, the third option, involves paying for the use of the plant over a number of years, possibly at a reducing annual or monthly fee, without ever actually owning the equipment. Finally there is the option of hiring the equipment at a given rate for some, usually limited, period. Generally, for the small items of plant for slab construction with which this paper is concerned, the second and third options are really inapplicable and the operator's decision becomes a straight choice between outright purchase or hire.

Relative Merits of Hiring against Purchase

The decision whether to buy or hire plant is always an individual one and unfortunately there is no reliable set of notes for guidance which may be followed to ensure that the correct decision is made in any particular case. In every instance it depends on the financial resources of the operator, his cash flow situation and his utilisation factor. Thus, if a company has unlimited money, the tax advantages and grants available may make purchase the more attractive option in all cases. More usually an operator, of more limited financial resources, will have a pool of equipment which he owns, his utilisation figure for which, in the author's opinion, should be on average in excess of 45 per cent, although clearly this figure will vary depending on company circumstances. If further plant is required and expected utilisation can be maintained above 45 per cent, purchasing should be considered; if this level of utilisation cannot be maintained, short-term hiring is probably the better solution.

There are of course other advantages to plant hire which may influence the decision. The operator can keep his capital intact and maximise his cash flow. Since he has no problems with maintaining or storing the plant when not in use he may be able to work from smaller premises, cutting his overheads. He can cost more easily since he knows what his on-costs are in bad weather or in strike situations. Being able to call on a virtually unlimited supply of capital equipment he can take on more extra work at any given time than he would otherwise be able to and since he can rapidly call on replacement equipment to keep output up if his own plant breaks down, the likelihood of sub-contractors being held up is minimised and he has a better chance of completing contracts on time. In addition the contractor may use plant hire facilities as a means of trying new equipment before taking a decision to purchase.

Hire Rates

Having made the decision to hire, for whatever reason, the operator should then be able to look to the plant hire firm to provide serviceable equipment in almost unlimited supply at extremely short notice. Obviously he must pay for this service and it is appropriate to consider briefly the question of hire rates.

In the author's opinion hire rates in the construction industry are too low. One of the reasons for this is the influence of the ill-advised plant buyer or finance committee. Accounting methods in the construction industry are such that their chief aim is to secure materials and plant at the lowest possible price, irrespective of the effect this may have on the overall cost of a job. For this reason it is easy to obtain plant at low hire rates, but it will probably be inferior, out of date and ill-serviced plant. The accountant may be happy with the low rates but the site agent has his frustrations and costs increased because the equipment does not meet his requirements.

If more realistic rates for plant hire could be set, the plant hire company could provide a proper service to the hirer. This should not only mean delivering equipment at the right time in full working order, but should include initial demonstration, where necessary, of all equipment and rapid repair or replacement in the event of breakdowns. By paying an upmarket price for hire plant, the hirer can exert considerable pressure on the hire company to ensure this standard of service. The amount paid on any contract for non-operated plant hire is a small fraction of the total risk cost and hire rates should be viewed with this in mind.

Future Trends in Plant Hire

Forecasting future trends in the plant hire business is just as difficult as in any other business. With restriction of capital and penal interest rates, every firm, whatever its size, is going to look ever more closely at cash flow and the best return on its money. There is no doubt that the booming rental business in the U.S.A. has influenced British thinking on hire. Comparison with the growth of the ready mixed concrete industry in this respect may or may not be appropriate, but it is most probable that the present growth in plant hire will continue.

Within the plant hire industry itself, although the small local firm can in many cases give a better, more personal service, the growth of larger firms covering the whole country through a network of local depots must be expected. Developments should include centralised accounting and improved administration to make it possible for customers to hire equipment anywhere at a standard rate, to bring it on and off hire at short notice at any convenient depot and to be invoiced for all transactions through any depot in one statement.

On the customer's side of the industry the single most useful development would be an effort to reduce the incidence of panic-hiring of small non-operated plant. At the moment most plant in this category is hired on this basis which makes it difficult for the hire company to give the best service. If the construction company would bring plant hire into its initial planning, then there could be a better appreciation of plant hire needs both in amount and timing and the hire companies could give a better service, including better hire rates. Hire in emergencies naturally will continue since this is one of the great benefits of plant hire availability, but not all emergencies should be unforeseen.

CONCLUSIONS

Every company and site has varying requirements in the preparation of concrete slabs. Modern equipment can often replace skilled craftsmen both by improved output and better workmanship. The decision on whether to hire or buy such equipment must rest with the individual because there is no certain rule which can be followed. However, if the equipment will have a utilisation of more than about 45 per cent it is probably cheaper to buy, lease or hire purchase.

Hire should be used to complement existing resources, in emergencies when labour may otherwise be standing idle, and when resources need to be conserved. Additionally it is often beneficial to use the hire company's resources to try out new techniques and equipment unfamiliar to the user. It is then much easier to make the right decision when purchasing.

With small plant, efficiency and reliability are just as important as with larger equipment and by using a hire company with safety and training facilities much effort can be saved. Hire rates should be judged with these criteria in mind and not just on which is lowest.

In view of the shortage of skilled labour in the construction and maintenance industries it is apparent that small plant can help to ensure output and the small non-operated plant hire company can help to cut costs.

DISCUSSION

Anthony R. Cusens. May I address a question to Dr. Browne. He presented a slide showing the change in strength in an epoxy bonded floor over a period of years; initially it dropped off quite significantly, with recovery occurring between two and three years. Can he give us any explanation of this particular phenomenon and could he also tell us if this information has been published anywhere?

Roger D. Browne. In answer to the second question, yes it was published in the proceedings of the Polymer Concrete Conference a few years ago. We never did identify why there was this increase. Dare I say it, it could be variation in the observation since we can never be sure that the samples were not taken from a more strongly bonded area. In fact, one has felt tempted to take some tests recently to see whether it has maintained the same level. I think the main thing one was surprised at was how persistently the bond had been maintained over a period of time. The initial drop off we felt was in some way due to the thermal stress cycling that had taken place as it was an external surfacing and daily cycling would obviously affect the performance of that material. So I am sorry but I cannot tell you specifically why we got that curve.

John D.N. Shaw. I am not sure what these results mean because I do not suppose anybody knows precisely the chemical composition of those resins used. You did not get results for epoxy, you got results for one single epoxy system applied in a certain manner with an undefined moisture content in the concrete, etc., so they are meaningless. We are not talking about ordinary Portland cement, we are talking about a very much more complex range of materials. I am afraid that engineers tend to take figures for one epoxy and assume that they are true of all epoxys. Epoxys are a tremendous range of different materials, some of which will stick under water, some will maintain properties up to 60°C, some creep at 20°C. There are so many different systems just to classify epoxy and polyester alone: was it an orthothalic polyester, an isothalic polyester, a crystal arrayed polyester; what was the styrene content, etc.? I doubt if that information is available at the time of use, let alone ten years after, so the figures are not very valuable.

Roger D. Browne. I think this just identifies the situation of the user today. If I may come back to your comment about my suggestion for a guidance document, my suggestion was not to produce definitions of the components, it was to produce a document in which we specify quite clearly the conditions that we require the materials to meet. We call these materials "poly what's its"; what do you use? You have heard this morning about a range of materials and if somebody has to decide what material to use, on the evidence of what we have heard this morning from the papers that have been presented, then he has got a major problem. What one would like to be able to do is to go to a document, and, whether one is an architect or an engineer, find a list of categories of materials covering different requirements. We then identify what we require, and since you may have requirements which we must meet, we may see that, for example, we must provide a certain grade of floor condition for your material. At the moment it is all too loose. I can understand the problem of the person on site or the person in the design office who can have to choose between thousands of materials. My point is that we should be defining the performance we require and you should be indicating what materials will meet those requirements.

Peter C. Hewlett. A point of information for Dr. Brown. In, I think, March 1978 there was published a document issued by F.I.P. entitled 'Proposals for a standard for acceptance tests and verification of epoxy bonding agents for segmental construction' and I think that much of the thinking on the themes developed in that draft document cover many of the points which you have raised. Can I be just a little contentious and say I do not believe your data until you define very clearly the method of failure for the various points on your curve. Whether it was an adhesive failure or whether it was a local failure, near the adhesive bond, but perhaps reflecting surface variation in the underlying mortar rather than any variation of bond in relation to the topping.

Roger D. Browne. From memory I think they all failed in the mortar.

Peter C. Hewlett. The epoxy mortar or the underlying mortar?

Roger D. Browne. No, in the actual epoxy mortar itself, but I would have to check back on that to be sure.

Peter C. Hewlett. Does it not reflect variation in the cementitious material rather than the performance of the organic?

Roger D. Browne. No, it was in the epoxy material itself.

Peter C. Hewlett. I am very surprised that it was in the epoxy based materials because you quote values approximately a third to a half of the normal tensile strength of cementitious mortar which must be 50 to 100 times less than the normal tensile strength of the epoxy mortar.

Clarrie Strain. Mr. Hodgson's paper was very interesting and I think it was very fair in its description of the added materials. My only impression from reading it is that while he has dealt with PVA and epoxy based materials, if you survey what is being used, for example, in the EEC, you find that neither of those could be considered as typical of the main usage. The materials that are given in a throw-away paragraph, such as acrylics, modified acrylics and, as you mentioned in your paper, the SBR latex, are in fact very popularly used. To give the impression that since PVA has inferior water resistance, as he freely admits and has always done so on previous occasions when he has given papers, you have to go to epoxys is not true. You do not have to go to epoxys, there are intermediate products which not only give an improvement in this particular factor, but can have other interesting benefits from each and every chemical kind. I do not think it was his intention to limit presentation to these two materials, but I think it should be made clear that if anyone is interested in this subject they would have to read a little more deeply or talk to a lot more suppliers in order to get a really balanced view of the situation. Perhaps Mr. Hodgson would like to comment.

I will just say one other little thing about Mr. Shaw's paper. I have been laying floors or toppings for a good many years and the point he made about chemical resistance being a very difficult thing to assess is one which, of course, has dogged me for very many years and I am equally as confused. However, I am fairly clear in my own mind that the solution to this problem will not be found by laboratory testing, but lies with us being able to design floors in such a way that the materials that attack the floor surfacing do not stay there very long. It is almost impossible, in fact, to get anyone to design a floor slab to a satisfactory fall and actually give you anything near what you have asked for. This is because of the incompetence of site work. For us to demand invert levels such that the drains are not tanks for holding corrosive liquids is pointless when one inch in two miles seems to be an acceptable fall to some people. All of this is part of the whole design concept and if we can design floors at all there is no reason why we cannot give corrosion resistance properties which are very, very good. Having had only one or two opportunities to achieve this, in my lengthy career, I have not been very successful. I have equally been unsuccessful in other cases when certain strictures were put on what I could do, and foolishly one does the best one can within the terms laid down, and of course the qualifications put on the job bring disrepute to the actual product. These are my personal views and I would like Mr. Shaw to comment.

Martin E. Hodgson. Yes I agree that there is quite a range of materials available and they all have their particular advantages and disadvantages; I tried not to go into that too much since some of them were mentioned in the other papers. There are the SBR's, the acrylics and, especially in Germany, I believe, a lot of vinyl propinate is used and these give quite fair water resistance. However, I think the epoxys give the ultimate in performance, and I mean here the 100 per cent epoxys, the epoxy plus filler sort of system. The water based epoxys are relatively new, at least in the U.K. Although they have been used in the U.S.A. for ten or fifteen years quite successfully, we only introduced them into this country three or four years ago. They offer, I think, most of the benefits of a straight epoxy, there is not quite such good chemical resistance because once you start putting cement into a system you lose some of the chemical resistance, but they are useful and are easy materials to work with.

George Barnbrook. I was very interested in Dr. Feldmann's paper on smoothing compounds. It is an area which is not reported on very much at present in this country. These products are used to remove an unacceptable texture or roughness from a floor surface which is to take flooring materials. The roughness of floors is very often a problem with some of the cement and sand screeds in this country. I would be very interested to hear from Dr. Feldmann of his experience in Germany of the percentage of floors where screeds are used and also where compounds are used on the concrete itself to receive floorings directly.

Dieter Feldmann. I have to say that normally in Germany cement-sand screeds are used in nearly 80 per cent of all cases but they are still too rough for flooring materials and therefore they have to be screeded with a smoothing compound. In large offices or buildings like that the concrete itself is often smoothed with a smoothing product. In these cases cement-sand screeds are not used, basically when there is no need for insulation against heat or noise which you can only achieve by means of the floating screed.

Pritpal S. Mangat. I would like to aim a question at Mr. Hodgson. The theme is based slightly on the question asked by another gentleman, that acrylics and styrene-butadenes can do the job fairly effectively just like epoxys. I would like to ask firstly if Mr. Hodgson can tell us more about the economics of using epoxys instead of polymers and secondly if he has done any work on the rate of gain of strength using epoxys instead of polymers. Is there an increase in the rate of gain of strength or does the epoxy tend to slow the hydration of the cement?

Martin E. Hodgson. Yes the curing system is a little bit different with an epoxy plus sand and cement. One has the curing mechanism of the cement and the curing mechanism of the epoxy both proceeding at the same time and to some extent independently. The epoxy will cure relatively rapidly. We say that if you lay a floor screed it can take foot traffic after twenty-four hours under normal ambient conditions and heavy wheeled traffic after three days. What effect epoxys have on the cement set I do not honestly know because the two mechanisms are intermingled and as far as I know nobody has ever tried to separate the two effects.

As for the economics, the PVA I think must be the cheapest, at a materials cost for 6 mm thickness of about £1.50 per m^2, whereas a water-based epoxy will work out at somewhere around £4.50 to £5 per m^2. Acrylics and SBR's must come somewhere between those two I think. There is the labour cost to add on of course, which is always difficult to quantify although I can quote one example for the water-based epoxy. We did a dairy floor a couple of years ago where a nominal 6 mm screed was laid, although in fact it was nearer 12 mm by the time it was finished because the floor was so badly eroded by water. The cost we worked out came to £8 per m^2 all in, materials and labour. We compared that with the other floor finishes that would have had to be used in that dairy which at that time ran from about £15 to £30 per m^2. So we came out cheaper on that comparison, but it was more expensive than the lower performance materials, such as PVA in particular, would have been.

John D.N. Shaw. Well basically the difference in base raw material cost between PVA's and acrylics and SBR's is only 10 to 15 per cent. In terms of the epoxy resin systems, on 100 per cent solids, forgetting the water content, you are talking about a factor of 4 over PVA. In the food industry, in the U.S.A. in particular, acrylic systems give higher early strengths than even epoxys and in many food processing industries in this country, meat processing in particular, acrylic floors are laid in a weekend as I said in my paper. I did try and give some indication of applied costs which are fairly realistic of specialist contractors' costs. I would certainly question whether a laid cost of £8 per m^2 with a system based on a high cost epoxy binder was realistic today.

Moving on from economics to health and safety, epoxy resin emulsion systems, like all epoxy resins, tend to be dermatitic products; because they are emulsions in water they are often treated like emulsion paints and workers forget that they are materials which can cause skin problems. In the slides shown by Mr. Hodgson there appeared to be a total lack of handling precautions which would be likely to give problems if the operatives were handling the products on a regular basis.

R. Colin Deacon. I would like to ask Mr. Shaw if he would agree that where coatings are going to be used, or are anticipated to be used, then the specification for the floor finish should very carefully consider the finish that is to be applied. For instance, would he agree that a power ground surface to a concrete floor or a power floated finish are very much more preferable than a power trowelled finish?

John D.N. Shaw. For any surface finish on concrete, a good clean mechanically sound concrete surface is required. The more sophisticated finishes such as epoxy resins do not require a heavily tamped finish or other means of producing a macro-mechanical key. They achieve bond chemically and all that is required is a clean dry surface without any weakness such as laitence. A power ground surface would, therefore, be very suitable. Good, well compacted concrete surfaces achieved by power trowelling (at the right time) in theory require no preparation since epoxy binder systems and SBR systems give a very good bond even to clean glass. In practice, however, most specialist flooring contractors do lightly scabble or acid etch the substrate prior to laying heavy duty resin finishes.

The penetrating 'in surface' sealers based on epoxy and polyurethane resin solutions or on fine particle size polymer emulsions exhibit good penetrative properties and with this type of finish preparation is not critical. In fact, they are often successfully used without any preparation to bind the surface of concrete floors prone to dusting.

Alan A. Lilley. I really want to direct my observations and questions to my colleague Mr. Chaplin. Perhaps I can take our minds back to Tuesday morning when Mr. Austin, the President of the Concrete Society said, and I hope I am paraphrasing him fairly, that if you accept bad concrete, you will get bad concrete. I think this comment applies probably more to floors than to any other part of a building. I also believe, however, that poor quality in terms of finish and abrasion resistance and dusting can also be attributed to poor specification. We see words in specifications such as: 'this floor shall be dust free and abrasion resistant'. This type of clause to my mind is quite meaningless. It leaves the contractor, at the tender stage, to ask one of two questions: what does it mean or what can I get away with? I believe that we must have a way of defining and quantifying in a meaningful and simple way, what we mean by dust free and abrasion resistant. In this sense I am very much a supporter of Roger Brown, I think he was on the right tack in his keynote talk this morning.

We have had to accept bad floors in the past, not through lack of knowledge but through lack of an ability to measure quality. Ralph Chaplin demonstrated quite clearly this morning, I thought, that as long as our cement content is right, our water content low, we have good compaction, firm trowelling and good curing, we are almost by definition going to get a good floor. We have known this for twenty-five years; we still do not get good floors. What Ralph Chaplin is supplying us with is firstly, a method of measuring the quality of the floor once it is completed, that is very important, and secondly, a method of classifying floors so that we can both specify the floor quality and test the end product. I know that Mr. Chaplin has been continuing his research since he wrote his paper and I have to ask him two questions. Has he any idea at what frequency he would envisage testing being carried out? Would it be one set of tests every X m^2 or on every construction strip or on each day's work? If a floor is shown to be below par does he think that the rebound index, which seems to be quite a valuable guide, could also be used as a means of providing a test regime to assess the benefit of any process used for treating a floor to bring it up to a specific standard?

Ralph G. Chaplin. In answer to the first question I have not really given any consideration to the frequency at which one would apply such tests because we have not as yet established that the test is a satisfactory one. We are on the way to establishing that and I think when we have done so we will then give consideration to testing frequency. I think that obviously one cannot test every floor slab over every square meter. I think one would have to select some areas for test which perhaps appeared to be unsatisfactory and some that appeared to be satisfactory in terms of the subjective tests which one does at the moment. Often a floor is tested by scratching it with a coin. I am not sure that you get very much information from that, but if you are unable to scratch a floor with a coin then you would be reasonably certain that it was a good floor; if you were able to remove a considerable amount of material by scratching then perhaps you would want to test it in a little more detail.

Moving on to your second point, I am sure that what the tool is doing is measuring surface hardness and any treatment which improves the surface hardness would show up with the device. However, I think these are really questions which remain to be investigated. I would first of all like to establish that the tool is a useful device and can be compared with accelerated wear tests before we take it any further.

Marion Chatterton. In connection with the de-watering process, I notice that the test slab started off with a water-cement ratio of 0.53 and finished with a ratio of 0.43. It is not particularly difficult to achieve concrete with a water-cement ratio of 0.43 in the first place, and so I must ask what would be the difference in the final result if instead of the de-watering process, which is probably quite expensive, one started off with a 0.43 mix which can still be fairly easily placed.

Colin Sankey. There is no reason why you should not start off with a 0.43 mix of course. First and foremost, when one talks about the expense of the system I would like to defray any sort of fears about this. This is not an expensive system, it is quite a cheap system and indeed perhaps one might consider it as just a question of a few pence per m^2. The unfortunate part about the concrete for normal slabs is that one has to consider the operatives as well as the end result. Consequently what happens is that the concrete which arrives on site must be in a reasonably workable state and often, as I pointed out, due to weather conditions, ambient temperatures, low cement contents etc., that particular concrete will stay in that particular state for a long period of time. All we are doing with the de-watering system is accelerating the stage from placing, using the facility of a reasonably workable mix, to the stage where they can start the finishing processes considerably earlier. I do not know whether that really answers your question.

Ravindra K. Dhir. I am not so sure whether, in fact, it answers the question which was posed by Mrs. Chatterton but it certainly raises another question and if I could put that question to you, Mr. Sankey. Are you implying or suggesting that it is not possible in this day and age to produce a reasonably workable mix with a water-cement ratio of 0.45 or 0.43, whatever the case may be?

Colin Sankey. I am not suggesting that for one moment. I am suggesting that it is more the case that mixes of higher workabilities than 0.5 are found on sites today. We do quite often reduce the water content to even less than 0.43, to virtually a no slump mix sometimes, so that you have a concrete that has gone, within a very short period of time, from a placing to a finishing situation, probably within half an hour to three quarters of an hour, regardless of ambient temperatures or weather conditions.

Nils Petersons. I would like to make a comment on the vacuum treatment system. Usually vacuum treatment is used to improve the surface quality only, although it also enables earlier finishing, but I must mention that the strength of the whole slab is also increased. The mean value of strength for the whole depth is increased by about 10 to 15 N/mm^2. Now we have developed a simple testing system which can be applied during the vacuum treatment which shows that this strength increase has been achieved and because of that it is possible to utilise the strength increase for design purposes. It is possible to buy concrete of one or two grades lower quality and apply vacuum treatment to increase the strengths. When designing, you can use the higher strengths at lower cost.

B.W. Kitching. That has half answered what I was going to ask. I would like to get a better idea of what the vacuum process does than I have at the moment. I think you indicated, Mr. Sankey, that all it did was speed the rate at which one could do finishing. I think the last contributor suggested that it actually reduces the water-cement ratio effectively, near the surface at least, and so improves the mix. Presumably it also gets round the practical difficulty that whatever your design mix theoretically is, in practice it is extremely difficult to maintain an accurate water-cement ratio. Presumably the process does achieve a greater accuracy of the ratio at least near the surface. How deep does this effect penetrate?

Colin Sankey. This is a surface applied system and consequently it is more effective near the surface than it is down at the bottom of the slab. What we do, in fact, is to place onto the fresh concrete surface a series of fine filter cloths, using a layer of 'Netlon' in between to provide a water course between the filter cloth and the impervious sheet. The filter cloths themselves are about 100 mm shorter than the actual width of the bay and around the filter cloths we leave a perimeter of wet concrete in which we seal this impervious sheet. On our particular system we then have a compressor that blows air through a venturi causing a vacuum in the cylinder. Release that vacuum and it causes suction, and obviously the air and the water immediately on the top of the slab is evacuated. Since the top impervious sheet is sealed around the perimeter, air cannot get in around the outside and consequently we create a vacuum in the area at the top of the filter mat allowing atmospheric pressure to press down onto the surface, accelerating the proportion of free water coming through to the surface and discharging through the pipe into the cylinder. So obviously being surface applied equipment, it is more effective near the surface. We have used this on slabs up to 200 mm thick and obviously on slabs of this thickness, provided they are well compacted with a poker vibrator, the compressive weight of the concrete is forcing the water upwards to the point where the machine is becoming effective so that it will in fact effectively de-water the whole depth, firstly with the assistance at the bottom of the compressive weight of the constituents of the mix and secondly by the suction on the top.

Brian Hayes. I want to address a question to Mr. Chaplin. At one point in your paper you were presenting information about the effects of finishing on the relationship between strength and the rebound index which you were proposing as a measure of abrasion resistance. I was quite surprised that in the middle range of strengths, probably 30 to 50 N/mm^2, early grinding was not giving the sort of change of rebound number that I would have anticipated. You rather skirted round that during your presentation, perhaps demuring to pass comments on a treatment which is going to be dealt with later, but I find that quite surprising. It seems a very relevant strength range. Are you saying that from your results, at the very highest strength range early grinding seems to be achieving the same effect as trowelling? What I am interested in is not only the actual results but why they came out as they did? I am quite surprised that these disparities exist and I would like some comments on the mechanisms which are at work.

Ralph G. Chaplin. I will give you my thoughts on how early grinding and trowelling work. I think that basically in a concrete you have an intrinsic strength. The bulk of the concrete will have a particular resistance to abrasion because of that strength. If one did not trowel the surface then one would have a top layer, a few millimeters thick, of laitence, in other words a very weak surface layer. The skip floating action used as a stage in early grinding leaves that layer, which is then removed by early grinding to get down to the mass of concrete underneath which has that certain abrasion resistance. Now in my estimation the trowelling process, when it is done properly, converts that potentially weak laitence layer into a case hardened surface, in other words the top three millimeters or so of that concrete is significantly harder and stronger than the rest of the concrete. The lower value on my Figure was concrete with a strength of approximately 15 N/mm^2. By trowelling that concrete you could slightly increase the abrasion resistance, or the hardness as we are measuring with the Schmidt hammer. In fact, however, the laitence on the surface, in its original state, probably is not very much different from the rest of the concrete at that particular strength level. I think that as strength increases, then perhaps because there is more cement there, the trowelling can have a greater effect. In other words, the higher the strength of the concrete, the greater is the effect of trowelling. However, I must emphasise that these results are only based on one set of floor slabs so one must not read too much into this. For very high strengths, early grinding will probably be just as good as trowelling; at lower strengths it may not be, but as I say, it was not really my purpose or intention to comment on these different finishing processes, especially early grinding, because I am trying to establish whether the rebound test is a proper test for assessing abrasion resistance. I do not think you can do the investigation in reverse.

John M. Rolfe. Coming back to the vacuum de-watering process, it seems to me that the mechanism is to withdraw water from the surface layer of the concrete, precipitating the development of the voids which normally develop later due to evaporation. The power trowelling after de-watering then closes these voids giving a more compact and denser concrete near the surface, with obvious benefits of reduced water-cement ratio and good compaction. However, it also seems that it produces layers of concrete with perhaps different shrinkage qualities. Is there any evidence that vacuum de-watering leads to aggravated curling at a later stage?

Colin Sankey. No, I do not have any evidence on this particular subject. I know that this has occurred, but I think it would be more true to say that this was perhaps not through misuse of the equipment, although it could have been used better to avoid this. If you have filter cloths that leave too large a perimeter around the slab, although the water will obviously migrate, to a certain degree, horizontally as well as vertically, if you leave too large a wet strip then you will get differential shrinking at that point which could cause curling to take place. If the job is done properly, however, then I see no reason why curling should be a problem. In fact, we have applied this particular system to bonded granolithic toppings and eliminated a lot of problems because of the removal of a large portion of the free water from the mix itself.

Peter C. Hewlett. A question to Mr. Chaplin. Do you intend extending your studies to look at the consequences of abrasion, such as the worsening slip resistance of floors when subject to abrasion, and whether there will be a correlation between wear, abrasion and slip resistance?

Ralph G. Chaplin. At the moment I have no plans to extend this work to that area. I think it is a very complex subject, the skid resistance of slabs is a very complicated phenomenon and certainly there is a complex interconnection between that and abrasion resistance. At the moment my interest is solely in attempting to establish a simple test which can be used on site to give some idea of whether a concrete slab will stand up to abrasion. At the moment we do not have any kind of test which can be applied in the field. In fact in the past we have very much resisted doing work on abrasion resistance of floors because there has been so much work done in the United States which is very difficult to interpret. I feel that we have already established the basic principles. Our interest at the moment is in establishing this simple test. What we do after that I really could not say.

David L. Cope. My point really is an aside to Mr. Chaplin's main theme, indeed I think he has shown that the Schmidt hammer is starting to look like a useful means of measuring the surface hardness of concrete. During his presentation, however, he said that it is not a useful tool for measuring the compressive strength of concrete because it is only accurate to within 20 per cent. I would like to point out that consideration of Figures 10 and 11 in his paper suggests that the accuracy is more likely to within 150 per cent.

Ralph G. Chaplin. I think you have to remember that in those particular Figures I was comparing the rebound values of a floor surface with cube strengths. I certainly think that we would not be able to use this test to determine whether a particular grade of concrete has been used in a floor slab because you can so easily increase the strength value by the way you trowel or cure the surface. So certainly in terms of trowelled and cured floor slabs you cannot use it to assess strength directly. What I would be trying to establish is whether, if the standard of curing and trowelling, etc., is maintained relatively constant in the laboratory, one can establish a relationship with strength. I agree that if a relationship can be established it will probably be a vague one.

David L. Cope. That was really the point I was trying to establish. What normally happens is that when the panic goes up when cubes fail, people say let us see what concrete you have got in here by using a Schmidt hammer. In those circumstances nobody has any idea whether it has been trowelled, how it has been cured, etc., so the sooner we get away from the idea of using a Schmidt hammer as a way of assessing concrete strength the better.

Sulaiman K. Danladi. Mr. Fairweather, you recommended a period of between 36 hours and 48 hours for the use of your power grinder. Is that recommendation related to any ambient temperature conditions? My question is posed because in my country the gain of strength you get in concrete after 36 hours is substantially higher than the gain in strength one gets in countries with cold climates, and I suppose that your tests have been done in British temperate conditions, i.e. cold weather conditions. Do you therefore still recommend 36 hours to 48 hours for the use of the power grinder in all climates or will you agree with me that one could use it preferably earlier, say after 12 hours in the tropical hot climate countries of Africa?

Ernest S. Fairweather. It will depend very much on the strength of the concrete that you are putting down, but one of the most important things about early age grinding is that you are not bound by time. As I said in the example I cited, the site agent from Laings had actually delayed grinding by a month and had had no serious effect. However, I would agree that in any tropical country, unless the humidity has affected it seriously, you should grind probably at 12 to 18 hours. This is really a question of adapting to local conditions. In Norway, where it is used extensively, I know that grinding is usually delayed for about three or four days. Certainly in Nigeria I would recommend grinding at 12 to 24 hours.

Maurice Levitt. Firstly I think that the aside contained in Mr. Lilley's earlier question was quite unforgivable. I can quote the cases of Taylor Woodrow Research and my own company, John Laing R & D, both of which have a very important operation called specification vetting. We make sure that everything in the specification is understood and it is not true to say that the construction companies look for what they can get away with. Having got that point out of the way, I would like to come to Mr. Fearon Wilson's paper. I found it very interesting; I could not understand some of it because I am not an economist, and we had enough comments on those last night. I assume that you do hire out CCL and Errut type products should the client want them and, if this is the case, my question arises from another question asked earlier and that is what does this lowering of water-cement ratio from 0.55 to 0.43 mean? Where do the economics reach a break even point, because if you are going to put a concrete mix down at 0.43 obviously it is going to have a higher cement content to get the minimum workability that is required. At what particular water-cement ratio is it worth using vacuum de-watering of concrete? At what particular mix design requirement is it worth bringing in early grinding systems? Have you got actual numbers which you can feed back?

Stephen G. Fearon-Wilson. I would just like to say that we can only hire out equipment as specified by the contractor. We are not in a position, and nor would we presume to be, to give any technical information on costings of that sort, it is beyond our ability to do so. Our job is to get the equipment that you may want onto the site in the best possible condition, at the best possible rate and we cannot really do more than that. We cannot get involved because we simply do not know. It must be the contractor who decides the economics of that part of his business. I am afraid that may not answer your question but it is the best I can do.

Ernest S. Fairweather. As far as early age grinding is concerned we do not specify the water content at all. We feel that is a matter for the design engineer who knows the working condition of the site he is working for and who knows, or should know, the type of concrete and the type of floor for which he is designing. We are not trying to do any speeding up as in de-watering; we are allowing normal curing, in fact we strongly recommend that curing be done properly as laid down, and we do not see any need, with our type of system, for us to make any attempt to restrict the design engineer in designing the mix as required for the purpose needed.

R. Colin Deacon. Could I make a comment on that last question. I would see both these techniques of early age grinding and vacuum de-watering, as I tried to indicate in my paper, as essentially alternative tools for overcoming the very real problem of finishing slabs which, with normal techniques of trowelling, require extended times of finishing. In my view anything that can reduce overtime working has economic benefits. All right, they have got to be offset against the costs of the equipment to be used, so each piece of equipment, each system, has got to be assessed on its merits, but overall both these systems can reduce overtime working and by bringing more of the operations within normal time working, and under the supervision of normal site control, the risks of malpractice are therefore further reduced. So the concrete technology aspects are fairly minimal. Although having said that, one must agree that there is some upgrading of the concrete, as Nils Petersons has said, with the vacuum de-watering process, but in my view the overriding value of both these techniques is an economic one.

Christopher Coward. I have a question for both Mr. Fairweather and Mr. Sankey who seem to be proponents of competitive systems. Why are they competitive? Why not combine the two systems? Why not use vacuum de-watering, then float finishing and then early age grinding?

Ernest S. Fairweather. I would say that each of them is aiming at a particular objective and I cannot see much benefit of combining them. I mean de-watering is a jolly good system and it has got its place in the construction industry. There is no point in de-watering, preparing all that and then finish up with grinding which does not need that preparation. I think the choice lies with the design engineer. I do not think that much could be gained by combining the two, could you Colin?

Colin Sankey. No I could not. I think from my particular point of view we rather adopt the attitude that it is better to get the concrete placed and finished in a normal working day, within our contracted time, than perhaps leave the concrete lying fresh overnight, during the winter months particularly. We like to get the concrete placed and finished in a normal working day. With our particular method of construction we are able to achieve this.

Session 6

Failures, Preventative Measures and Remedial Treatments

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CONSTRUCTIONAL FAILURES AND TREATMENT—A REVIEW

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INTRODUCTION

Concrete slabs are relatively simple structures consisting of concrete, steel, toppings or overlays, joints and substructure. The materials and methods used, together with principles of design and detailing are both well tried and declared (1,2) - therefore the incidence of problems and, particularly, failures, should be low. Regrettably, that is not so and failures are commonplace, if not usually critical. There are few well documented texts on the subject of failures and it does not appear to be a topic that engineers wish to highlight for fear of implying some basic inadequacy. However, a rather dramatic review of structural building failures is given by Scott (3) which, whilst not specifically relating to concrete slabs, does indicate the likely legacy of poor specification, detailing and workmanship generally.

As a theme for the keynote paper and for Session 6, we should ask ourselves why this situation persists. It surely stems from inadequacy but perhaps not lack of knowledge.

Cracked and spalled concrete floors, bridge decks, runways, jetties and roads are commonplace and, unless the failure is structural, will usually be tolerated for as long as possible. The maintenance of concrete is a nuisance and represents expenditure that at first sight might seem a waste. Aesthetics for slabs is low on the list of maintenance priorities.

In civil engineering and construction it would perhaps not be too misrepresentative to say that failures are usually matters we wish to forget or ignore, preventative measures reflect an institutionalised attitude of mind and preoccupation with trivia and remedial treatments are expensive and better put off until matters are critical and that, of course, they may never be, at least, not within the obligatory maintenance or ownership period. Such a view, I am sure, would not be tolerated if one was constructing aircraft rather than buildings and units as pedestrian as slabs. But there, I suggest, is the root of the problem. The consequences of inadequate design, materials or procedures are not likely to be catastrophic. In addition, building is a well tried activity with traditional attitudes and this too may lead to complacency.

Whatever the reasons, structures and slabs do fail or show signs of distress. Whilst not alarming initially, such states can be aesthetically unacceptable and with time could reduce the useful life of the structure. Attempts to provide diagnostic telltales as well as treatment are often fragmented, poorly conceived and even more poorly carried out. The two codes mentioned (1,2) are totally concerned with giving direction such that failures should not arise and consequently make no significant mention of methods of repair, structural or otherwise.

Why should this situation arise? Is there not a worthwhile commercial activity to be had in structural repair and maintenance? It might be considered that the incidents of failure are so few that the remedial requirement will remain a fringe rather than main constructional activity. I suggest this is not true.

Information on the money value of the remedial activity in overall construction, together with trends that might be occurring, prove very difficult to obtain. Published opinions invariably originate from the statistics and projections made by the Economic Development Committees (building and civil engineering) of the Department of the Environment. For instance, under the general heading of repairs, the anticipated expenditure for 1979 is as follows:

Item		Money Value (£ million)
Housing		1906
Public non-housing		1342
Private non-housing		624
	Total	3872

Within these rather nebulous figures there is an indication that the anticipated 6 per cent money value increase in total construction is largely attributed to "continued strong growth in the repairs and maintenance market" (4). This trend was also corroborated by the Director General of the Building Material Producers Association at a recent private meeting at the Cement Admixtures Association (5). In his view, significant cut backs on new construction activity must put pressure on the need to repair and maintain existing structures. Published money value figures seem to bear out this statement. Actual estimated and forecast figures up to 1980 are shown in Table 1 (4).

Table 1 Money Values for Construction Repairs and Maintenance

	ACTUAL VALUE (£ million at 1975 prices)							ESTIMATE	FORECAST	
	1971	1972	1973	1974	1975	1976	1977	1978	1979,	1980
Repair and Maintenance	3352	3678	3861	3761	3417	3220	3341	3765	3870,	3860
Percentage Annual Change	+1	+10	+5	-3	-9	-6	+4	+13	+3	N/A

In a recent publication (6), the National Council of Building Materials Producers were optimistic about the repairs, maintenance and improvement sector of the industry, although the total repair expenditure would remain at its present money value into 1980. The dramatic rise in repair expenditure in 1978 will diminish although will remain at a very substantial expenditure level. Despite attempts, it was not possible (7) to reduce these figures into repairs relating to concrete and, particularly, slabs. Nevertheless, at let us suppose 1 per cent of the 1979 forecast value, it still remains a substantial figure, approximately £39 million.

The materials and methods of repair are not a fringe activity and perhaps warrant a more acknowledged and formal presence in construction. The cost value of the general repair activity also reflects the magnitude of the failure/downgrading problem, within which concrete slabs are a part.

Why do concrete elements, and, in particular, slabs fail? After all, concrete has been used in some form or other for approximately 8,000 years (8).

Firstly, concrete is not an inert material that once cured remains unchanged with time. Atkinson (9), quoting from "Principles and Modern Building" - The Fitzmaurice Edition, which is now forty years old - "materials such as brickwork, stone and concrete, were assumed to be stable in volume and constructional details were based on this assumption, often erroneously, with the result that there were many failures. Many of the buildings depend on the use of concrete which was handled just as though it was a completely stable material. Drying shrinkage produced widespread cracking, in many cases this was so bad that houses became uninhabitable and had to be demolished". I would suggest that we have learnt very little over the last 40 years as far as translating preventative treatments (10) into routine and guaranteed practice.

CAUSES OF FAILURE

A commonplace indication of failure is cracking. It must be accepted that cracks are not generally avoidable within normal economic practice but they can be minimised and controlled by proper construction methods and detailing (2, 11, 12). Cracks do not always result in structural failure but they can result in a definite loss of performance, together with accelerated deterioration, thereby rendering the building unserviceable for its original purpose and consequently reduce its effective life. The main cuases of distress resulting in cracks may be considered under the following headings (13):

a) Negligence or bad workmanship,
b) Errors,
c) Movements and changes of concrete due to physical/chemical characteristics,
d) Reduction in continuity,
e) Errors in design,
f) Ageing and weathering,
g) Improper use or altered use of building,
h) Maintenance.

Negligence or Bad Workmanship

Negligence or bad workmanship on site usually results from inadequate or inexperienced supervision and is particularly prevalent on the smaller contract, where the owner/architect may feel unjustified in maintaining a qualified engineer on site. Without proper supervision, poor joints, honeycombing and inadequate compaction in placing the concrete can occur, which will result in planes of weakness. Improper curing results in varying and sometimes inadequate strengths and such a situation is aggravated by cold weather and frost.

The desire of the contractor to maximise shuttering use can result in striking forms too soon, which aggravates problems due to improper curing, particularly in cold weather. This situation is due to the contractor having an inadequate appreciation of the requirements and purpose of the specification. Again, overworking the shutters without proper maintenance can lead to improper fits causing leaching of grout, thereby causing weaknesses at construction joints.

Inadequate supervision can lead to misplaced and improperly secured reinforcement, resulting in too much or too little cover, either of which, in exposed conditions, can lead to corrosion, cracking and spalling. When considering supervision, it is essential that, in the event of any accidental damage at site, a thorough examination be made so that the full extent of the damage is recognised and concise instructions are given so that the reconstructed area should fulfil the original design concept.

Errors

Errors in construction can again be due to inadequate supervision or straight forward mistakes. The most common error in reinforced concrete structures, and, for that matter, slabs, occurs in the placing of the reinforcement. Sometimes it is the detailer who has not appreciated the sequence which must be effected on site in order to place reinforcement. Forcing of reinforcement into position can result in incorrect cover, misplaced or misbent reinforcement, inadequate bond, incorrect stirruping and positioning of cranked up bars. Uneven bunching of steel effectively changes the moment of inertia within any member and can result in localising stresses to one position as opposed to an even distribution. In the event of errors in setting out being discovered, these must be brought to the designer's notice so that eccentricities can be considered. Sometimes, however, they are covered up and cracks naturally result later.

Movement and Changes of Concrete Due to Physical/Chemical Characteristics

Considering the anomolous and natural volume changes which occur within concrete due to shrinkage, plastic and drying (14), creep, thermal movement (15, 16), carbonation and the effects of varying mixes, then it is only the unconsidered reserves of strength which will preserve the structure against cracking. Stress cracks will be due either to shear, tension or compression or any combination of these states of stress. Primary cracks are invariably accompanied by secondary cracks.

Concrete is porous and permeable, it is chemically sensitive and changes dimensionally due to temperature variation (16). It has poor adhesive properties and for materials to stick to it the surface needs preparation, either chemically and/or physically. Concrete, when exposed, can also be subjected to surface chemical changes such as carbonation and, in the long-term, can exhibit irreversible strain or creep. This latter property is often of benefit, rendering the concrete a more compliant material. Nevertheless, it should be accounted for.

During the change from wet plastic concrete to the hard and dry condition, the concrete may show plastic cracking and drying shrinkage as well as thermal cracking. Cracks may also occur due to plastic settlement which, whilst relevant to columns and deep beams, is not usually significant for slabs. It is perhaps not surprising that such a material can fail unless properly used.

One of the factors militating against good concrete is the water necessary to allow the concrete to be placed and compacted around reinforcement. Most concretes used in slab construction have water-cement ratios of 0.5 or more and slumps in the range 50 to 100 mm. Such concretes are characterised by the problems mentioned above, although readily compactable concrete may be made at water-cement ratios below 0.5 by using normal plasticising admixtures (see later Section).

In considering the above problems due to movement of concrete resulting from different physical characteristics, it must also be judged and appreciated that concrete testing cannot be a satisfactory identification of physical characteristics whereby the above problems can be avoided. Even the in-situ cores, often preferred by engineers, will not give the true physical characteristics of the in-situ concrete because of the cores lack of mass and confinement under test conditions. However, they do tend to give a worse condition than would occur in the parent concrete.

In advising on the treatment of cracks resulting from the movement of concrete, it is essential that such cracks are properly interpreted and that any treatment carried out deals with both primary and secondary cracks. Also the significance of surface crack widths in relation to reinforcement corrosion require careful judgement. A recent article (17) states that no significant relationship exists between crack widths and corrosion, despite the limits quoted in C.P. 110 (2) and the 1970 C.E.B. recommendations (18) which give,

Maximum crack width for exposed members	0.1 mm
Maximum crack width for unprotected members	0.2 mm
Maximum crack width for protected members	0.3 mm

Errors in Design

Errors in design are generally due to inexperience coupled with inadequate supervision in the design office. There is obviously no limit to such errors but the following short list is thought significant when considering consequent cracking.

1. Lack of appreciation of physical characteristics and requirements of concrete, particularly with regard to thermal movement and long term creep.

2. Use of dissimilar materials without provision for relative movements. This can be due to lack of co-ordination between architect and engineer.

3. Inadequate appreciation by the Design Engineer of site planning and sequence of construction. Conversely, site personnel disregard a necessary sequence of construction simply because they do not appreciate the true significance of a requirement developed in the design.

Ageing and Weathering

Generally cracks above 0.1 mm in concrete permit the ingress of airborn water, which initiates corrosion of reinforcement. If this occurs, a breakdown of the surface is rapid, due to a volume increase of reinforcement with corrosion, and will cause structural weakening. Such breakdown will be accelerated if chemicals are dissolved in the water, particularly if salt is present, either in the water or original concrete mix. However, the relationship between surface crack width and that at depth near to reinforcement is not established (17). Again, any weathering effect will be accelerated by frost, particularly vulnerable are those areas of the structure subject to wetting and drying as in marine locations. The geometry of a building will localise such cracking and thermal expansion and contraction movements will work on and accentuate existing hairline cracks.

It is important in recommending remedial/maintenance work, to determine with the owner the required viable life of the slab or structure. In such instances, any recommendations for treatment should identify essential structural repairs and recommend maintenance requirements.

Improper or Altered Use of Building

The change of ownership of a building can result in it being used for a different purpose. Environmental change will bring a new set of circumstances, which can accelerate or decrease existing deterioration or even initiate new, or possibly more serious, ageing. Increased heat in a building will accentuate thermal effects. Chemical/water attack can occur and possibly the most common, cutting of holes for services may affect structural stability in connections. In the case of increased structural loading, the use of synthetic compositions, such as resins for repair work, can be used to establish a composite section adequate to carry the increased loading. For example, the removal of a floor screed and its reinstatement with a resin bonding agent, can give an increased depth composite slab, capable of sustaining increased loading. Careful control of propping in such circumstances is essential but it can be done.

Maintenance

All buildings crack, usually more acutely immediately after completion as they dry out.

Proper maintenance and early recognition of any signs of distress will prolong the life of a building and will repay the costs spent on effective maintenance. Resins should never be considered as an expensive filler or good quality paint. A particular resin formulation must be carefully specified and selected to ensure that it is used correctly (see later section). For example, resins are ideal in providing a protective wearing coat for bridge decks and incorporating various additives can provide differing characteristics by way of skid resistance and permeability. Similarly, greater durability of floor screeds for a particular performance can be obtained. Resins can be elastic and will provide more durable seals in expansion joints than for example the more common polysulphides but, of course, at an increased cost, which must be judged against the life of the building.

When specifying a remedial treatment as part of the maintenance schedule, it is important to establish the primary cause of the failure or inadequacy. It may seem obvious but all too often cheap temporary solutions are used to cover up rather than eliminate. Elimination requires careful diagnosis and selection.

PREVENTATIVE MEASURES

There are many materials and procedure options that, if properly used, could be regarded as remedial measures. Indeed, many novel material possibilities remain dormant for reasons of cost, convenience and perhaps lack of awareness. As with repair treatments there is scope for engineers to innovate and use novel solutions to failure problems. For this reason, the various options should be known to such a degree that objective assessment of their use can be made. However, I suggest there is a reluctance by the client and, in some cases, the consulting engineer, to consider novel remedial measures and treatments because of an implied risk. Yet often the traditional approach has caused the failure to begin with. There is a need for an approval procedure that encourages responsible innovation within the remedial work area.

For instance, the incorporation of fibres, be they steel, glass or plastic, can impart functional benefit to concrete and slabs. The improvement of flexural loading and impact resistance is noteworthy as is crack control (19,20,21,22,23, 24). The use of a pozzolan to improve resistance of concrete to sulphate attack, give reduced heat evolution and perhaps thermal cracking, as well as enhancing cohesion and workability (25,26,27) and also minimising the affect of opalescent aggregates (28,29) should be considered, as should the use of admixtures to improve and extend workability at low water-cement ratios, entraining controlled amounts of air and modifying the setting and strength/time properties of concretes (30,31,32,33,34). Indeed, the introduction of superplasticisers (35,36) to impart extremely high or collapsed slump concrete could have practical merit for heavily reinforced slabs, although this has been doubted for reasons that are not clear (37,14). Polymer compositions can be applied either integrally or externally to the concrete to improve plastic bulk cured properties (38). Similar technology has been extended to the decorative and functional coating of concrete (39,40). These are a few of the continuously developing options aimed at improving and offsetting the inherent inadequacies of concrete, whether used as slabs or generally. Some of these developments are covered in more detail below.

The Use of Water Reducing Admixtures

It has been established for some years that, by using a dispersing admixture of the plasticising or superplasticising types (30,31,35,36), well compacted and higher strength concrete can be achieved by taking advantage of the increased workability state imparted by these types of admixtures. Unfortunately, in order to get acceptance of these admixtures, particularly of the normal water reducing/ plasticising type, concrete made to a given strength with lower cement content has usually been the commercial route. In other words, direct materials cost saving rather than improved concrete. Whilst this approach does not seemingly reduce the quality of the concrete within the cement content range 250 to 400 kg/m^3 (41), it does not produce a better quality concrete. It produces a concrete having unchanged 28 day strength for less cost. When using this type of admixture there are several options, as shown in Figure 1.

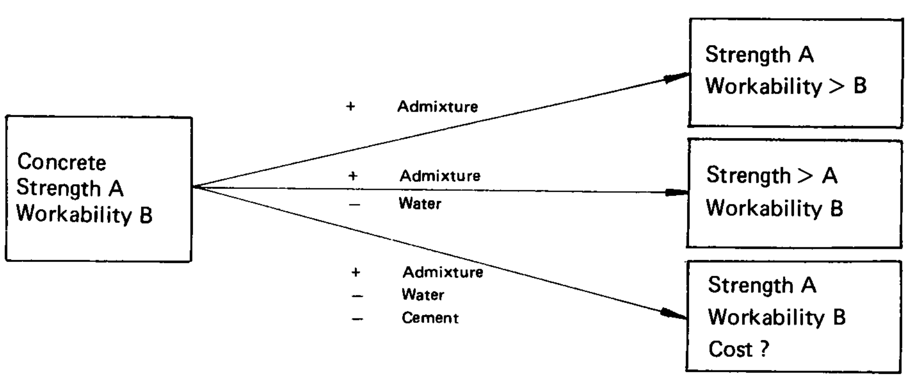

Figure 1 Various Options when using Plasticising Admixtures

If slabs are being made that contain awkwardly placed and/or close bunched reinforcement, then the increased workability property might offer greater benefit and the added admixture cost has to be set against the reduced likelihood of poorly compacted and porous concrete close to the reinforcement. Alternatively, if materials costs dominate, then plasticising admixtures might be used to effect an 8 to 12 per cent cement saving whilst maintaining concrete strength.

Thirdly, and perhaps most relevant of all, sufficient cement only is removed to cover the cost of the admixture, (approximately 2 per cent for 300 kg/m^3 concrete) although as much as 8 to 10 per cent could be removed and compensated for by water reduction. However, if the water content is still reduced by 10 to 14 per cent, this more than compensates for any strength loss due to the nominal cement reduction (42). This should result in a similar workability concrete, having better than specified strength, at no extra cost (Figure 2, curve III). In other words, we have available the means of making non-bleeding, readily placeable, cohesive concrete at no cost increase. It is odd that this practical advantage has not been siezed upon. Again, I suggest we are dealing with entrenched attitudes and the wish not to add another nuisance to the usual problems of making and placing concrete.

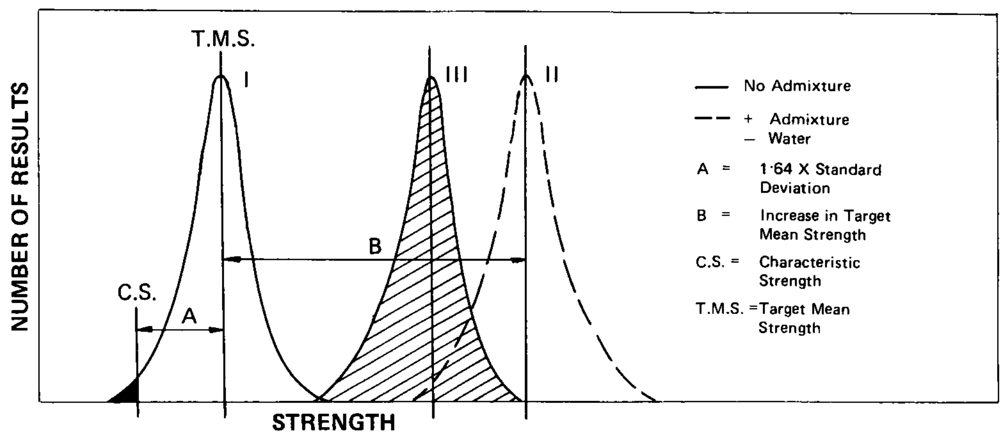

Figure 2 Strength Options as a Result of Water Reduction and Partial Cement Replacement

As an extension to using dispersing admixtures, the use of superplasticisers will remain special, due to the high relative cost compared to normal plasticisers (10 to 15 times greater). However, very large water reductions can be made (up to 30 per cent) which are reflected in low shrinkage properties and early high strength of such concretes (35,43).

Other admixtures, such as air entraining agents, may, improve freeze/thaw durability and prevent surface spalling of weathered and exposed slabs. Whilst there is no British Standard covering such admixtures (44) their use has become commonplace for roads and runways. An extension of their use is to overcome inadequate materials such as poorly graded sands. As a guide, 1 per cent of stable entrained air is equivalent to 15 to 20 kg/m^3 of fines addition, or 50 kg sand and 5 kg coarse aggregate, (45,46) or, alternatively, 5 per cent air increase results in an increase in compacting factor of 0.06 or an increase in slump from 12 mm to

50 mm (30). Other admixture types, such as accelerators, retarders and water proofers (30,31,32,33), can give benefit. However, C.P. 110 (2) does not encourage the controlled and positive use of admixtures. Quoting from Section 6.2.4 "Admixtures may not be used in ordinary structural concrete by definition. Admixtures may be used, however, in special structural concrete - all subject to the approval of the engineer". The engineer has to be a paragon indeed if he is to evaluate each and every situation relating to admixtures when used in structural concrete. Without a clear recommendation he is likely to be unenthusiastic.

In large slab construction the effects of thermal expansion/contraction can be reduced by means of cement replacement using pozzolanas such as slags and fuel ashes (25). Whilst early strength may be reduced, ultimate strengths are usually satisfactory. The use of these materials is now more commonplace in large pour construction. Having obtained good concrete, the material has to be placed and cured. Slabs have a high surface area in relation to concrete volume and so may lose water rapidly. Inadequate protection or curing can cause plastic cracking, due to rapid water loss, and shrinkage cracking, also due to water loss but over a longer period (14,47). To minimise these effects the wet concrete should be protected so that setting and early strength build up can occur under ambient conditions that are stable. In any event, water loss should be kept to a minimum.

Simple and effective methods for curing concrete when placed as a slab are well known (48,49,50) but so often carried out in a dilatory way. Simply covering the concrete may not be adequate since, if the outside temperature is high enough, thermal cracking may still result. Keeping the concrete damp is a simple and effective precaution, but often difficult to guarantee on site. An alternative would be to use a curing membrane that limits the rate of loss of water vapour. These materials were recently reviewed (51). In situations where very high temperatures are likely, membranes may not give adequate thermal protection when used alone. Some form of insulation is also required. In any event, Turton (14) is of the opinion that, to prevent plastic cracking, evaporation within the first 30 to 60 minutes should be minimised and this precludes the use of these types of membranes.

Protection of Steel Reinforcement

The problem of reinforcement corrosion has been well reviewed (52,53), both for onshore and offshore works (54). Steel comprising reinforcing bars corrodes if the surrounding pH is low enough and there is a supply of water and oxygen. Embedded steel surrounded by damp hydrated cement is, to a large degree, stabilised or passivated, due to the high alkalinity, with pH = 11.5-12.5 (55). However, carbonation and/or the presence of chloride ions, can negate the passivating effect and the steel will then corrode. The corrosion products are less dense than the steel itself and cause expansion, putting the surrounding concrete and, in particular, the cover, under stress and, specifically, tension. This sequence creates cracking and, ultimately, detachment. This type of downgrading is both unsightly and eventually will reduce the structural well-being of the slab.

Steel reinforcement as supplied is often in a rusted condition. A judgement has to be made whether such rusting is significant (56) before it becomes integrated into a slab. However, according to CIRIA Report 71 (57), such rusting may or may not have consequential effects. To paraphrase the conclusions,

1. The presence of rust on plain round bars improves its bond to concrete. Bond stress failure increases with the amount of rust up to a limiting factor, such as dimensional loss rendering the steel inadequate.

2. For deformed bars the presence of rust reduces the bond in proportion to the increasing amount of rust.

3. As to determining the acceptable degree of rust (assuming cross-section not reduced), it appears to approximate to that remaining after the bar has been subject to sudden impact.

4. As to very rusty bars, the recommendation is non-committal.

5. As to removing rust by means of wire brushing, at best the procedure is of no benefit and at worst it is detrimental to bond because of the polishing action caused by brushing.

Conclusions 1, 2 and 5 above might be regarded as unexpected. Assuming for the moment that, even if rusting can be accommodated within normal site practice, it still remains a rather vague issue and perhaps we should eliminate this problem altogether, rather than take regard of the vagaries of trying to quantify the effect, if any. In other activities than construction, it is generally agreed that unless corrosion protection is given adequate attention at the design stage, effects to control corrosion later will be more expensive and less efficient. That is the opinion of the Government Corrosion Committee (58). The same article quotes that some £1,365 million are wasted every year as a result of uncontrolled corrosion. No doubt corrosion of reinforcement plays its part and yet, for a price, the problem could be largely eliminated. For instance, reinforcing steel can be protected by coating it with solvent based protective oils (59) through to sophisticated materials, such as powder epoxy coatings (60). In the case of oil based anti-corrosion materials, care has to be exercised in that the retained oil layer may cause as much bond reduction as the original rust (57). The same caution would apply to alcohol/water emulsions (61).

An interesting and, perhaps, very relevant development relates to a single pack coating called Ferrotek 900 (62). This coating is thought to be a combination of an acid etch medium and a tough polyisobutyral polymer (63). Ferrotek 900 will apparently withstand flexing without detachment and the bond to concrete is not impaired, as can be seen from the results of push-out tests given in Tables 1 and 2, where load relates to rupture of the steel/cement bond. It should be noted that the results of accelerated corrosion tests, using sodium chloride solutions, gave consistently high values for coated bars, Table 2.

Table 1 Maximum Load Attainable in Push-Out Tests

TEST SERIES	MAXIMUM LOAD, kN (Mean of 5 Tests)	
	Control	Coated
1	7.1	7.0
2	9.0	9.0
3	12.7	12.8
4	10.3	10.1

Table 2 Push-Out Loads for Samples Subjected to Accelerated Corrosion

TEST SERIES	MAXIMUM LOAD, kN	
	Uncoated Bar	Coated Bar
A	9.0	11.0
B	9.0	13.0
C	4.0	7.0
D	2.0	7.0

Finally, the treatment is relatively cheap, costing approximately £15 per tonne of reinforcement (approximately £0.06 per metre length for 25 mm round bar). Surely we should take practical regard of materials such as this. The material, however is not without limitations and its use for protection to structural steel work subject to severe marine exposure is limited, particularly at thicknesses less than 100 μm, approximately 0.004 in, (64).

As far as the expensive and sophisticated coatings are concerned (60), the main conclusions drawn from a programme including 47 materials, 36 of which were epoxides, are quoted below.

1. The majority of coatings exhibited satisfactory chemical resistance to aggressive chemicals such as alkali (caustic and lime based) as well as sodium chloride. However, solvented epoxides were doubtful, although most were impervious to chlorides.

2. Flexible coatings were best as far as abrasion and bending resistance was concerned and powder epoxy coatings gave better flexibility compared to liquid epoxides. Polyvinyl chloride coatings were also very flexible but less tough than epoxies.

3. For deformed bars, powder coatings gave a more uniform layer, even at the high spots, compared to liquid coatings. Corrosion initially commenced at the high spots on bars.

4. As to load carrying ability of coated bars, epoxy coatings performed best at a nominal thickness of 125 to 275 μm (0.005 to 0.011 in) - optimum film thickness overall 0.007 ± 0.002 in. Creep was acceptable for all but PVC coated bars.

5. Selected epoxy powder coatings were considered the best means of protecting reinforcing steel as far as function was concerned but, of course, the materials are relatively expensive and methods of application not amenable to site use.

It is worth mentioning the more conventional methods for protecting steel against rust. Zinc coating (galvanising) can suffer from white rust under site conditions and corrosion under acid conditions (certain industrial atmosphers) will still occur. Hot dip galvanising increases reinforcement cost by 25 to 30 per cent. Most conventional alkyd type paints would be unsuitable, being attacked by alkali causing saponification, which results in the coating becoming more like a lubricant. Bituminous coatings also give poor bond between steel and concrete and, whilst cheap, would not be suitable.

Having taken regard of the concrete materials and the steel, the two are now brought together in intimate combination to produce a monolithic slab. Once formed, the slab may be subject to weathering, abrasion, impact, high stress due to local loads, flexing and bending, etc.. Apart from the last two circumstances, the response of the slab may well depend on the integrity and condition of the exposed surface. This surface is chemically sensitive, porous and temperature responsive. Can the surface properties be improved?

Surface Treatments for Concrete

The use of floor screeds (65,66,67) are not covered in this section so much as superficial surface treatments that integrate with the outermost layers of the concrete. Related topics such as synthetic resin based toppings (68) and bonding agents (69) are omitted.

Concrete may undergo surface treatment for functional or decorative reasons. Materials used for the latter purpose one may regard as paints and, other than the requirements of adhesion, the properties normally relate to conventional paint properties, such as colour stability, good film formation and retention of weathering properties. Functional coatings may well penetrate into the first few millimetres of concrete surface structure as well as cover the surface. Such surface penetration may completely fill the voids or, alternatively, coat pore walls, so changing the wetting properties of the surface. Non-functional concrete paints have been the subject of three recent Current Practice Sheets (70). Functional coatings or treatments are dealt with below.

It is generally true that the coating of concrete is better done after the concrete is cured and has undergone some weathering. Surface carbonation will tend to neutralise the alkalinity associated with the interior pores of concrete and so reduce attack behind the coating by saponification. However, coating treatments should bind the carbonated material. Surface efflorescence should also be removed (deposits of gypsum on and within the near surface) by washing with water and/or sequestering solutions (71). In addition, contamination and stains generally would need to be removed (72). Broadly functional coatings or surface treatments may be split into temporary and permanent treatments. These are dealt with in turn below.

Non-Permanent Water Repellents (Pore Liners)

These materials are distinguished from those that might be added to the cement during grinding to render it waterproof when hydrated in concrete (73). Coatings used for this purpose are usually dilute solutions, comprising stearates (74) such as polyoxyaluminium stearate or, alternatively, silicones, in organic solvents (75,76), as well as aqueous solutions of siliconates as well as silicone emulsions (77). These materials have little or no strength and, on trafficked areas, would not last long. Indeed, under normal weathering conditions, a ten year life span is claimed (77). Latex-siliconate suspensions (78) have also been used for sealing porous and lightly cracked concrete surfaces with the latex depositing on contact as a result of the suspension filtering out, and so filling the pore or discontinuity (79). Some waterproofers may also act as a curing membrane, such as the bitument/rubber latex emulsions (80) and claims are made about the capability of these coatings to accommodate small movement cracks. However, it should be noted that a BRE study, using various silicones, siliconates and silicatesters, when applied to masonry failed to reduce the weathering decay (81).

One material, an alkoxysilane, developed under the name Brethane, has indicated good surface consolidation and stabilisation that also prevents the migration of salts and subsequent re-crystalisation, causing surface damage (82,83). This material is in some ways between the simple silicones and the more permanent epoxy resin solvented systems. The degree to which such hydrophobic coatings function is arguable. Nevertheless, the treatments are temporary and not suitable for wearing slab surfaces subject to scarifying and superficial mechanical damage, however they can improve damp-proofness (84).

Permanent Water Repellents

Some 15 to 20 years ago, it was a growing practice to treat concrete surfaces with a dilute solution (organic solvent mixture) of high strength resin compounds such as epoxides. The dilute solutions (10 to 30 per cent mass/volume) penetrated 2 to 5 mm into the surface. The solvent evaporated, leaving a thin film within the pore spaces that then cured to a tough, slightly extensable, film, lining the pore space and moving without rupture in response to the concretes' environmental movements. A tough deposited and cured resin strengthened the outermost layer of concrete - similar to case hardening - they also prevent staining, whilst improving frost and chemical resistance. This concept was known as 'deep-in conservation' (85) and can be very effective in upgrading the surface of concrete. There is a quoted instance (86) where similar solutions were used to seal drying shrinkage and thermal cracks (from 3.75 mm down to 0.75 mm) in reinforced gunited slabs in combination with a neat epoxide resin, the in depth seal being obtained with the diluted composition. There are variations of this technique. For instance Quentglaze Sealer (87) cures by reaction of the resin with atmospheric moisture and that residing within the concrete, to form a strong in-situ elastomeric membrane, eliminating dusting and improving surface strength and chemical resistance. Polyester (88) and water dispersible epoxy resins (89) have also been used for this purpose.

The coatings or sealers above are not reactive with respect to the cement matrix interface, although they change from liquid to solid by reacting within themselves. Solutions of soluble silico fluorides (90) in combination with a wetting agent, are used to react with the free lime within the concrete pore space, so precipitating insoluble calcium fluorosilicates that are extremely hard quartz-like compounds. As well as strengthening the surface, such materials also reduce dusting. By comparison with the sophisticated organic sealers, these compositions are cheap (by a factor of 10 or less) and could be more widely used.

Interesting and related materials are the ethyl silicates and, in particular, condensed ethyl silicate (consisting of tetraethylorthosilicate and some 10 per cent of ethylpolysilicate) that hydrolyse and deposit silica from solution.

$$(C_2H_5O)_4 - Si + H_2O \rightarrow 4C_2H_5OH + Si(OH)_4 \downarrow$$

$$Si(OH)_4 - H_2O \rightarrow H_2SiO_3$$

These solutions are used in the foundry industry for increasing surface hardness and, whilst the deposited films are discontinuous, weathering resistance is much improved (91).

There are occasions when a coating or surface treatment may have to withstand aggressive agencies other than mechanical wear and tear, for instance, within nuclear facilities. Due to concrete's porosity, contamination with nuclides could occur easily and their removal would be almost impossible, without removing the concrete. To seal concrete under such situations requires extremely thick coatings, approximately 6 mm (0.25 in). There have been attempts in the U.S.A. to discipline the choice and specification for such coatings, which include epoxide based materials (92). As far as the author is aware, there is no similar guideline in the U.K. Data has been available for approximately 10 years on the embrittlement of cured epoxide resins, together with the effects of non-uniform cross linking as a consequence of exposure to nuclear radiation (93).

Within the general theme of preventative treatments, it has to be acknowledged that many materials options are available and an understanding of concrete's physical/chemical make-up exists. So much so that precognition of the likely problems should result in a specification embracing some of these measures. However, construction generally and that relating to slabs in particular, is concerned with designing to a price as well as a function. Full regard is not always taken of the possible preventative measures because they appear unnecessary at time of planning. Competitive tendering often adds to this limited outlook. With few exceptions, consultants are also unaware of what can be done to improve concrete and we have a ready-made blueprint for eventual failure, be it gradual or catastrophic. Whatever the outcome, it will be expensive.

REMEDIAL TREATMENTS

Many of the remedial treatments are themselves the preventative measures recognised as being necessary after a failure has occurred. Such treatments are therefore both undesirable and, with hindsight, should be necessary. Even so, failures do occur and no doubt will continue to do so. For concrete slabs, treatments can be divided into:

a) Structural over/reinstatement (massive repair),
b) Non-structural (superficial).

The second category may be regarded as less demanding, although using similar basic compositions to the structural materials. Epoxy resin mortars (94) and similar concretes (95), as well as flexible epoxy asphalts (96), may be used as durable, hard wearing overlays to exposed concrete surfaces. Novel inorganic cement compositions (97,98,99) are also used for patching spalled areas of concrete. The gypsum/OPC blends (98,99) cure very quickly and bond well to broken out concrete due to slight expansion on hardening. For non-structural gap filling, a resin bonded plug of this type may well be all that is necessary. There is some added merit in using an inorganic material in these situations.

Each category is dependent upon identifying the cause of failure and/or inadequacy before a specific treatment can be considered as having benefit. Obvious though this requirement may be, it is often not given full consideration because contractors and clients alike are often more concerned with doing something that appears to arrest a deteriorating situation, rather than diagnosing the cause and being very particular about the correct remedial treatment. Diagnosis can be elusive and result in highlighting the general obscurity surrounding responsibility, be it inadequate design, construction control or materials specification. Perhaps it is best not to become preoccupied with establishing responsibility in this paper, other than to state that the type of remedial treatment chosen is not always selected on technical merit and adequacy and one of the reasons for this is the obscure area of identifying the nature of faults, a case of "Doctor, just cure me but don't keep telling me that I am unwell".

Structural Over/Reinstatement of Failed Concrete

If a concrete slab is temporarily overloaded, such as by impact or differential settlement, then a fracture may result that, if left unattended, may substantially reduce the load carrying ability of the slab. Structural reinstatement means repairing the slab or other item to its original load carrying capacity and with similar load/deflection response. It does not mean that the repaired slab would have improved performance over that originally designed for. Over reinstatement can result by increasing the dimension, i.e. thickness of the slab by bonding an additional skin or layer of concrete, or, alternatively, the stiffness can be improved by bonding external reinforcement, i.e. steel plate to floor beams (100) or, alternatively, steel plates bonded to offset design inadequacy in bridge decks (101).

Structural reinstatement invariably depends upon the use of synthetic resins in some capacity (102,103), i.e. as adhesive, binder, stress transfer agent, load spreader or simply gap filler. Without such resins as epoxides, polyesters, polyurethanes or polyacrilates, structural reinstatement by mechanical means would be unwieldy and perhaps unsightly. The many structural repair case histories that can be cited have left the construction industry somewhat bewildered as to the degree of acceptance these resinous materials should have. Notable successes have to be set against notable failures.

Despite attempts to give clear guidelines on the use of resin repair compositions in construction (104,105), reservations remain. Doubts are often exaggerated by the apparent lack of quantitative information that relates to the structural use of these materials. In actual fact, much data is available but all too often it is fragmented and difficult to bring together against a particular structural problem. Also, nonconformity of test methods prevents a unified approach. With this shortcoming in view, Davies (102) as well as Hewlett and Shaw (103) have endeavoured to collect and quantify the structural requirements of various resin systems. Topics such as new to old concrete bonding, segmental bonding, bonded external reinforcement, structural resin concretes and resin injection are all dealt with. In a more generalised way, Allen (106) has also tried to categorise the structural use of resin based repair materials, including rubber latexes and polyvinyl acetates.

Interest in the use of resin compounds in construction, and, particularly for remedial work, was clearly focussed at the Plastics and Rubber Institute/Insitute of Civil Engineers Symposium, 'Resins and Concrete', held at the University of Newcastle in 1973. Clarke and Dussek (107) showed that damaged areas of slabs could be repaired by cutting out the faulty zone and replacing it with a precast section that was bonded to the parent slab, using epoxide resin grout, thus making a monolithic repair. Another paper by Butcher (108) highlighted the need to take account of the difference in properties between a repair resin mortar and concrete. For instance, the coefficient of thermal expansion of a resin mortar is greater by 2 to 3 times than that for concrete. A typical value being 25×10^{-6} per °C. If detachment of the mortar is to be avoided, then the cross sectional areas of the repair and the bond have to be balanced, taking regard of differential thermal expansion and the limiting shear strength of concrete (approximately 7 N/mm^2).

Structural repair of fractured (cracked) slabs, using resin injection, is the subject of the paper by Hewlett, Morgan and Savage in Session 6 and total reinstatement appears to be a practicable proposition. Their work confirms the work of Chung et al (109). In reinforced slabs the need to rebond the reinforcement to the concrete is important, the treatment of the cracks alone not being sufficient for total structural reinstatement. Epoxy resin injection is now well established as a sound and practicable method of concrete structural reinstatement (103, 110).

However, the author suggests repair disgnosis and treatment are specialised and not the province of the maintenance man, despite recent kits being made available for resin injection work (111). Crack sealing may, however, be achieved using such simple arrangements but the repair is unlikely to be structural.

A variant on resin injection is resin impregnation (112, 113) and one such treatment, well suited to slabs, known as Balvac, is worth mentioning. The resin impregnation process can be both structural and aesthetic. Areas or volumes of concrete or brickwork, etc., to be treated are isolated by means of a membrane or shroud under which resin passes at approximately 1 atmosphere pressure and is sucked into the surface pores and discontinuities, due to a reduced pressure within the substrate material itself. The resins are varied (silicones, polyesters and acrylates), depending upon the treatment required.

The advantages of this system are that large areas may be treated without isolating individual cracks. The system is non-disruptive, since pressure differential across the material is zero. The disadvantage is that the process is generally unwieldy and can take a long time as well as being wasteful on expensive resinous materials. Staining may also occur if structural adhesives are used. Nevertheless, it is a useful extension to the structural repair activity.

Resin injection may also be used to re-bond delaminated toppings. The detachment of toppings or surface screeds from a concrete base slab shows itself initially as audible hollowness followed by random crazing. Left unattended, sections of topping may detach causing a rut which steadily worsens. This problem is quite commonplace (114) and, whilst mundane, can be both expensive and very disruptive to remedy. Removing the topping and replacing it using a resin bonded screed or breaking out and patching are quite unacceptable in factories and warehouses where floor utilisation is at a premium.

Resin injection using polyesters is a convenient and relatively cheap solution. The technique and selection of resins has been reported by Hewlett and Hurley (115). Newtonian and thixotropic resins can be used having variable flexibility and cure rate.

Delamination detection and assessment of extent of repair are discussed in Hewlett, Morgan and Savage's paper in Session 6.

Assessing and Diagnosing Remedial Needs

The specification of repair materials and techniques is often hampered by insufficient materials test data that would allow quantitative judgement to be made. For instance, resin compositions are usually tested against standards that invariably originate from tests for plastics (116). This may be satisfactory when considering resin materials in isolation but, when used compositely with concrete, they may be misleading.

A recent paper by Tabor (117) highlighted the need to have test data that related epoxy repaired materials to representative surfaces and as composites. He proposed an improved slant shear test for investigating shear bond strengths of repaired composites. Tabor does not consider it a choice between absolute materials data as opposed to composite but more a matter of relating absolute bulk properties of both resins and concrete to the practical problem situation.

Ultimate strength properties are not the only criteria. As Tabor points out, most resins have much higher ultimate strengths than concrete. However, being lower modulus materials, the strain related properties could well be more relevant.

For instance,

a) Compressive, flexural and tensile moduli at several strain rates and temperatures up to the heat distortion temperature and down to -10°C,
b) Poisson's ratio,
c) Creep response within the temperature range 5 to 60°C,
d) Long term adhesion,
e) Response to cyclic loading and load reversals.

It should also be stated that many of the resinous repair materials available for use in construction are also used in very demanding situations such as the aircraft industry. Exhaustive evaluation preceded their use but then there was a need and a willingness to exploit these materials. The same recognition is not so apparent within the construction industry. It will be pertinent to gauge the response to the recent FIP verification and acceptance tests of epoxy bonding agents (110).

In order to establish the repair materials requirements for a slab and to assess the problems involved, we should look for a worst case situation and see how that can be accommodated. I would suggest that bridge decks represent a demanding slab requirement, since they are subject to dynamic flexural loads, causing load reversals, large temperature gradients, both across and along the slab, that across being the most demanding. In hostile climates, where de-icing may be an annual chore, then corrosion due to ingress of chlorides can become a very serious remedial work problem. For instance, the 41 year old Golden Gate Bridge at San Francisco has a deteriorated roadway that will cost some 9 million dollars to repair (119), this repair package consisting of:

a) Epoxy resin treatment to seal cracks, some of which traverse the depth of the slab.

b) Rust removal between slab soffits and stringers, coupled with grouting of voids between flanges and soffits, as well as spalled areas.

c) Installation of cathodic protection to arrest chloride induced corrosion of reinforcement.

d) Re-surfacing of deck with 25 mm thick layer of epoxy concrete.

As a comparison, deck replacement would cost $28.5 million and take some 2½ years. However, a temporary second deck would be needed to offset disruption during road replacement and would inflate the cost to $85 million (1976 costs). The original cost of the entire bridge when built was $27.6 million. This example, whilst somewhat extreme, does indicate requirements and their magnitude. Deterioration of bridge decks generally had reached a remedial cost in excess of $70 million in 1975 (120).

The use of salt for clearing snow-bound roadways gives rise to chloride corrosion. Chloride reduces the normal passivating effect of concrete on steel and corrosion results in cracking, both normal and parallel to the reinforcement, initially causing spalling and exposing the reinforcement further. It is possible to detect the state of ongoing corrosion before it reaches the point of obvious breakdown by measuring the electrical potentials of the reinforcement against a reference cell. Generally speaking, if potential differences equal to or greater than -0.35 volts are registered, then there is a 95 per cent probability that the steel is corroding, although obvious cracking and concrete distress may not have occurred. With potential differences equal to -0.20 to -0.35 volts, then there is a 50 per cent chance of corrosion occurring. With potential differences equal to or less than -0.25

volts, then there is a 95 per cent chance that the steel is not corroding.

If repair work has been carried out, corrosion would continue unless,

a) concrete below and around the steel had been removed before patching was carried out,

b) low permeability patching materials, such as epoxy mortars, had been used, since they are quite effective in reducing the rate of ingress of chlorides after repair.

Van Da Veer (120) also mentions a delamination detector instrument (121), which is similar to the device described in Hewlett, Morgan and Savage's paper in Session 6. The problem of assessing structural downgrading, porosity, delamination, etc., by non-destructive means, is very real. Most techniques, such as ultrasonics, tend to be local tests and gamma radiography somewhat qualitative as well as unwieldy. As well as diagnosis, there remains the verification of repair.

A systems test approach (122) has benefit but interpretation of response can be very complicated. A recent paper by Chung (123) highlights the problems of detecting voids and honeycombing within concrete using ultrasonics. Voidage smaller than 2 times the grid interval used would not be detectable. This is a real limitation and can make the technique difficult to use in practice. Correspondence from Tomsett (124) only compounds reservations about this method. There is a real need for diagnostic and proof of repair methods.

Van Da Veer (120) states that, for an uncracked OPC concrete with a water-cement ratio of approximately 0.4, then the minimum concrete cover to reinforcement of 51 mm (2 in) is required to insure against chloride ingress and subsequent corrosion. For a 0.5 water-cement ratio concrete, the minimum cover is 76 mm (3 in).

A relevant development for cracked slabs that are already cathodically protected, is to use resin injection to seal and render the concrete monolithic again but the resins themselves need to be electrically conducting. This is thought possible using techniques developed from the conducting paint industry (125, 126). Similar compositions could be used to offset the usually slow low temperature cure rate of epoxide mortars and bonding agents by using low voltage electrical heating, similar to that developed for segmental concrete bridge construction using epoxy jointing compositions (127).

Many of the repair treatments and remedial measures are very dependent upon resinous or organic compositions. Acceptance and specification of these materials is often both reluctant and infrequent and usually subject to individual scrutiny. There is a need for clear performance test requirements and individual but appropriate tests, such as the Arizona Slant Shear Test (128) for measuring resin/concrete adhesion as well as the approach mentioned by Tabor (117).

The use of resins to effect structural repair needs to be studied at higher temperatures in order to determine consequential response to a hazard such as fire. Indications are that, for deep crack repair, the surrounding concrete would degrade before significant resin downgrading occurred (129, 130). However, if contractors invest in developing techniques and understanding of the limitations of these materials and their repair possibilities, it is not unreasonable to expect the construction industry to adopt and exploit the methods. This is not a plea for slack evaluation but more for a co-ordinated and positive approach to novel methods, rather than repeating the traditional approaches that may have failed and given rise to the repair situation to begin with.

CONCLUSIONS

Perhaps, from the preceding text, it is possible to draw practical guidelines that should prevent failures and reduce the incidence of remedial and repair treatment, specifically for slabs.

Slab design is dependent upon well tried principles and does not represent departure from normal practice. However, detailed design may be lacking for the very reason that it appears to be pedestrian and not requiring attention to detail.

Good design and attention to detail are wasted if specifications are not implemented as intended. Control and competence at site are fundamental.

The basic materials, namely concrete, steel and formulated compositions, either to be used integrally or externally to the concrete, have to be properly prepared and applied.

Medium to high workability concrete (75 to 150 mm slump) can be achieved at water-cement ratios not exceeding 0.4 and the use of such concrete should be encouraged. The use of normal plasticising admixtures to achieve this situation is recommended. For concretes that are subject to freezing and thawing and containing an air-entraining admixture, then water reduction should be made compatible with the improved workability usually achieved by using such admixtures. Well compacted, dense and uniform concrete is desirable.

Preventative treatments are already a partial admission of failures. However, there may be instances where materials are poor and control at site unavoidably lacking, in which case, supplementary treatments to the concrete and/or steel would be of benefit. Nevertheless, proper application of preventative treatments also require a clear understanding of the problem as well as control over their proper application.

Inspection and monitoring of slabs, particularly in critical situations such as bridge decks, is a none too practicable task. Even so, early detection of faults is conducive to cheaper and more permanent repair. Non-destructive integrity testing should be exploited more, particularly those methods offering a systems approach, rather than a point test.

Novel methods of repair offer to the engineer a chance to innovate and extend a normal practice. Without wishing to curb such involvement, it is nevertheless necessary to have relevant materials test data available. All too often, this is not so and repair works themselves give rise to secondary repairs.

A formal evaluation or approval scheme for several of the remedial methods and repair treatments should be considered. Approval should then ease acceptance and exploitation of responsible innovation.

In-service training for engineers that will make them more aware of the many materials options available and the understanding that goes with them. The patent medicine approach just will not do.

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ASPECTS OF DESIGN PROBLEMS AND REMEDIAL WORKS

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ABSTRACT Good integration of design and construction is essential if economical structures are to result. Attention must be paid to all aspects of the design and construction processes to avoid later problems. Remedial treatments must be considered individually within the context of the function of the structure and the cost of alternatives.

INTRODUCTION

The problem of slab design within a normal office practice is assessed in the light of the development of the slab, design methods and materials. Various limitations of design methods, detailing and construction techniques emphasize the need to integrate the various processes involved to achieve an economic and functional result.

Remedial treatments are discussed and amplified by three examples of different slab failures, emphasizing the need to assess each case individually.

DILEMMA OF CHOICE

Historical Development of the Concrete Slab

The first recorded use of a slab floor or roof construction was in Greece during the Hellenic period. Civilisations prior to this,including the ancient Egyptians, constructed some magnificent two storey buildings but the suspended floors were constructed using timber poles at close centres slung between walls 'adobe' style and the gaps filled with wattle and mud. The Ancient Greeks did not have the luxury of readily available timber and turned to stone for their structures. This essentially consisted of stone post and beams with stone slabs spanning between the beams. However, the shortcomings of stone used in tension made its presence felt and roofs generally developed in the form of arches.

Slabs are a grossly inefficient structural form and it was the pressures of scarcity of land, the need to carry successively higher loads, together with the rediscovery of concrete and the development of wrought iron and steel, particularly during the industrial revolution of the Victorian era, that led to development of the slab that is used today. The Victorians developed the joist filler slab to the level of a fine art and the end of this era saw the start of modern reinforced concrete techniques.

Concrete is a satisfactory material for use in compression. Its inherent inability to support any but the smallest loads in tension, however, limits its efficiency in conventional reinforced concrete. The ability to carry high compressive stresses is utilised in prestressing so that the tension limitations are avoided.

Flat Slabs

The filler joist floors, Figure 1(a), developed by the Victorians proved to be very popular and were widely used for a long time. They were, however, fairly wasteful in terms of steel used and were only capable of spanning in one direction in a simply supported manner. It was not until the turn of the century that reinforced concrete framed buildings, generally of beam and slab construction, slowly began to appear in Scotland. As the need for wider spans and improved load carrying performance of the slab grew, the concept of the flat slab which took advantage of the plate properties of the slab was developed by the Americans during the 1930's, see Figure 1(b). The method of analysis and design for this type of slab is empirical and is based on experience of performance over the years. It is a concept that has proved to be very successful and is still widely used today despite or even perhaps because of its empirical nature.

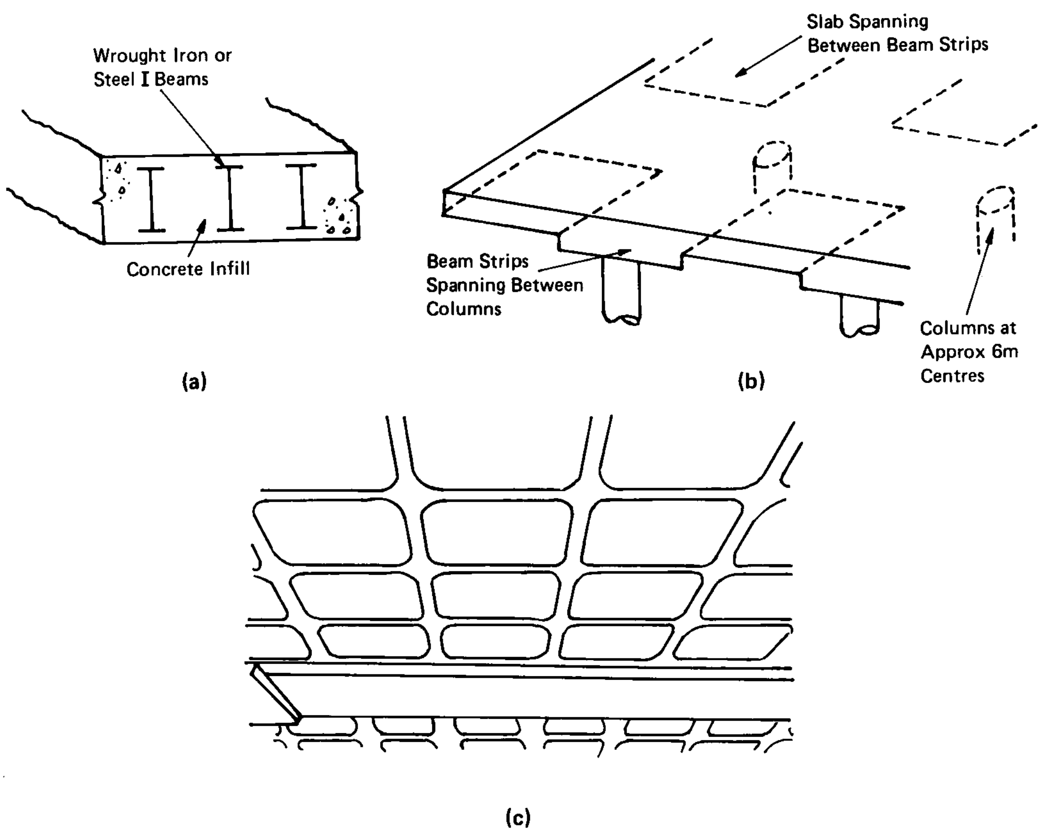


Figure 1 Concrete Floors, (a) Filler Joist Floor
(b) Flat Slab and (c) Voided Floor

Voided Floors

Designers were very conscious of the fact that slabs had drawbacks. The inneffi-ciency of the slab is such that considerable surplus concrete is associated with slabs. This became a substantial problem as spans and loading increased, with the dead weight of the slab itself becoming a significant part of the total design load. This was overcome by the use of voided or coffered floors shown in Figure 1(c), in which concrete was provided only to surround the tension reinforcement and to provide the compressive strength of the slab. Beams could also be intro-duced into this system by the simple expediency of removing the moulds or blocks. Slabs are relatively thin structural members having a high surface area and are, therefore, vulnerable to problems of shrinkage and curing. Since the coffered slab has a much higher surface area, it is more vulnerable in these respects. Another shortcoming of the coffered slab is due to the fact that the major portion of the concrete is at the top of the slab and this limits the ability of the slab to accommodate reverse bending.

Precast Floors

It was recognised that cast-in-situ slabs had some shortcomings. They were very much at the mercy of the weather, particularly in winter, required considerable amounts of timber shuttering which had minimal re-use and were subject to the vagaries that can beset site mixed concrete. The young, but rapidly growing, pre-casting industry was quick to realise these shortcomings and to a reasonable degree capitalised on them, taking advantage of high quality control and a protected environment by casting slabs indoors in large sheds with heated floors. In addi-tion, most precast units had hollow cores of one type or another, thus improving their efficiency. The precasting industry was, therefore, able to offer slab floors capable of being speedily erected without formwork and immediately capable of sustaining load, thus providing a platform for men and materials.

One of the prime disadvantages of precast slabs was the cost of handling and trans-portation. Transportation also limited the practical width of such units, thus effectively ensuring that precast units are capable only of spanning in one direc-tion. The preformed nature of such slabs also limited their ability to perform other than as a simply supported system. However, the advantages of precasting were readily combined with the pioneering work of Fressynet to produce a precast prestressed floor unit using long-line casting techniques. This product is perhaps the one most suited to precasting techniques capable of producing high strength high quality concrete, and indeed precasting prestressed units is now virtually an automatic process using concrete mixes capable of retaining a moulded shape. These units are formed without moulds, in a process similar to that of extrusion, and are then cut to length and the wires released the following day.

The precast industry essentially deals in component parts of a structure, thus jointing techniques, if not carefully controlled, can lead to the Ronan Point Building type of collapse. Since this disaster, much attention has been focussed on this problem and the industry has overcome many of the difficulties in providing suitable connections, capable of accepting and transferring loads and moments.

Composite Floors

The introduction of composite construction by the precast industry has done much to combine the advantages of in-situ concrete with those of precast. Thus, the precast units are designed to carry their own weight plus that of the structural topping. Subsequent imposed loading is then carried by the precast units and in-situ concrete, acting together in structural combination, see Figure 2. This effectively removed the need for formwork and provided a structural system which could accommodate reverse bending. Composite construction is also particularly at advantage when having to cope with high superimposed loadings and has been used to great advantage on bridge decks for both these reasons.

Steel dowels or stirrups were used initially to cater for the horizontal shear forces developed at the interface of the in-situ topping and precast slabs, which proved to be difficult in relation to the manufacturing process of precast units. Recent developments in the precasting industry would suggest that provided reasonable care is taken with the making and placing of the in-situ concrete no steel is required to cater for these stresses.

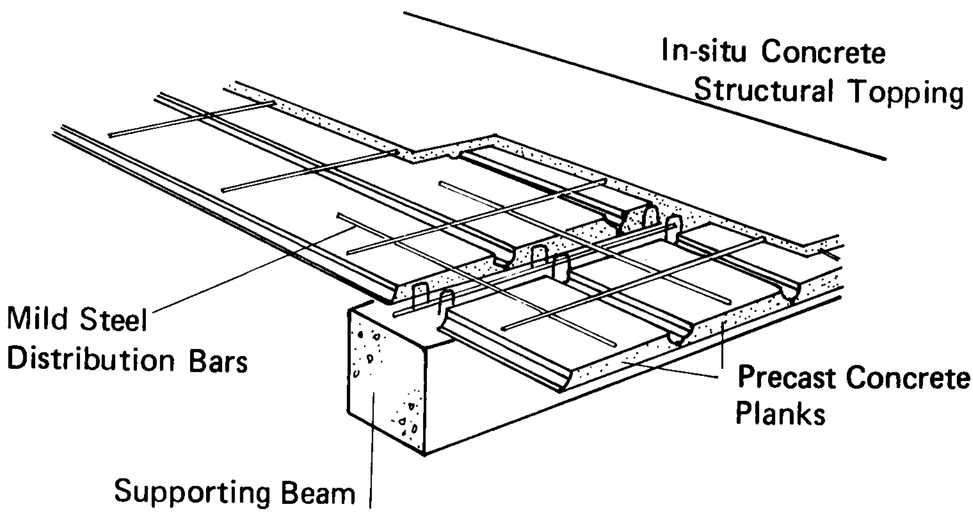


Figure 2 Composite Floor Construction

Lift Slab Construction

The most significant attempt to combine the advantages of precast with those of cast in-situ slabs that has taken place on construction sites has been the development of the lift slab system. Although this system attempted to reproduce on site the advantages of precasting, it has not been widely used in the United Kingdom.

Concrete in Tension

The major dissatisfaction in respect of reinforced concrete is undoubtedly the poor performance of concrete in tension. The mechanism of steel reinforcement in the tension area of the slab has only had moderate success over the years and cracking is always associated with concrete in tension. This problem is of course exacerbated by shrinkage and has on occasion led to corrosion of tensile reinforcement. Cracked concrete is particularly unacceptable in sterile areas of hospitals and water retaining structures. The modern trend is to try to achieve 'no tension' or crack-free concrete. The concept of producing floor slabs that are subjected to either compressive stresses only or minimal tension, has been developed by the

Americans who have post-tensioned flat slabs successfully for some time now. This produces a light, efficient structure with minimal problems in respect of concrete since the concrete is rarely subjected to high tensile stresses.

The production of a concrete which can be stressed in tension without attendant cracking would readily be welcomed by engineers, contractors and clients alike. For this reason, therefore, the advent of glass fibre reinforced concrete is the subject of much interest. However, like many innovations, the industry is only beginning to realise some of the properties and uses of this material. What has been somewhat disappointing, although understandable, has been the fact that companies which have the rights to develop this material are, for economic and other reasons, not yet pursuing a vigorous policy of development. This is obviously one aspect of the future but perhaps further development of the use of prestressed concrete in novel situations may yield better dividends.

SOME FACTORS INFLUENCING THE DESIGN AND PERFORMANCE OF SLABS

Conventional Analysis

The behaviour of simply supported and continuous slabs spanning in one direction only is relatively easy to predict and C.P. 110 and other codes have much to say on the analysis and design of such structures. These codes also give guidance on the analysis and design of concrete slabs spanning in two directions based on the Grashof-Rankine formula and on mathematical analysis by Westergaard. These guides, however, cater only for slabs carrying a uniformly distributed load supported on all four sides. Many concrete slabs can be more accurately defined as elastic plates and variations to support conditions as well as loading conditions can make analysis of such slabs extremely complex.

Classic Plate Theory

Classic plate theory has been well documented and is based on the two usual assumptions, that plane sections remain plane and that elastic behaviour only is realised within the range of loadings considered. While this method is occasionally used in design offices to analyse slabs, it is lengthy and cumbersome, involving complex mathematics. Thus, it is limited in its ability to handle varying support conditions and forms of loading and has limited application in practice.

Yield Line Analysis

Yield line analysis, developed by Johansen and modified by others, assumes a plastic collapse mechanism at ultimate load conditions by the formation of yield lines. The ultimate load is then calculated on the basis that the work done by the loading must be at least equal to the internal work dissipated. This method of analysis is simple in concept and ammenable to hand calculation. It is also applicable to a wide variety of slab shapes and load conditions. For these reasons, therefore, it is widely used in design offices. However, failure to determine the yield line pattern which will give the lowest ultimate resistance will lead to an over-assessment of the ultimate capacity of the slab. Thus, it is important that this type of analysis is carried out by engineers with considerable experience, as a poor choice of yield line will result in lowering of the load factor. Furthermore, the system is mainly suited to predicting the collapse of a slab of known reinforcement, and the method gives no indication of steel reinforcement required other than at the yield line. For these reasons, therefore, steel disposition should not depart too far from that required for elastic analysis and as with all methods of analysis of the ultimate condition, it is important that the performance of the slab under working conditions is checked. Another disadvantage of this method is its inability to predict the capacity of a slab which varies substantially in thickness.

Finite Element Analysis

The advent of the computer, with its ability to solve simultaneously a great number of equations, has provided the designer with one of the most powerful tools, namely finite element analysis, for analysing all types of slabs. In brief, the method comprises of dividing the slab into a number of notionally discrete portions. The load conditions, boundary conditions, bending and deflections, etc., of each discrete portion can then be expressed as in classic plate theory. By assuming compatibility of the boundary conditions for each adjoining discrete portion, a substantial number of simultaneous equations will result, which can then be solved simultaneously by the computer to provide an elastic analysis of the total slab being considered. The accuracy of the results can, of course, be improved by refining the mesh. However, the finer the mesh the more computer time and hence cost is involved. The flexibility of this method is such that it is possible to vary the size of mesh over the slab, thus limiting the refined portions of the mesh to more localised problems within the slab.

Idealisation of the Slab

The methods of analysis which have been briefly described above are unable to cope with local effects such as holes, re-entrant corners, restraints and abrupt or even varying changes of sections, although to be fair, aspects of these can be modelled on a finite element programme to give some indication of their effect. Despite this the designer has a reasonable choice of methods of analysis of the slab. Helpful as this may be, it is to no avail unless the notional slab to be analysed reasonably represents the real slab. This is perhaps one of the most important aspects of design as the process of idealisation of the slab to be analysed inevitably requires some simplification of the real slab. Much care is, therefore, required in choosing the form of simplified slab to ensure that the aspects that become oversimplified or neglected are of secondary importance in relation to the performance of the slab in the real situation; failure to do this will result in the distress of the slab.

SLAB ON SOLID GROUND

Evaluation of Design Criteria

It is true to say that in comparison to the attention devoted to suspended slabs, ground bearing slabs have been neglected and little information is available. This is possibly due to the fact that the majority of these slabs are small in area and are not required to carry any substantial loadings, thus they are seldom utilised to their full capacity. Occasions do arise, however, where ground slabs are either large in area or are required to carry heavy loads. Much of the information available on the performance of such slabs has been developed from analysis and empirical data obtained from roads design. In general, the performance of such slabs is directly related to the properties of the soils below the slab. Thus, the CBR values together with the compressibility and other properties of these soils have to be taken into account. However, the critical factor in the performance of such slabs is uniformity of support rather than the actual bearing strength of the soils.

The Cement and Concrete Association has published literature on the performance of such slabs, suggesting various thicknesses of slabs depending upon loads carried and nature of subgrade. Should a more sophisticated evaluation of the performance of such slabs be required, it would be necessary to design the slab as a plate supported on elastic springs and subjected to the envisaged imposed loading. This can be very complex and to the authors' knowledge cannot be fully evaluated. It is more realistic to assess the consistency of support of the soil and if necessary replace part of the soil with more suitable material to provide a reliable base for the slab. Care must be taken when dealing with compressible or expansive soils.

Joints

Joints within ground slabs are also the subject of much speculation. Much depends on the use of the building and whether the formation of small cracks within the floor slab is acceptable. Chequerboard construction is now accepted as being ineffective in preventing shrinkage cracking. Moreover, it was an inefficient method of construction and is steadily being replaced in favour of casting large areas of floors in long strips. Either way, location of joints in one direction is governed by the practicalities of laying and tamping by boards and these joints are usually about 5 m apart. The distance between joints in the other direction depends on control of cracking and the Cement and Concrete Association in the U.K. recommend a spacing of the order of 10 m for reinforced slabs for crack control joints and a spacing of 70 m for expansion joints. Much depends on the internal environment and degree of cracking which may be acceptable. Thus, the circumstances may require much closer joint spacing than already described.

The joint details are of paramount importance in highly loaded floors to prevent differential settlement of adjacent panels of slabs. Substantial steel dowels located horizontally across such joints are generally adopted in such circumstances, see Figure 3(a). The adoption of a concrete strip below such joints has been in the authors' experience very effective with slabs supporting high concentrations of load, see Figure 3(b). It is however, a costly exercise and joints should be positioned to provide the maximum area of slab between joints. The advantage of the system is that it provides support for the vertical slab shutter and also allows the construction of the slab without having to provide dowels through the vertical shutters.

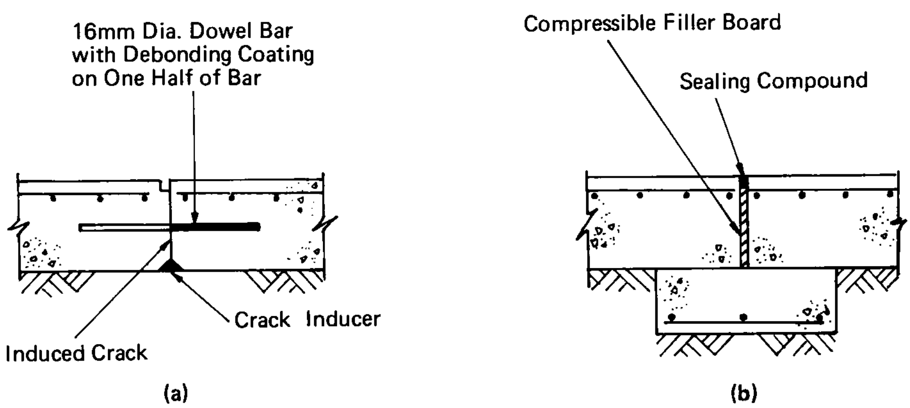
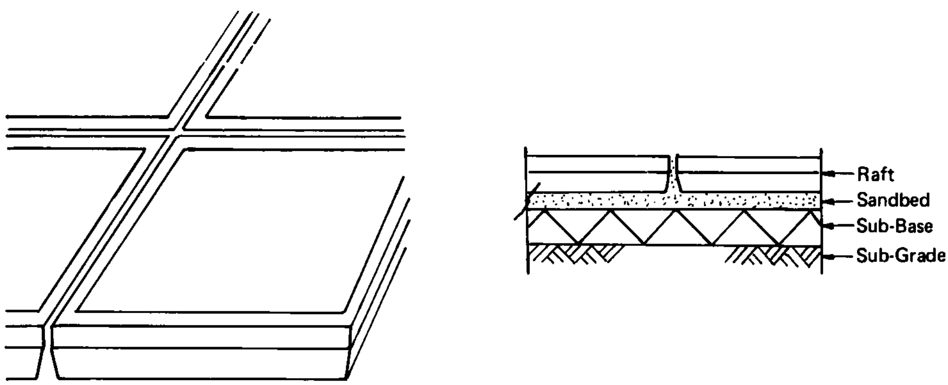


Figure 3 Construction Joints for Highly Loaded Ground Floor Slabs

Precast Ground Floor Slabs

One alternative to in-situ ground bearing slabs is precast rafts, not too dissimilar from the 600 mm x 600 mm and 900 mm x 600 mm paving slabs used domestically. Although larger, say 2 m x 2 m, thicker, say 100 to 215 m, reinforced internally and with steel angles along their edges, the units behave in the same way by acting as a hard running surface over a sand bed. A typical cross section of a precast ground floor slab is shown in Figure 4. Depending on the bearing capacity of the subgrade a sub-base layer may be required on top of which a bed of moist clean sharp sand is laid and compacted to give the required falls and profiles. Slabs are then laid maintaining the joint spaces which are subsequently filled with a sand slurry or sometimes dry sand. The joints must be kept full in service.



Slab

Typical Joint Detail

Figure 4 Precast Ground Floor Slab

The factors influencing the design are: 1) the need to find the critical points on the slab surface where cracking will occur under the loading configuration, and 2) the interaction of a simple design method for use in practice, which will balance an economic slab with a minimum of ground preparation.

The slab is idealised as a number of finite element plates with the nodes supported on Winkler foundations, symmetrical about both axes and with the stiffness of the spring analogy varied to allow for varying subgrade. Analysis of the stresses within the slab and the underlying soil allow design charts to be produced which take into account the nature of the subgrade. An assessment must then be made on the conflict between a high load factor in a heavy and uncompetitive slab and a lower factor which allows a more competitive product in which distress can occur on occasions. This is one of the few instances in structural engineering when the replacement of an isolated failed unit can be accepted, as a heavier slab can be used if the same loading conditions are to be repeated. The advantages these slabs provide over a cast-in-situ slab include,

1) they are capable of carrying load immediately after laying;
2) they allow easy access to underground services;
3) they can be relevelled if settlement of the sub soil occurs;
4) they can be easily replaced in the event of damage;
5) they can be recovered if alternative use of the paved area is required.

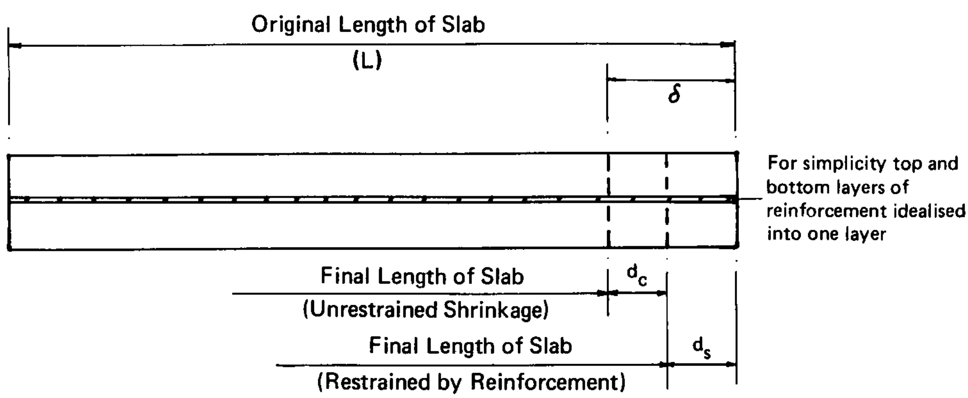
Precast slabs have been used both internally and externally over long periods in many different locations as the running surface of access roads, industrial level crossings, dock quays, transit quays, transit sheds, warehouses and factories.

PREVENTATIVE DESIGN

Effect of Movement

The movement of concrete during its lifetime is a phenomenon that has been well documented by many people. While expansion of concrete can arise due to an increase in temperature and/or humidity, perhaps the most damaging effects to concrete can be as a result of shrinkage and creep. Initial shrinkage and cracking can be controlled by insulating the slab to prevent rapid loss of the heat of hydration and by preventing loss of moisture during the curing period. The use of judiciously placed reinforcement can do much to control cracking. The long-term shrinkage can also be reduced by using low shrinkage aggregate such as limestone. While these precautions can do much to reduce shrinkage of concrete, however, movement will still occur and the presence of restraint to shrinkage movement has to be considered.

In considering the effect of reinforcement in restraining shrinkage of concrete it can be shown as given below that this can be of a low order.



let, A_c = Area of concrete
A_s = Area of steel
σ_c = Stress in concrete
σ_s = Stress in steel
E_c = Young's modulus of elasticity for concrete
E_s = Young's modulus of elasticity for steel

For equilibrium, the total compressive force in steel (T) is equal to total tensile force in concrete (C).

Thus, $C = T = A_c \times \sigma_c = A_s \times \sigma_s$

Also, $$d_c = \frac{CL}{A_c\ E_c}$$

$$d_s = \frac{TL}{A_s\ E_s}$$

Total Movement = $d_s + d_c$, assuming the following properties,

$\delta = 400 \times 10^{-6} \times L$
$A_s = 0.01\ A_c$ (1% steel)
$E_c = 24 \times 10^3\ N/mm^2$
$E_s = 200 \times 10^3\ N/mm^2$

Thus, it can be shown that
$d_s = 369 \times 10^{-6} \times L$ and $\sigma_s = 74\ N/mm^2$ compression
$d_c = 31 \times 10^{-6} \times L$ and $\sigma_c = 0.7\ N/mm^2$ tension

It should be noted thus that the effect of reinforcement in restraining shrinkage of concrete is of a low order, but is of course related to the percentage of reinforcement considered. In a similar manner, it can be shown that the effect of columns in restraining a floor is of a low order, mainly because the stiffness of a floor slab is much greater than the stiffness of a column.

Cracking and high tensile stresses have more properly been associated with external restraints such as concrete walls with their long axis parallel to the direction of shrinkage of the slab, or L shaped slab floors, see Figure 5.

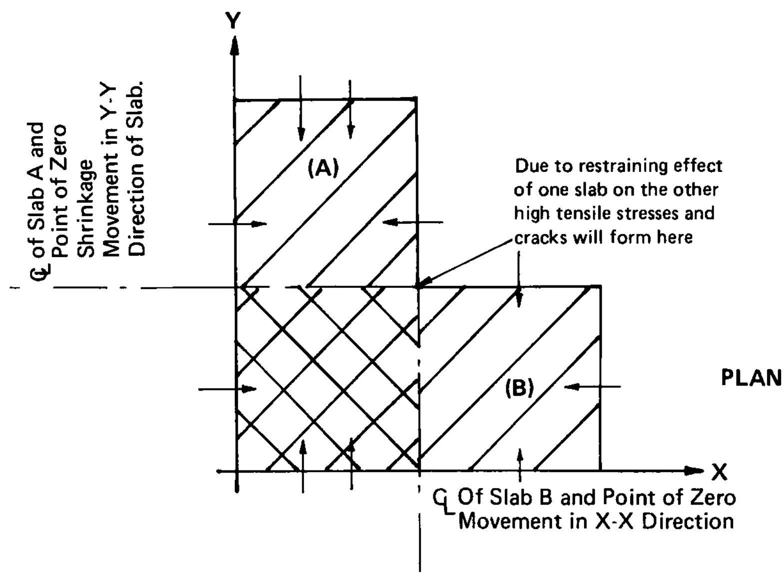


Figure 5 Influence of Slab Shape on Shrinkage Stresses and Cracking

As total shrinkage movement varies depending upon the areas of the slab, it is important to consider the total area of slab to be cast between movement joints or around significant restraints as the locations of such joints can radically influence the behaviour of the slab. In general, a floor length of 30 m could result in an unrestrained movement of $400 \times 10^{-6} \times 30 \times 10^3 = 12$ mm. While such movement may not cause distress to the floor slab, it can have a significant effect on wall claddings and partitions if they are unable to accommodate movement. Additional quantities of steel reinforcement will provide further restraint and increase the tensile stresses experienced by the concrete, possibly resulting in cracking of the concrete. Should this steel be located on one face only of the slab, the movement of the unrestrained face will cause the slab to deflect and increase the size of any cracks already generated. This effect when considered with conventional loading on the slab can lead to much higher deflections than would normally be associated with the slab floors.

In conclusion, therefore, it is of paramount importance to ensure that the choice of design of the slab is truly representative of the true slab and that the design of the slab takes account of the concept of the building as a whole. Careful detailing of the slab is also important to ensure that the building is constructed in the manner intended by the designer.

REMEDIAL WORK TO SLABS

The Cause of Distress

If a structure, or members of a structure are showing signs of distress, then that is an indication that at some time previously there has been a shortcoming in the design or construction sequence. Some of the possible reasons are:

1. Design. During the design stage some factors may have been overlooked, their importance under-estimated, or a simple human error in calculation may have taken place. This can result in an inadequate structure being passed to the detailing stage.
2. Detailing. The concept which the designer had in mind in the layout and design of the structure may have been such that unsympathetic detailing altered the performance of the structure in such a way that it suffered distress. Consequently care must be taken to ensure that the detailing adequately performs the function required of it without inhibiting the overall behaviour of the structure, or other members of the structure.
3. Construction. Given an adequate design with good detailing, failure can result from the structural concept as detailed not being executed on site. Again, care must be taken to ensure that the construction is carried out correctly and that the structure behaves as the designer intended.

If the design, detailing and construction have somewhere been inadequate, the inbuilt factors of safety that the structure would normally have are eroded and in the worst instance significant distress occurs. On the occasion where the level of integrity of the structure as a whole, or elements of it, are lowered unacceptably, either in terms of it failing to fulfill a functional requirement or in the worst case, leading to imminent collapse, remedial measures must be initiated.

Nature and Extent of Remedial Works

The extent of the remedial works required obviously depends on the significance of the distress and hopefully total demolition can be avoided. However, factors beyond the control of the engineer do sometimes influence the remedial work which can be carried out, e.g. litigation, continued use of the structure during repair, or cost.

In the majority of instances the remedial works are required to render the structure safe and allow it to fulfill its design functions. As indicated above, restriction of occupation of the building during remedial work can often determine the nature of these works and force the engineer to adopt a particular solution. Similarly, the cost involved in obtaining building performance exactly similar to that originally intended may prove prohibitive and a less functional solution has to be adopted.

A Precast Floor

One example of the failure of precast floor slabs to behave normally but yet not to render the floor unserviceable was due to the unfortunate siting of an office thermostat. The building in question was a steel framed, two storey office block with the first floor being constructed of precast reinforced concrete slabs with a screed topping. Within the screed, cables for underfloor heating were laid and due to the geometrical layout of the building, which was a long narrow structure, the thermostat for these heating cables was positioned mid-way down the corridor which ran the full length of the building at first floor level. Unfortunately, the aggregate used in the manufacture of the precast slab had a high shrinkage value and the slabs were only reinforced in the bottom tension zone. Because the swing doors at each end of the corridor were usually kept open the thermostat rarely cut out the heating circuit, and as a result the slab suffered an abnormally high drying out. This in turn led to significant shrinkage and because the slabs were only reinforced in the bottom they suffered additional deflection. Unsightly cracks opened in the floor finishes where the ends of the slabs butted against each other, on the steel cross beams, and because of the office layout this frequently occurred in the middle of open floor space. The deflection stabilised relatively quickly and as the slabs retained adequate strength no structural remedial measures were required in this instance. Resiting of the thermostat was recommended.

Ground Floor Slabs

An example of the failure of a ground bearing slab was in the instance of a steel framed warehouse built on infill in an old harbour. The promotion of the project was such that no integrated design team was involved and consequently the qualifications pertaining to the design were not carried through to the construction of the warehouse. The ground slab settled due to consolidation of the underlying fill whereas the piles supporting the steel frame of the building in the majority of instances suffered no deflection. Consequently the slab tended to hang from the perimeter beam spanning between the outside columns and also from the piles supporting the internal columns. As shown in Figure 6 the contractor opted to have construction joints on two sides of the internal columns and it could have been expected that only one of the four slabs meeting at the internal column would have been affected by the presence of the 'hard point' at the column. However, in constructing the head of the pile a large void had been created around it which had been infilled by concrete and consequently all four slabs tended to hang from the column base. At the perimeter of the building the slabs in some instances hung from the edge beam and where a construction joint had been made it settled away from the adjacent perimeter beam. Various forms of remedial work were considered. A grid of slipcoated mini-piles totally supporting the slab on the underlying bedrock would have allowed various loadings to be achieved depending on the grid spacing chosen. Lifting the in-situ floor and replacing it with concrete raft slabs would have allowed for future consolidation of the fill. Both these alternatives would have proved more expensive and involved a longer evacuation period than the solution finally adopted.

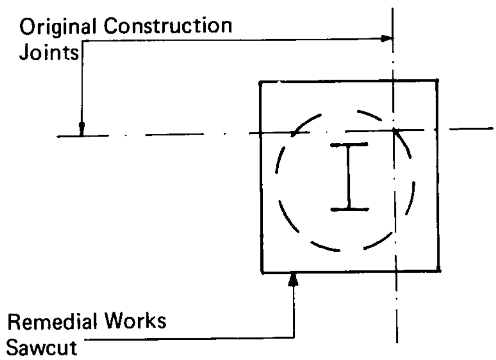


Figure 6 Remedial Work to Ground Bearing Slab

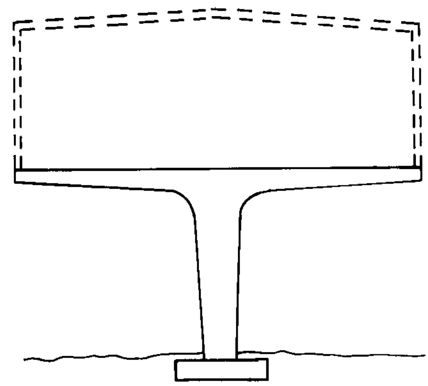


Figure 7 Cantilever Slab Floor

The principle in this solution was to entirely separate the slab from the internal piles and perimeter edge beam by sawing through the slab. The voids beneath the slab were infilled by grouting using a mixture of fly ash and cement and in the event of future consolidation of the fill the slab will settle without any tendency for it to hang from the building structure. To achieve a level floor within the warehouse, black top was laid which can be topped up as required in the event of settlement.

Cantilever Slab

When constructing a cantilever slab obviously it is essential that the reinforcement should be at the top of the slab but there have been occasions when, for different reasons, the reinforcement has been badly placed. In instances such as these the remedial works are determined not only by cost and whether the structure can be evacuated during the remedial works but also by the visual change which certain types of remedial works would necessitate. This last factor may well be the over-riding one when alternatives such as a steel cantilever support from beneath are proposed.

One cantilever structure suffering distress was in the shape of a mushroom, a glazed room sitting astride a central column as shown in Figure 7. The slab started to deflect relatively soon after the building was completed. An exhaustive design check was carried out and various possible causes of the deflection were considered. No satisfactory conclusion was reached and when cores were taken through the slab it was discovered that the reinforcement had settled to near the middle of the slab, substantially reducing the moment of resistance. Four possible solutions were considered as follows:

1. Prestressing across the top of the slab using a central ring and end anchors located at the edge of the slab.
2. Raising the edge of the slab by jacks prior to introducing high tensile steel flats radiating from the centre of the slab. These flats would be connected through the slab and a new topping added

before releasing the jacking force, thus, providing, in effect, new reinforcement acting compositely with the existing slab.

3. Installing a steel arch system supporting the existing structure from beneath and relieving the concrete of structural stresses.
4. Building a brick supporting wall around the edge of the slab to support the weight of the existing perimeter wall and roof thus changing the slab structure to a propped cantilever. This has the effect of reducing the moment presently experienced by the slab by a factor of 10 and consequently provides substantial stress relief.

The final decision in respect of these possible solutions has yet to be taken but it is unlikely that prestressing will be chosen due to undesirable concentrations of stresses likely to occur. For aesthetic reasons solutions 3 and 4 are also less acceptable and it would appear that solution 2 is at present the most favoured.

In general no engineer wishes to become involved in remedial works where a fault as outlined above has occurred. There are, however, instances of accidents or terrorist damage which pose just as dramatic problems, but again in these situations each case must be dealt with individually although the achievement of a satisfactory remedial solution is all the more rewarding.

CONCLUSIONS

Many varieties of slab forms have been developed in response to economic, social and physical constraints. In view of this, it is of the utmost importance to consider the shortcomings as well as the advantages of any type of slab to ensure that the slab form chosen is consistent with the requirements of the project.

Design methods available to the designer are either mathematically complex or are computer orientated or, in the case of ground bearing slabs, relatively undocumented. These design methods inevitably require the rationalisation of the slab into a simplified form. It is important, therefore, to ensure that this process does not lead to substantial errors due to misinterpretation of the design philosophy of the slab.

Regardless of precautions taken, some degree of movement will be experienced by the slab. The effects of slab reinforcement, movement joints and external restraints can be considerable and must be evaluated during the design process.

Remedial operations require a re-assessment of the design, detailing and construction of the slab to identify the nature of incompatibility in relation to the real slab. It is essential that the economics of the situation are reviewed and that the measures proposed are sympathetic to the developed structural form of this slab.

RETEXTURING OF CONCRETE SLABS

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ABSTRACT Concrete slabs which are subject to trafficking by vehicles, pedestrians or animals will wear. In time, no matter what original texture was applied to the slab, it will wear to a smooth surface and present a hazard. This deficiency can be remedied by many methods; this paper concentrates on the retexturing of slabs by means of grooving with tungsten carbide tipped flails, a method which enables grooving to be carried out in confined areas and eliminates the need for water as a coolant. The equipment used is described, as is the process which has proved to be economical and very long lasting, with no detrimental effect on the concrete. The paper is illustrated by case studies of successful applications of the grooving technique.

INTRODUCTION

Over the years there has been a growing but misconceived demand for concrete floor slabs, be they for factories, warehouses or workshops, to have a good cosmetic appearance. In pursuit of this objective it is common practice for such floors to be trowelled, either by hand or power trowels, to a highly polished smooth surface. Although such a surface looks very good, it is, of course, extremely slippery. With the wear which occurs due to the passage of vehicles or pedestrians then, depending on the type of aggregate used in the concrete which all too frequently is not given sufficient consideration, the floor will polish still further and become extremely hazardous.

Engineers and architects who are responsible for specifying industrial floors have not paid sufficient attention to the developments which have taken place in the surface finishing of concrete slabs for roads and runways. For these slabs it was also common practice many years ago to trowel the surface to obtain a good appearance. Although trowelling did achieve this, the smooth finish also caused many accidents and eventually, and quite properly, the highly polished road pavement was rejected on safety grounds.

One of the first developments in anti-skid surface finishing for road slabs came from the U.S.A., where burlap drag was used to give the surface some texture. While this was an improvement on the previous smooth finish it was still found to be insufficient and was quickly superceded, first by transverse brushing, then by deep wire brushing and now by what is considered to be the ultimate in safety terms, transverse grooving.

No road would be accepted by the client authority unless the surface texture complied with accepted safety standards. Unfortunately no such standards exist for industrial floor slabs but the system described in this paper can be used to restore to such slabs a finish which is as safe as that required for a high speed road.

SPETIN CONCRETE GROOVING EQUIPMENT

Grooving concrete floor slabs presents problems which do not occur on road slabs. One of the major problems is access, equipment tends to be fixed in position and grooving has to be carried out in the intervening areas where the normal movements of traffic have to be maintained with the minimum of disruption. Another problem is that diamond tipped saw blades cannot be used to cut the grooves because of the quantity of slurry which would be formed from the cooling water required. This slurry could cause problems with drainage in the building but more importantly the very fine dust left as it dried may damage the goods or machinery occupying the building.

The Spetin concrete grooving machines overcame these problems. The units are very compact and have built in dust-extraction equipment to control the fine airborne dust produced by the dry method of grooving used. The coarse detritus left after grooving can be readily removed with mechanical sweepers. Two versions are available, the Spetin-12, which has a 300 mm wide cutting head, and the Spetin-20, shown in Figure 1, which has a 500 mm wide cutting head. Both machines can work to within 225 mm of a wall or obstruction in the forward direction. The Spetin-20 has a fixed rearward overhang from the cutting head of 1600 mm. On the Spetin-12 the rear overhang is variable, if the dust collector box is back mounted there is a 1825 mm overhang from the cutting head but if the dust box is side mounted this can be reduced to 1010 mm. Using the Spetin-12 in this latter configuration it is possible to operate quite easily in an area only 2 m wide, forming the grooves across the width by means of two half lane passes.

THE GROOVING PROCESS

In both versions of the Spetin machines the grooves are formed by tungsten carbide tipped tines or flails impacting lightly on the slab surface at the rate of 7400 impacts per minute. The cutting head consists of a belt driven main shaft around which revolve a set of secondary shafts positioned by cast spiders. The tines or flails which are mounted on these secondary shafts are guided into the required linear pattern by machined spacers, see Figure 2. When the head is rotating at speed the tines become radial to the main shaft and as the machine moves forward the successive light impacts of the tines cut and form the groove.

The grooves formed, shown in Figure 3, are 8 to 9 mm wide by 3 to 4 mm deep, spaced on average 40 mm apart and with a deliberately ravelled edge. An absolutely clean, square edged groove, such as would be produced by sawing, provides a less efficient interaction with the traffic using the floor than the ravel edged groove, which thus gives a superior non-slip action.

The grooving is normally carried out transversely to the main flow of movement across the floor. Where cost is less important than achieving the ideal surface texture, however, the normal groove spacing can be increased to 80 mm and the slab grooved in two perpendicular directions to give a diamond pattern which provides an omni-directional non-slip texture.

The process is suitable for use on any good quality concrete, the strength of the concrete ideally being 30 N/mm^2 or better. It is also possible to use the technique to re-texture granolithic toppings, which can equally well become polished with use, and it has been used successfully on metal-bound Stelcon pre-cast slabs.

Figure 1 Spetin-20 Concrete Grooving Machine

COST OF GROOVING

The cost of this method of grooving obviously varies depending on the particular situation. The most important factor affecting the cost is the size of the area to be treated. A second is the strength of the concrete or, more importantly, the type of aggregate used which is normally considered in three broad groups: i) flint aggregates, ii) quartzite, granite and gravels and iii) medium hard limestone. Typical average costs prevailing at November 1978 are shown in Table 1.

It is apparent from Table 1 that grooving costs are very much lower than the costs which would be incurred in preparing and placing a special non-slip topping which can average out at approximately £3 to £4 per m^2.

In addition to the direct cost savings, grooving enjoys a further advantage over toppings: the non-slip effect of grooving remains even if the slab surface is worn down by 2 to 3 mm. Since under normal circumstances it will take many years for a concrete floor slab to be worn down to this extent, the grooving has an extremely long life.

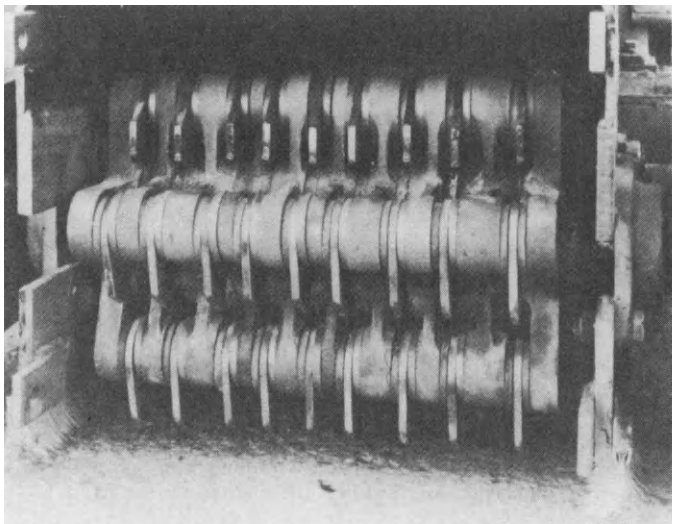

Figure 2 Cutting Head of Spetin-12 Grooving Machine

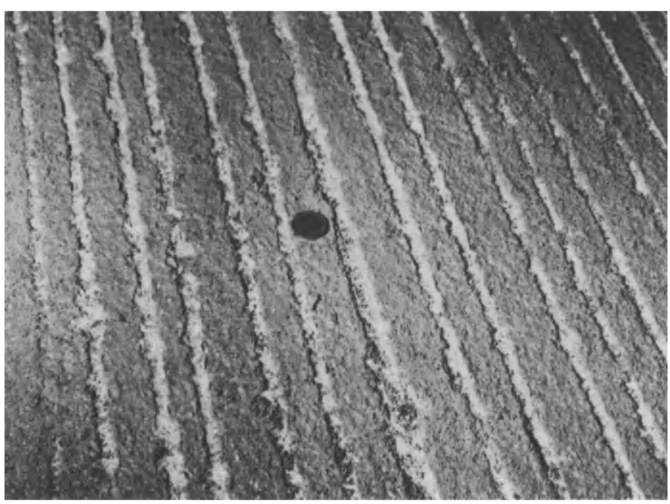

Figure 3 Surface Texture Produced by Spetin Grooving Machines

Table 1 Typical Costs of Spetin Concrete Grooving for Different Aggregate Types

AREA TO BE GROOVED m^2	COST, £/m^2		
	Flint	Quartzite, etc.	Limestone
<500	1.50	1.50	1.50
500 - 2000	1.00 - 1.25	0.92 - 1.17	0.87 - 1.12
2000 - 5000	0.95	0.87	0.85
>5000	0.85	0.77	0.75

SIDE EFFECTS OF GROOVING ON THE CONCRETE SURFACE

Due to the nature of the process, forming grooves by impact cutting, some concern has been expressed with regard to the possible side effects of grooving, mainly in respect of the occurrence of microcracks around the grooves which, if present, could prove detrimental to the durability of the slab surface.

To investigate this possibility a short series of comparative tests, including direct microscopic examination and the assessment of abrasion resistance and freeze-thaw durability, was carried out on 22 cores taken from an exterior concrete slab before and after grooving. The slab was over ten years old and had been designed for a characteristic strength of 28 N/mm^2 (4000 psi) using 20 mm (¾ in) maximum sized gravel aggregate and incorporating 3.75 per cent entrained air for frost protection. Tensile strength tests (solid disc Brazilian type) carried out on the remainder of the cores used for preparation of the abrasion test specimens indicated no significant difference either between the grooved and ungrooved cores or between different levels, from the top surface, of the grooved cores.

Microscopic Examination

Crack maps were prepared by microscopic examination, at up to 45 x magnification, of axial sections cut from the cores at right angles to the direction of grooving or to the direction of the original brushed finish. All the sections examined showed the expected appearance of concrete throughout their length (1) with a predominance of aggregate bond cracks, primarily caused by such factors as settlement, bleeding and the normal differential movements associated with hardened concrete, namely shrinkage and thermal movement. Some mortar cracks were present but were not significant.

Typical crack maps from the top 40 mm of grooved and ungrooved sections are shown in Figure 4, and it can be seen that there is no tendency towards any increase in cracking due to grooving. The bond cracks breaking the surface at groove 2 may have been caused by the action of grooving,although it is apparent that there are similar cracks around aggregate particles breaking the top surface of the ungrooved section,while on the grooved section the area around the large aggregate particle just below groove 3, approximately 1 mm from the exposed surface, shows no sign of any bond or other cracks.

All the sections examined showed similar patterns of cracking to that illustrated in Figure 4. Thus, from the microscopic examination there is no direct evidence of any detrimental effect on the concrete from the grooving process.

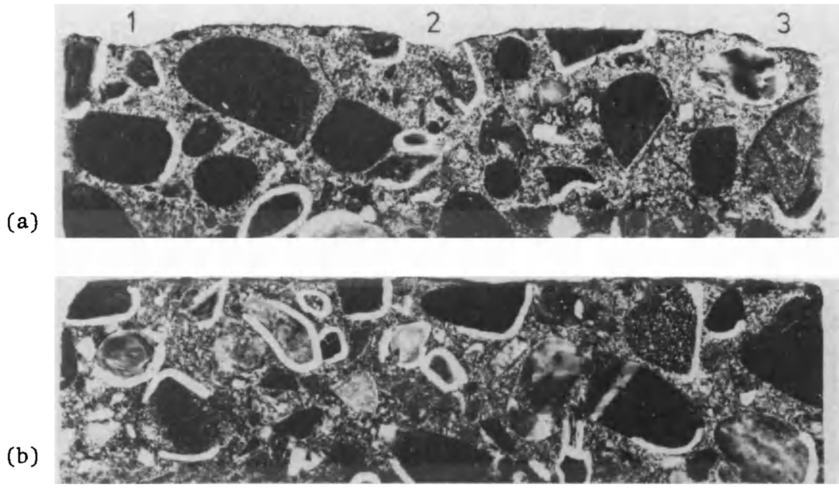

Figure 4 Typical Crack Maps of Grooved (a) and Ungrooved (b) Cores

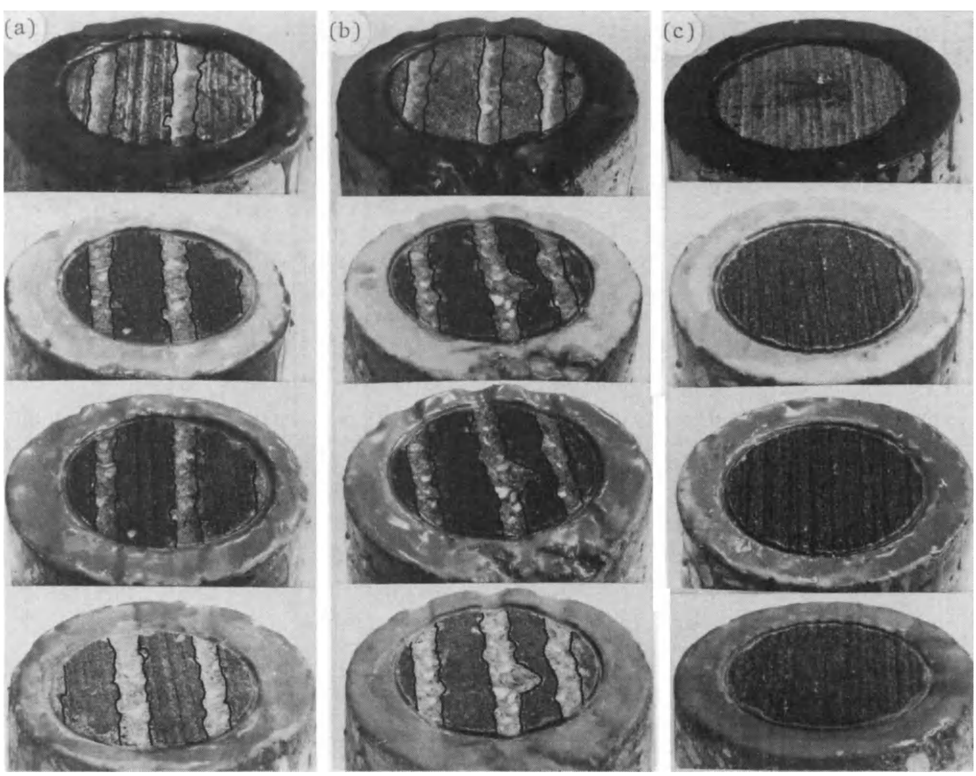

Figure 5 Effect of Freeze-Thaw Cycling on Typical Grooved (a and b) and Ungrooved (c) Core Specimens (top to bottom 0, 30, 90 and 150 cycles)

Freeze - Thaw Durability Tests

The 150 mm diameter cores were recorded to three-quarters of their length from the top surface using a 100 mm diameter thin-walled diamond tipped drill. After drying, the annular ring so formed was filled with an epoxy resin so that moisture entry into the core during the freeze-thaw tests would be through the top or bottom surface only. The tests were carried out in a cabinet designed to the recommendations of the Permeability and Durability Working Group of Unicemento (2), freezing to -25°C being carried out in air and thawing to +5°C in water. The tests were run on a cycle of 4 hr freezing and 4 hr thawing, with damage being assessed by visual examination only.

Typical photographs of grooved specimens (a) and (b) and an ungrooved specimen (c) at the start of the test and after 30, 90 and 150 cycles are shown in Figure 5. Deterioration of the surface of the grooved cores was negligible in almost all cases, a slight extra ravelling of the edges of the grooves, see Figure 5(a), being the only effect. The area of damage at the centre of the core shown in Figure 5(b) occurred during the first 30 cycles, and was caused by the spalling of a piece of aggregate which overlapped the groove and had been left with a very shallow embedment by the grooving process. It can be seen that once this piece of aggregate had been removed, little further deterioration took place between 30 and 150 cycles of freezing and thawing. No deterioration of the surface of the ungrooved cores was found, as is clear from Figure 5(c).

It is apparent from the freeze-thaw tests that the process of grooving had no significant effect on the durability of the concrete.

Abrasion Tests

Abrasion tests were carried out using the Dorry abrasion apparatus on 50 mm diameter x 12.5 mm long discs cut from the top surface of the cores. The discs were set in resin and abraded with fine quartz sand (0.6 to 0.3 mm single size) fed onto the steel wheel of the apparatus at the rate of 0.75 kg/min. The ungrooved discs were tested under a load of 2 kg, the grooved discs either under an equal load or a load sufficient to give an equal stress on the face of the discs. Measurements of loss in mass were made after every 100 revolutions of the wheel.

The amount of wear expressed simply as loss in mass was related to the load on the discs, as shown in Figure 6(a). When expressed as a loss in mass per unit area of the face, the wear was related to the stress on the face, as shown in Figure 6(b). Wear was found to be uniform across the discs with no evidence of any tearing of the edges of the grooves which again suggests that the concrete had not been weakened in any way by the grooving process.

APPLICATIONS OF SPETIN GROOVING

Some particular applications where grooving has proved extremely effective in restoring a non-slip surface texture to floor slabs are illustrated in Figure 7.

Figure 7(a) shows a Spetin-12 cutting 4 mm deep grooves in the high quality granolithic floor of a warehouse. The warehouse is owned by a cardboard case manufacturer and is used by heavily loaded smooth wheeled fork lift trucks which had worn the surface, producing a potentially dangerous situation in the wet. Previous attempts to restore adequate skid resistance by chemical treatment or mechanical means had failed, but the Spetin-12 successfully grooved 300 m^2 of the very hard

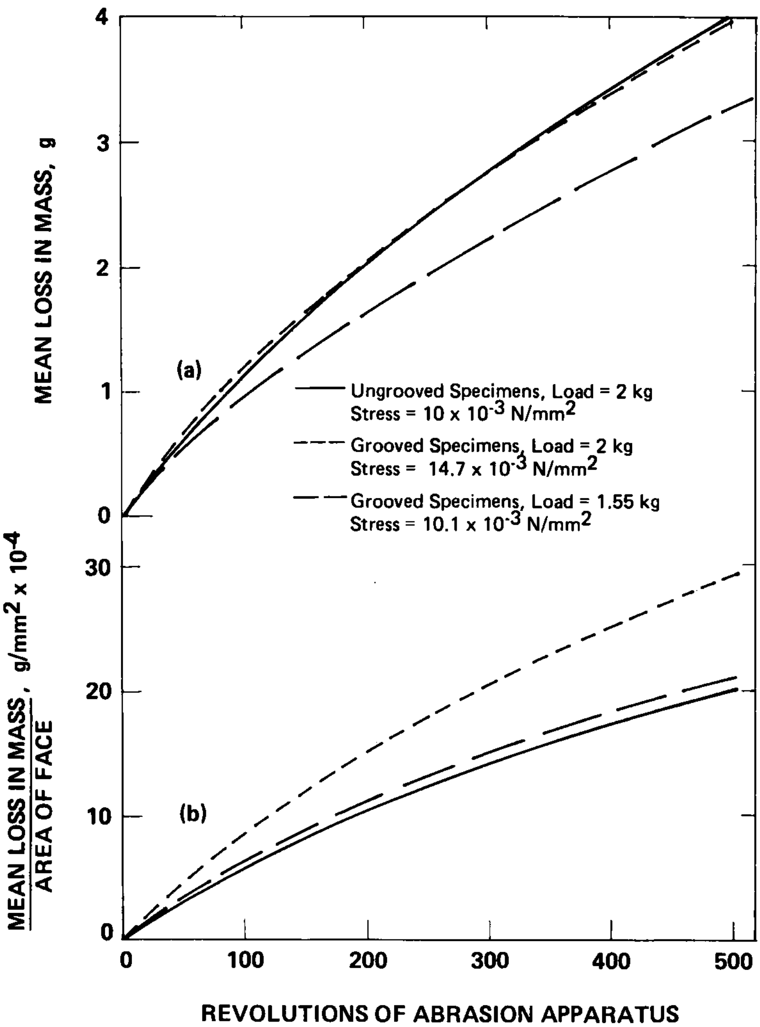

Figure 6 Wear on Grooved and Ungrooved Specimens during Abrasion Test

granolithic surface in 5 hours.

A similar problem is illustrated in Figure 7(b). An area of 1200 m^2 of good quality granolithic floor in a storage shed owned by Dundee Port Authority was causing concern due to heavily laden smooth wheeled fork lift trucks skidding on the polished floor. The Port Engineer's requirement for a method of producing a non-skid surface without causing structural damage to, or lifting of, the expensive granolithic screed was met by a Spetin-12 grooving machine. The work was completed in two working days at an average rate of between 70 and 100 m^2/hr. In this way a long lasting and safe working area for both vehicles and people was achieved in a very short time without the expense and disruption of re-construction or re-surfacing of the floor.

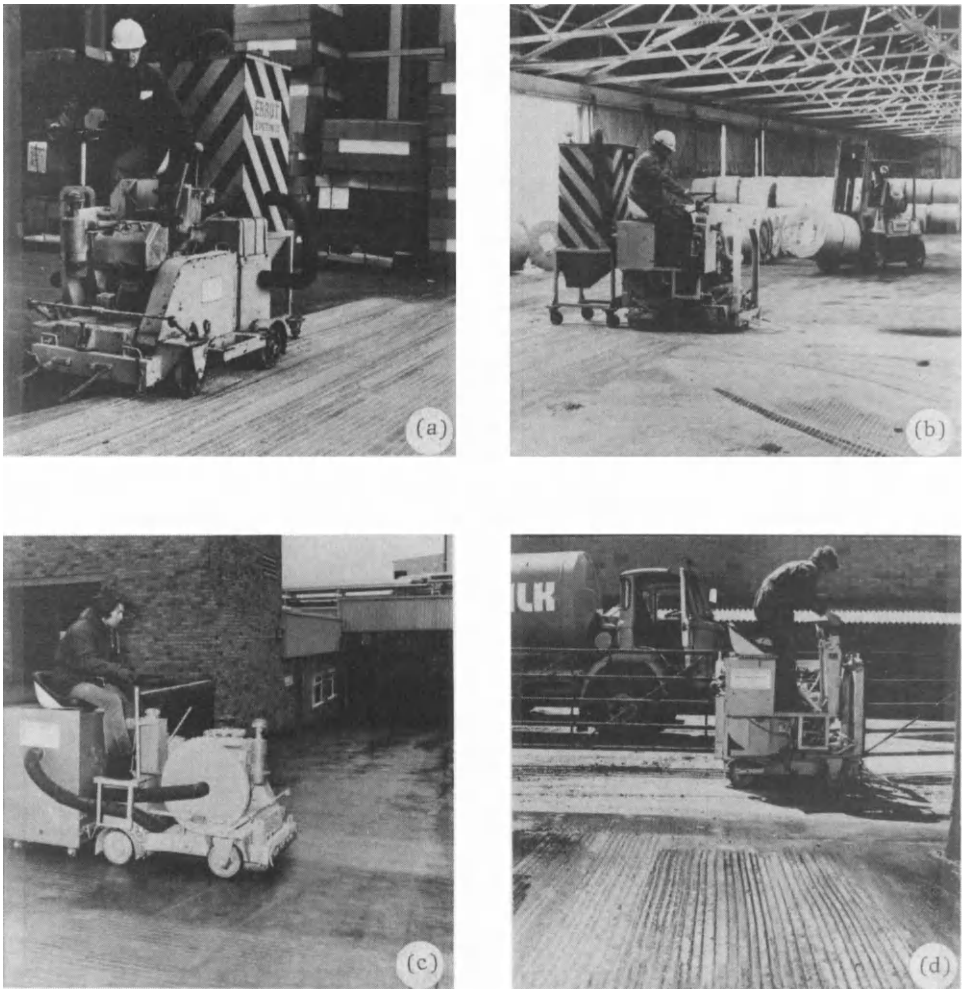

Figure 7 Typical Applications of Spetin Grooving

Figure 7(c) shows the Spetin-12 cutting transverse grooves on a concrete ramp with a gradient of 1 in 12. The ramp, which is 27 m long by 4.6 m wide is the connecting link between two cold stores. The contractor who had laid the ramp attempted to eliminate the problem of vehicles skidding by providing a brushed texture but this had proved inadequate as a protection against skidding in wet or greasy conditions. One of the main problems on this site was that the ramp is in constant use and the owners could not afford to have it out of service for any length of time. The work was eventually carried out by transverse grooving in half lane passes using the Spetin-12. Due to the small size of the machine, fork lift trucks were able to pass freely up and down the ramp while the work was in progress and the entire area, with grooves cut at 45 mm average centres, was retextured in only

4½ hours.

Cattle falling on slippery floors, possibly injuring themselves and having to be destroyed, is a problem that dairy farmers constantly face. The continual cleaning and scraping of cow slurry from livestock housing and collecting yard floors soon wears smooth the original tamped surface of the concrete. A large farm recently found that 1560 m^2 of concrete floor were in need of treatment and after considering many other methods decided to have the floors retextured by grooving, as shown in Figure 7(d). In the collecting areas and milking parlour where the problem was greatest, multi-directional grooves were cut, but on inside passages and feeding areas single direction grooving was found to be sufficient. When working inside, dust collection equipment was used on the Spetin-12 which carried out the work and because of this work could continue all day, even in close proximity to the cattle during milking. The whole job was completed in only three days with no disruption to the regular farm routine. Before the grooving was carried out, an average of 5 to 6 cows had been lost in the past two years at the farm. The total cost of the grooving carried out at the farm was well below the replacement cost of one year's losses so even in the first year there will be a considerable cash benefit; the additional saving of lost milk production makes the operation even more advantageous.

CONCLUSIONS

The system of tungsten carbide tipped tines used on the Spetin grooving machines makes it possible to apply just the right type of grooving required to overcome the particular problem in an area. In most cases simple transverse or longitudinal grooves will be all that is required, but the Spetin machines can produce any pattern of grooves to give a multidirectional non-slip texture to a floor at a cost which is well below that of applying a specialised chemical topping. In addition the non-slip finish of the grooved texture will last longer than that of any other normal topping, the grooves being effective throughout the life of the slab.

The Spetin system of concrete grooving is extremely effective and as shown by the microscopic examination does no damage to the remaining surface concrete of the slab which maintains a similar freeze-thaw durability and abrasion resistance to the original surface. The method of operation is speedy and lost time is kept to an absolute minimum since the grooving can be so programmed that the normal work of the factory, warehouse or farm can continue uninterrupted.

Since the process does not use water, no slurry is formed to cause problems in drainage or from dust when it dries. The availability of dust extraction on the Spetin equipment means that the process can be carried out without any harmful effects either on the working environment or on equipment or materials adjacent to the grooving work; even in the presence of highly strung cattle no problems have been encountered.

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FLAME CLEANING OF CONCRETE

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ABSTRACT The purpose of flame cleaning concrete surfaces is to improve the bonding of various finishes, such as plastics or mortar screeds, or to remove, for example, oil-staining from concrete floors to give an aesthetically pleasing appearance, or to increase skid-resistance. The studies have shown that flame cleaning with normal blowpipe speeds does not cause any significant deterioration of the basic properties of the concrete. Flame cleaning is thus a good method for cleaning concrete surfaces.

INTRODUCTION

The purpose of flame cleaning concrete is to improve the bonding of various finishes, such as plastics or mortar, on its surface. Flame cleaning can also be used for cleaning such surfaces as oil-stained concrete floors and to give an aesthetically pleasing appearance to concrete facades, as well as to increase friction on trafficked surfaces.

Flame cleaning is carried out by passing a special multi-flame oxygen-acetylene blowpipe (temperature approximately 3100°C) at uniform speed over a concrete surface, see Figure 1. Depending on the properties of the concrete a surface scaling or a partial melting of the surface layer is obtained. After flame cleaning, any melt residue or loose surface particles are removed with a wire brush, surface scaler or the like.

The Swedish Cement and Concrete Research Institute was commissioned to carry out a series of investigations to study in detail the effects of flame cleaning on concrete, in connection with the development of a new range of blowpipe systems, and the results obtained are described in this paper.

EARLIER INVESTIGATIONS

Flame cleaning of concrete has earlier been studied at Techn Hochschule in Aachen, Wesche (1,2), and at Techn Hochschule in Hannover, Weinhold (3). It was summarized in the first report (1) of the studies undertaken at Aachen that flame cleaning could be carried out on concrete made with diabase aggregate and that the same results could be expected with other rock aggregates. The temperature from the flame affected the concrete only a few millimetres under the surface.

Figure 1 Flame Cleaning a Concrete Floor
(The bright glare of the flame in the front of the blowpipe nozzles is due to the fact that the floor is stained with diesel oil which ignites during flame cleaning)

THE OBJECT OF THE STUDIES AND THEIR BASIC OUTLINE

The studies had two main objectives: a) to ascertain the positive and negative effects of flame cleaning on the concrete and b) to determine, for different qualities of concrete, what influence the speed of the blowpipe has on the effects of flame cleaning.

The studies undertaken involved both field and laboratory tests (4,5,6). The field experiments included measuring the rate of concrete removal at different blowpipe speeds, as well as surface scaling and the cleaning capabilities of flame cleaning on oil-stained concrete. The laboratory experiments embraced studies of any deleterious effects on the concrete, ability to remove oil stains, painted and plastic-coated concrete as well as the effects of flame cleaning on bonding to neat cement paste and epoxy resin. Tests were also carried out to determine what effect the individual properties of the concrete had on flame cleaning. Specially manufactured concrete slabs were used in which the following factors were varied: compressive strength, type and grading of aggregate, age, moisture content and temperature. A separate series of tests were performed to study the frost resistance of flame cleaned concrete surfaces.

THE MOST IMPORTANT FINDINGS

Cleaning Effect

The thickness of the concrete layer which is removed depends upon the speed of the blowpipe and the properties of the concrete, Figure 2. The most suitable blowpipe speed lies between 0.02 m/sec and 0.03 m/sec. Concrete removal takes place in the form of a spalling and a melting off of the surface. Of all the properties of the concrete, moisture content has the greatest effect on concrete removal.

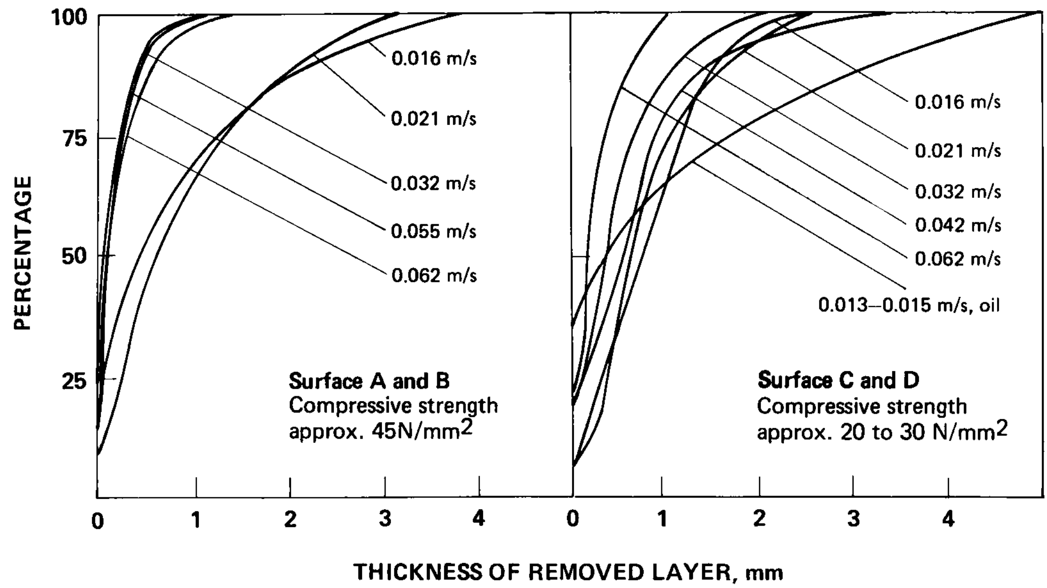

Figure 2 The Distribution of the Thickness of the Removed Layer

The laitance layer is removed to a depth of 1 or 2 mm, although at a few individual points a thickness of up to 4 mm can be removed. Paint layers of average thickness and 1 to 2 mm thick plastic coatings can be removed without problems. The surfaces of oil-stained slabs can be cleaned so effectively that, for example, plastic coatings can be applied directly to the concrete.

Completely dry slabs contained large areas where flame cleaning had not removed the material. Slabs that were soaked in water for several hours before testing, however, exhibited uniform concrete removal results. Slabs soaked in oil exhibited somewhat poorer surface scaling results. The laitance layer of the concrete exhibited a lesser degree of scaling in the form of pop-outs. Aggregate type, compressive strength and age of the concrete had no significant effect on the results of flame cleaning.

Temperature Distribution

Temperatures which have a deleterious effect on concrete (over approximately 200 to 250°C) were reached only in the uppermost 2 mm of the concrete at normal blowpipe speeds. At lower depths, temperatures were substantially lower; for example, at 7 mm below the surface, the temperature reached a maximum of approximately 70°C, as illustrated in Figure 3.

Bond

The bond of epoxy resin to flame-cleaned slabs was as strong or stronger than in the corresponding non-flame-cleaned slabs. This indicates that damage in the form of cracking cannot have been caused by flame cleaning. The bond of neat cement paste to flame-cleaned slabs was, almost without exception, stronger than on the corresponding non-flame-cleaned slabs. The standard deviation of the bond values is relatively large and can, at these compressive strength values, be assumed to be approximately 0.6 to 0.9 N/mm^2. In the further comparisons, it has been assumed that the spread for all the slabs is no more than 1.0 N/mm^2. Thus, in normal cases, a bond strength for flame-cleaned concrete of approximately 2.0 to 4.0 N/mm^2 or higher, depending on concrète quality, can be expected with both epoxy resin and neat cement paste materials. Lower values were obtained for some of the oil-soaked slabs, but the fracture always occurred in the concrete.

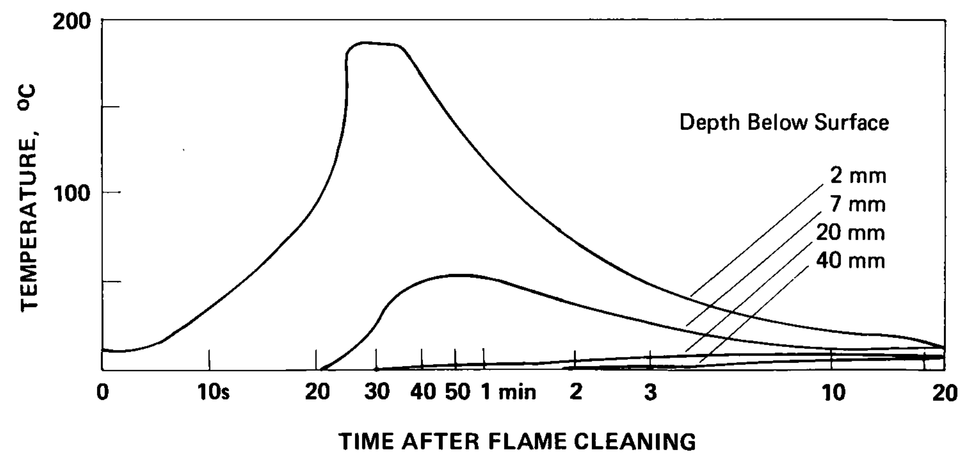

Figure 3 Temperature at Different Depths and Times

Cleaning Capacity on Oil-soaked Concrete

Two slabs, one with a high compressive strength concrete and the other with a low compressive strength concrete, were soaked with motor oil. In addition, one low strength slab was soaked with linseed oil and one high strength slab with diesel oil.

Intense heat was generated during flame cleaning of the slabs. On all the slabs except that soaked with linseed oil, burning continued for a while after flame cleaning had been completed. Test specimens were removed from the uppermost 10 mm of the slabs soaked in mineral oil (motor oil and diesel oil) 7 days after flame cleaning, and penetration depth and content of extractable material were measured as given in Table 1.

Assuming that all the pores in the cement paste are filled with oil and that the porosity of the concrete is approximately 10 to 12 per cent, the content of oil in the concrete before flame cleaning is estimated to have been between approximately 4 and 8 per cent, depending upon the coarse aggregate content in the test specimens extracted. Thus, it can be seen from Table 1 that the amount of diesel oil has decreased substantially due to flame cleaning. The reduction in motor oil content was more moderate, although the measured bond strength showed that the surface itself was free from oil after cleaning.

It was feared that oil in heavily soaked slabs which could not be removed by flame cleaning due to its depth of penetration could possibly be drawn towards the surface again and interfere with bonding. However, no difference was observed in

Table 1 Effect of Flame Cleaning on Oil-Soaked Slabs

COMPRESSIVE STRENGTH LEVEL	TYPE OF OIL	PENETRATION DEPTH mm	CONTENT OF EXTRACTABLE MATERIAL POSSESSING OIL-TYPE PROPERTIES AFTER FLAME CLEANING %
High	Motor oil	≈ 18	3.2
Low	Motor oil	≈ 12	2.4
High	Diesel oil	70 to 80 (partially soaked through)	0.1
Low	Linseed oil	≈ 26	

bonding between circular steel plates glued immediately after flame cleaning and those glued 7 days after flame cleaning.

Alkalinity

Previous studies have shown that concrete surfaces subjected to flame cleaning become as alkaline as new concrete surfaces, which in some cases can have some effect when the surface is painted. It is claimed that this alkalinity dissipates in air within 30 days.

Moisture Content of the Surface Layer

The moisture content of the surface layer of a water-saturated slab decreased from 5 to 7 per cent prior to flame cleaning to 4 per cent 2 hr after flame cleaning and 3 per cent 3 days after.

Tensile Strength in Bending

Tensile strength in bending was determined by subjecting the surface layer of beams sawn from a flame-cleaned and from a non-flame-cleaned slab to tensile stresses. A lower blowpipe speed (about 0.017 m/sec) was used for the flame-cleaned slab than for all the others. Moreover, the blowpipes were stopped in the same position for 10 seconds on this slab.

The purpose of testing the tensile strength in bending was to reveal any non-visible cracks perpendicular to the concrete surface and the results obtained are given in Table 2.

The spread of the values in Table 2 suggests that strength differences may be purely accidental, although other tests have shown that a certain slight reduction of tensile strength in bending can be expected. If this is caused by cracks, such cracks can be no more than a few millimetres deep. From a practical viewpoint, such very fine cracks are normally insignificant.

Table 2 The Tensile Strength (in bending) Results

BEAM TAKEN FROM	TENSILE STRENGTH IN BENDING, N/mm^2	
	Individual Values	Mean Values
Flame-cleaned slab	6.2 7.1 6.7 7.5	6.7 (0.6*)
Non-flame-cleaned slab	7.0 7.8 7.3 7.9	7.4 (0.5*)

*Standard deviation.

Frost Resistance

Fears that cracks could form the starting points for frost attack have often been expressed. The results of a study of cracks and frost damage in concrete in hydraulic structures indicate that such fears are groundless (7). The results of frost resistance tests showed that both flame-cleaned and non-flame-cleaned concrete possessed excellent frost resistance properties. Although the amount of flaking due to frost damage was greater for the flame-cleaned slab, from the frost resistance point of view it was considered to be insignificant.

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RE-BONDING OF FRACTURED ELEMENTS INCLUDING FLOOR SLABS

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ABSTRACT Concrete elements often show distress by cracking which, apart from being unsightly, may result in further downgrading causing structural inadequacy, and clearly requires remedial treatment. This paper is concerned with one such repair method, namely synthetic resin controlled injection. This technique (Structural Bonding Process) has been used to reinstate unreinforced and reinforced slabs and beams, as well as to integrate detached toppings and laminates to the base or slab concrete.

It is often difficult to: a) establish the form and nature of the failure requiring repair, b) specify materials and methods for adequate repair to be made, and c) establish the extent and degree of repair that has been achieved. This paper attempts to offer some quantitative data on all these aspects by making particular reference to a programme of experimental and practical work undertaken by the authors.

It has been shown that, in certain circumstances, reinstated repair strengths are greater than those of the original unfractured element. In addition the response of a resin repaired element to repeated load reversals is also described. The dependence of repair effectiveness on the bulk and flow properties of these materials, as well as their adhesion, are covered in an attempt to establish the limits of this type of treatment. Some data is presented that deals with induced resonant vibration response of fractured and repaired elements in an attempt to develop a non-destructive test method of failure/repair assessment.

INTRODUCTION

Concrete is not the inert, massive material that is often assumed. It is chemically sensitive, moisture bearing, porous and often non-homogeneous, stressed and subject to thermal movement. Concrete, be it precast, in the form of slabs, beams or panels, or placed in-situ, invariably shows some form of cracking during its useful life. The causes of cracking can be variable, for instance, drying shrinkage, plastic cracking, impact, static overload, thermal excursion. In other words, an unplanned for discontinuity may arise, preventing the concrete from acting monolithically. In addition, concrete in a fractured state, may degrade further, due to the combined effects of reduced strength and/or reinforcement corrosion. Such downgrading can be rapid and consequently repair procedures are necessary.

This paper collects together work carried out over the last two years, aimed at detecting, repairing and measuring the degree of repair for cracked, delaminated and otherwise distressed concrete items, including slabs. The work is still active and the results are in part of a preliminary nature.

DETECTION OF DELAMINATION

A very common problem is the detachment of toppings from the base concrete. It is so common, in fact, that it is surprising that a totally effective method of repair, or better, floor laying procedures that prevent the problem from occurring, have not evolved.

Before attempting repair, the extent of delamination has to be determined and this is often done by hammer tapping and listening for changes in response. This procedure, whilst reducing the contractor to his knees, and as such may have merit, is time consuming and subjective as far as interpretation of response is concerned. To overcome this problem, using initially an imposed vibration/response technique, a device that serves as a delamination detector has been developed. The imposed vibration/response technique or SHRIMP*, as it is known, has been reported elsewhere (1 - 3). With this apparatus, a series of tests were performed on a warehouse floor using known delaminated areas, fully attached areas and resin injected (repaired) areas. Both transient response and frequency scan tests were performed on all three types of area. Results from the transient response tests indicated that, for a constant force transient input, the delaminated areas responded with an amplitude of acceleration about one order of magnitude more than either the attached or repaired areas. Similarly the decay envelope is much increased, see Figure 1.

No doubt this was due to the scattering and reflection of internal sonic waves from the included fault boundaries back to the transducer. Similar results in the frequency domain were obtained for the same three areas and are shown in Figure 2. The trace sensitivity of Figure 2 should be noted. The amplitudes of the debonded area are tenfold greater than those of the other two traces. This work resulted in a portable device, capable of detecting delamination (refer next Section). The floor areas in question are traversed in a regular pattern, being given a fixed force impact as required. The response of the floor is measured and displayed on a 'go', 'no go' basis, which in turn relates to the decay envelope time base.

The principle of SHRIMP has also been applied to bridge deck delamination. A series of experiments were conducted on the underside of an elevated section of motorway to detect delamination at the interface of the prestressed concrete soffit planks and the in-situ concrete forming the carriageway base. It was supposed that frequency and wave shape differences would exist between integrally bonded and delaminated sections of the bridge deck and, as shown in Figure 3, marked differences do occur for the two conditions. This field test was extended to a series of laboratory tests for bonded and delaminated concrete slabs. The frequency scan comparison for debonded and bonded pairs of slabs are shown in Figures 4(a) and (b) respectively. Many slabs have been repaired in this way and all show the same general trend.

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*The acronym SHRIMP stands for Savage and Heierli Resonant Investigation Method - Patented.

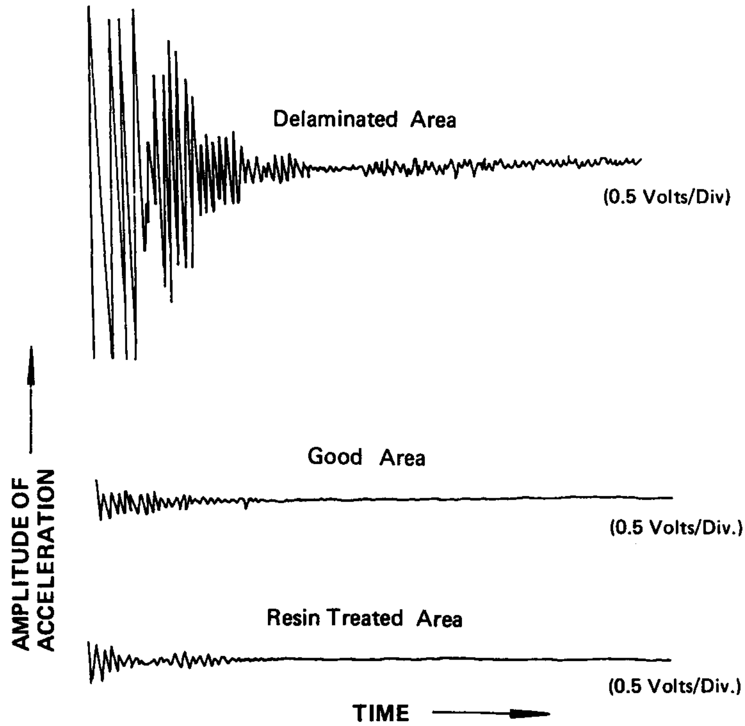

Figure 1 Time Response from Fixed Force Impact from Three Floor Areas

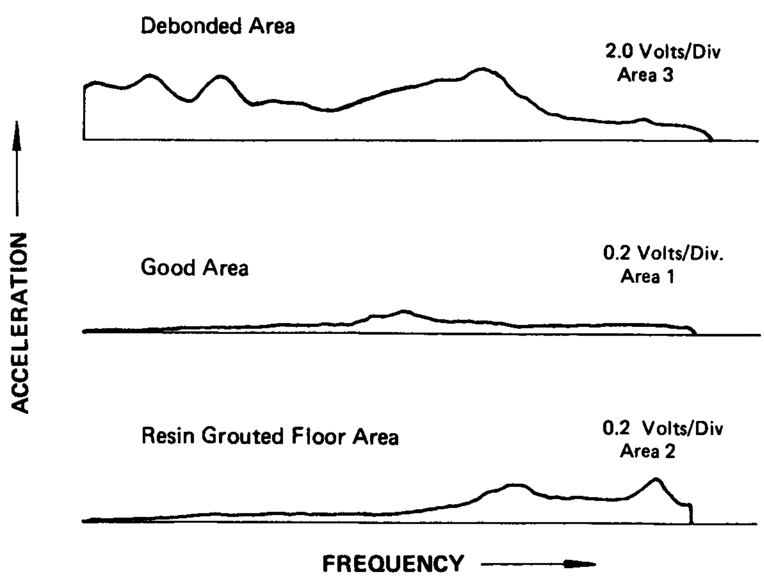

Figure 2 Swept Frequency Responses from Three Floor Areas, Frequency Range 422-1437 Hz

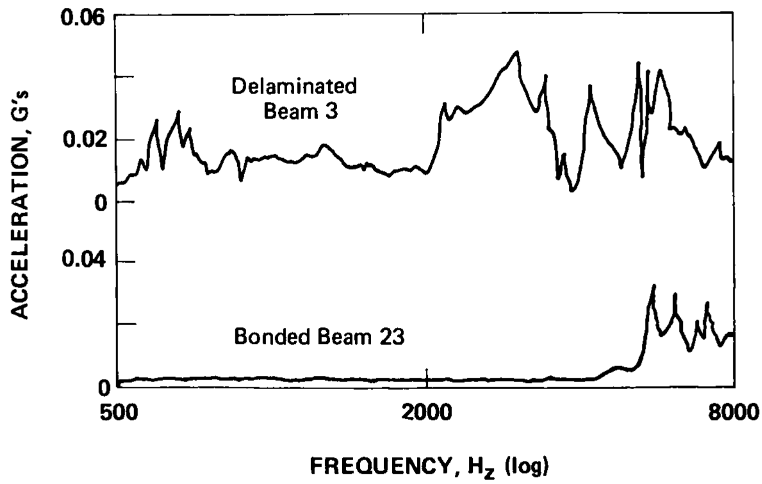

Figure 3 Responses of Delaminated and Bonded Concrete Planks

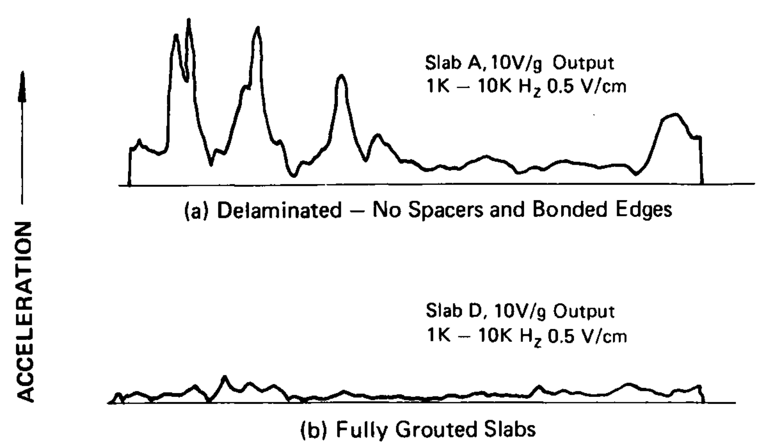

Figure 4 Delaminated/Bonded Slab Responses

Development of Delamination Detector

Theory

If a surface is impacted, local oscillations will occur at the surface's natural frequencies. The amplitude of these oscillations will decay exponentially at a rate determined by the elastic characteristics of the near surface. It has been found that if a good solid concrete surface is impacted by a unit force, the decay envelope will die away quite rapidly. However, for a surface under which delaminations and/or cracks exist, this decay time will be considerably longer, due principally to the many reflecting interfaces and boundaries usually associated with delaminations. A quick comparison test may, therefore, be made which will allow the user to identify areas under which delaminations have occurred, simply by timing this decay envelope.

Operation

The delamination detector is simply a device which will measure the period between two given amplitude levels of the acceleration decay envelope associated with an impact at a surface, (see Figure 1). The constant impact force is produced by an aluminium walking stick incorporating an automatic spring-loaded centre punch. The centre punch is set to a given compression value and the walking stick pressed home on the surface until the centre punch is activated. This is nothing more than the usual hammer test method, except that, in this case, the end of the walking stick comes into contact with the surface at the same instant of time as the impact is delivered. The handle end of the stick contains a sensitive accelerometer which records not only the initial shock wave of the impact but the ensuing reflections from beneath the surface. The accelerometer output is then transmitted to an electronic device which is carried by the operator. The initial peak starts a quartz clock and, when the signal amplitude falls below a selected percentage of this peak, the clock is stopped and the elapsed time is displayed on a liquid crystal. As each site situation will be different, it will be necessary to 'calibrate oneself in', to a given set of parameters, for instance, by deliberately varying the gain and decay time adjustments in good and bad areas of the site. By this means the delamination device can be used in almost every site condition.

DELAMINATION REPAIR BY RESIN INJECTION

The technique of epoxide resin injection and, in particular, that known as the Structural Bonding Process I*, has been reported elsewhere (4,5,6). Whilst suitable for medium/fine crack repair (0.01 mm - 10 mm), the SBP 1 technique was not made to cope with high resin injection rates and large volumes, often required for delamination repair, and it was desirable to develop an extension of the same principle using, in place of epoxide resins, polyesters (cheaper on unit volume basis) and developing a relatively high volume resin pump, having instant control over flow rate/pressure and minimum contact between materials and operator (known as SBP II).

The equipment developed for this purpose is shown in Figure 5 and consists of a conical hopper feeding a stainless steel gear pump, which is driven by a variable speed high torque electric motor. Batch mixing of the polyester is used, which, at high flow rates, is not the disadvantage it would be for fine crack injection. The technique is ideal for placing resin within a delaminated zone. It is compact, portable and can be used in confined areas. Various polyester compositions may be used, covering a range of cure times, flexibility and rheological properties (7). For instance, a non-Newtonian (semi-gell) may be preferred in wide cracks and delaminated areas rather than a Newtonian resin, normally used for finer gaps. The usual sequence of delamination repair is:

1. Location of delaminated area, drilling of access and venting holes;

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*Trade name of technique operated by Cementation Chemicals Ltd., Rickmansworth, Herts.

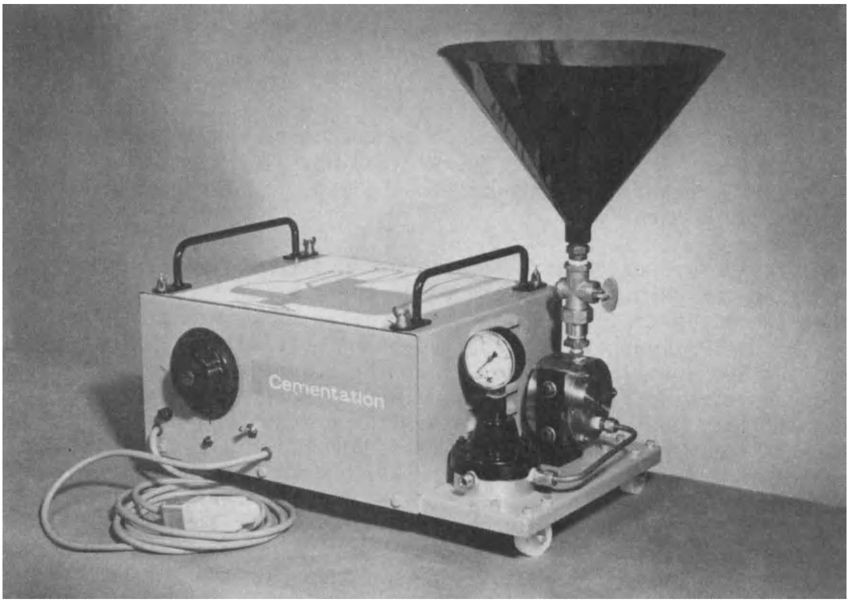

Figure 5 Polyester Resin Injection Pump for Delamination Repair (Structural Bonding Process II)

2. Temporarily plug injection holes and apply quick setting resin as crack seal;
3. Commence injection and, at point where resin vents, stop off using cork plugs.

When injection is finished, plugs are removed and, if required, the crack seal can also be removed, leaving no remedial scars, although this is not often necessary. The repair is permanent.

ASSESSMENT OF DEGREE OF REINSTATEMENT

Static Load Response

In reinforced concrete elements, cracks of small width, (hair line cracks) are generally of little consequence and can be tolerated without affecting the serviceability of the structure. Cracks of widths much more than about 0.5 mm (0.02 in), however, can seriously weaken the structure and increase the deflections under load, whilst in weathered structures they can increase the possibility of reinforcement corrosion, which leads to still further downgrading.

Clearly, however, there are uncertainties as to the size of crack and extent of cracking which can be repaired satisfactorily since, at a certain stage, irrepairable damage to the reinforcement, and debonding between the reinforcement and concrete, must ultimately occur. As far as repair methods are concerned, there is also a reluctance to use resin injection in dynamic load situations as it is supposed that the presence of a wedge of hardened resin within a crack can facilitate its propagation when the load is reversed.

The aim of the research work described in this part of the paper was to establish the viability of the resin injection repair technique (SBP I) in a variety of conditions for beams having proportions typical of commonly occurring structural elements, such as floor beams, rail supports, etc. The work included studies of the performance of repaired beams with cracks of both the tension (transverse flexural) and shear (diagonal) types when tested under conditions of both uni-directional sustained and multiple reverse loading. The general conclusions drawn from this work are regarded as relevant to the repair method generally no matter what the form of the element.

Most of the previous reported work into the effectiveness of resin repair methods has been carried out on beams with cracks of the tension type only. Chung (8) reports that the ultimate failure load of repaired beams was greater than the original. The deflections of the repaired beams were slightly greater than those of the original beams at lower loads within the normal working range. However, at higher loads approaching failure, the repaired beams actually appeared to be stiffer than before. Work resulting in similar conclusions was carried out by the Austrian State Testing Establishment for Building Materials (9). In this work only a small amount of additional cracking was noted, even when the beams were loaded to their original failure load. Celebi and Penzien (10) report on the behaviour of reinforced beams subject to cyclic reversed load before and after repair by resin injection. It was found that the loads required to cause yield in the repaired beams were considerably higher than for the initially undamaged case. There was no evidence of worsened propagation of the original cracks due to the presence of resin. In most cases the new cracks ran alongside the original cracks and the resin bond remained intact. The energy absorption characteristics of the repaired beams, as evidenced by the load-deflection hysteresis loops, were similar to those of the original beams. The repaired beams appeared to be considerably stiffer than the initially undamaged beams throughout the load range.

An item not covered in the previous work is the effectiveness of the repair technique in reinstating beams damaged with a predominance of diagonal shear cracks. This type of cracking is frequently encountered. The authors' research work sets out particularly to study the effectiveness of the epoxy injection repair technique (SBP I) in restoring the normal structural performance of beams damaged under abnormal load conditions and also to study more fully the case of beams with a predominance of shear cracks. The response to repeated load reversal was also studied. The experimental details and test procedures will be published elsewhere (11).

Results on Tension Cracked Beams

The load history is shown in the overall load - deflection curves given in Figure 6 and relates to the same beam (3a) as in Figure 12. Initially the beam was loaded in 2.5 kN increments straight up to failure (AB). The load was then released in increments (BC). The beam was then loaded again to obtain the damage load-deflection characteristics (CD). The beam was next repaired by the resin injection process and the loading sequence repeated (EF). The pattern of cracks resulting from the loading sequence is shown in Figure 7. At the maximum load of 57.5 kN, cracks 1 and 2 opened to about 0.3 mm. The outer cracks, 3, 4 and 6 were of insufficient width to be repaired and resin injection was only carried out on cracks 1, 2 and 5.

It will be seen from Figure 6 that the deflections of the repaired beam are somewhat greater than the original deflections at lower loads. However, the structural repair does seem to have stiffened the beam appreciably as may be seen when the behaviour of the repaired beam is compared with that of the damaged beam prior to repair.

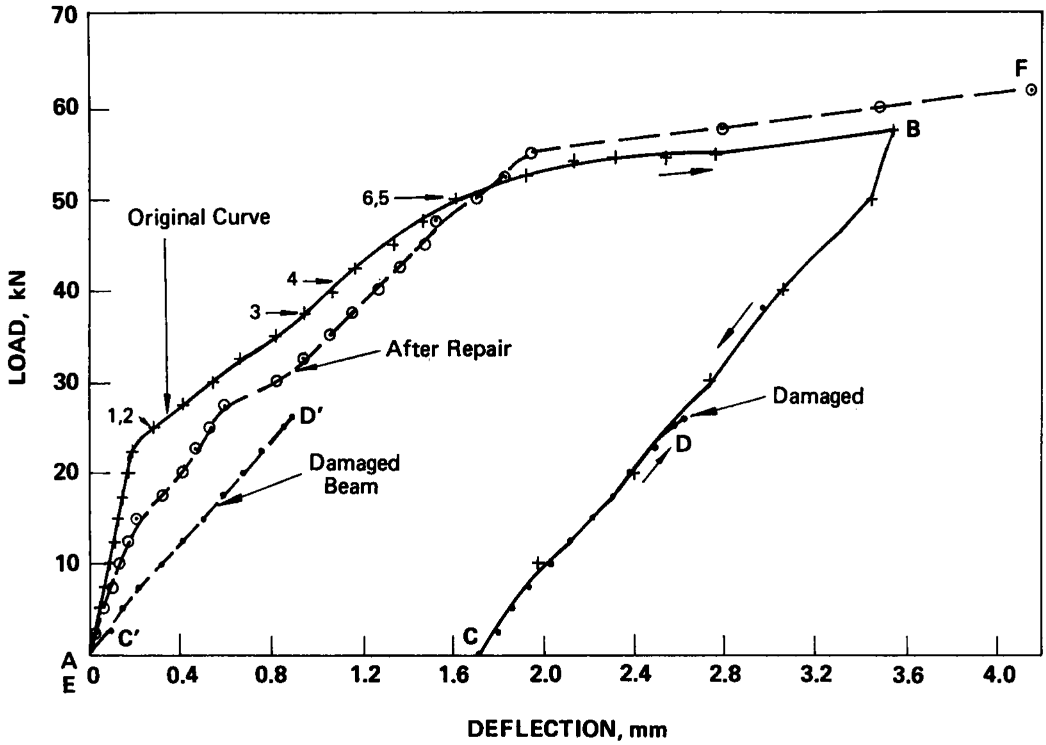

Figure 6 Load-Deflection Curves for Beam 3a (Tension Type)

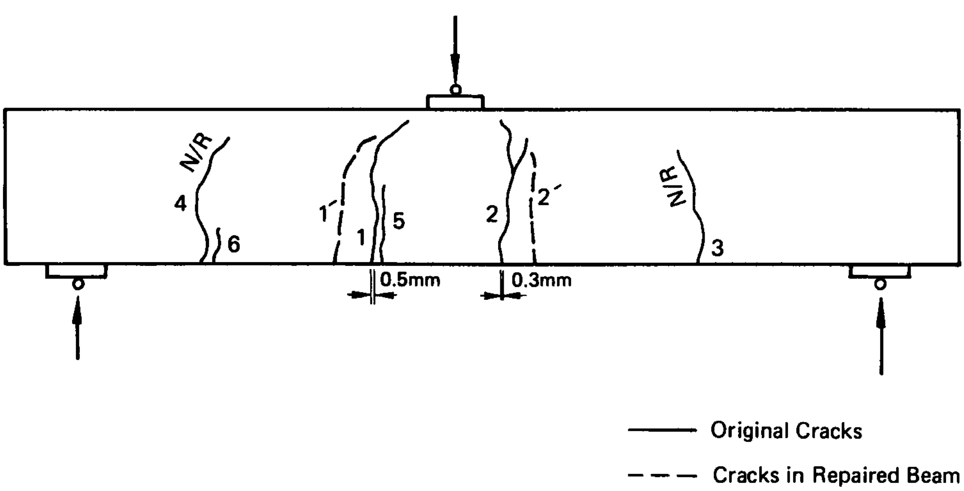

Figure 7 Crack Distribution in Tension Beam 3a

Results on Shear Cracked Beams

In this particular example the beam was initially loaded straight up to failure. Figure 8 shows the load deflection characteristics of the beam at the various stages of testing and Figure 9 shows the pattern of cracks produced. As shown in Figure 8, in the initial loading (AB), no cracking occurred until a load of 52 kN had been reached. Between 52 and 104 kN a number of hairline shear cracks appeared in the soffit of the beam. These extended only slightly as load was increased and widened little even at the maximum load applied. The first shear crack (number 10) was observed initially in hairline form at a load of 120 kN. As load was increased further, a number of shear cracks were formed as can be seen in Figure 8. At a load of about 180 kN the shear cracks began to open, the tension cracks remaining closed. At a load of 200 kN, shear crack number 10 opened to a width of about 0.4 mm. As load was increased through 250 kN, the principle shear cracks (10, 13, 20 and 24), had opened up to a width of about 1 mm. At 255 kN failure occurred suddenly and crack 20 opened to a width of about 4 mm. The load was then released (BC) following which a loading sequence up to 80 kN was applied to establish the load-deflection characteristics of the failed beam (CD).

The beam was then repaired by the Structural Bonding Process (I). Only the shear cracks were treated as it was found that the tension cracks were too narrow to accept any resin. The shear cracks, however, accepted a considerable quantity and it was found that these cracks were mostly interconnected, i.e. resin injected in one crack would appear eventually at various points over the beam. After allowing the resin to cure for the specified period, the beam was then re-loaded (EF). This time it was found that the beam was considerably stiffer than hitherto. The deflections of the repaired beam under load were actually less than those of the original beam. One or two small additional tension cracks (1' and 4', shown in Figure 9) appeared at lower loads. All the repaired shear cracks remained intact. At a load of about 190 kN, the first proper shear cracks occurred (7' and 8'). Eventually the load was taken up to the previous failure load of 250 kN. At this load only one new shear crack (number 8') showed any signs of opening, and there were no other signs of failure.

Dynamic Load Response

A series of reversed load tests was carried out on repaired beams as part of the present investigation. The aim of these tests was to establish the effectiveness of resin injection repair when the beam had been initially damaged by application of an abnormal load in one direction and then, after repair, subjected to a number of applications of working load applied in both directions. This is a fair simulation of a commonly occurring repair requirement in structures subjected to dynamic load reversals such as are caused by wind or wave loading or by moving loads on continuous beams, for example, overhead crane rail supports. As with the static load studies, both tension and shear beams were used. The reversed load sequence was applied as follows:

1. The beam was loaded in increments up to working load, applied in the same direction as the original load, after which the load was removed in increments, the load-deflection behaviour being noted throughout from dial gauge readings.
2. The beam was loaded in increments up to working load, applied in the reverse direction and then removed in increments.
3. Stages 1 and 2 were then repeated a number of times, load being applied in increments with the equipment

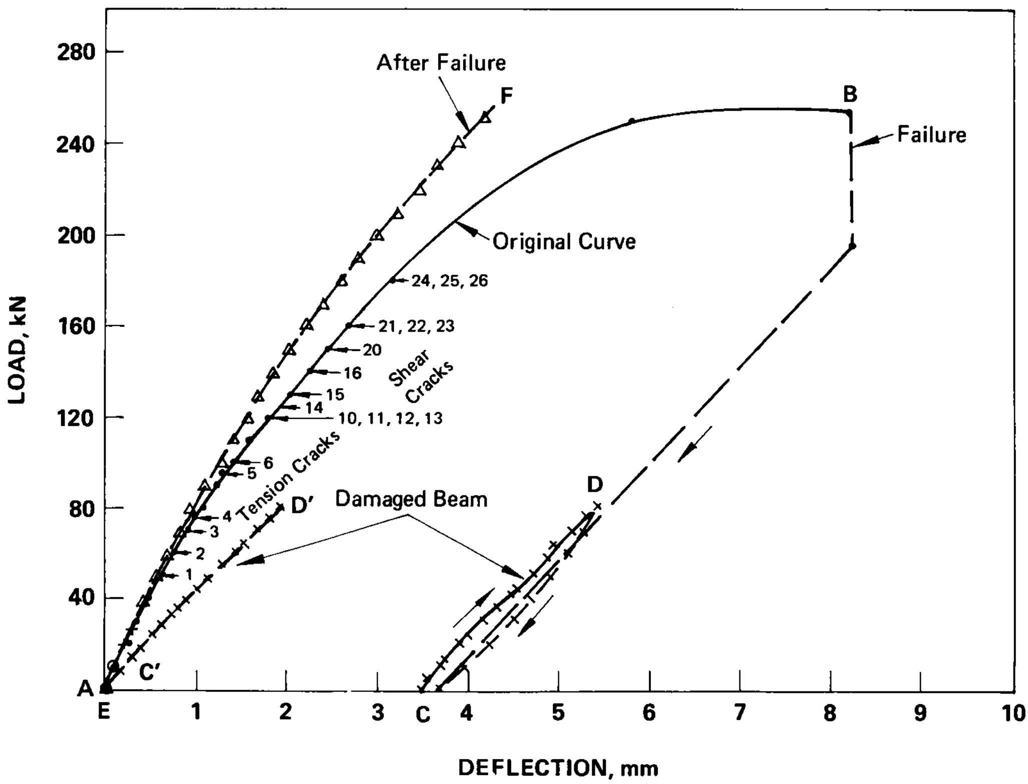

Figure 8 Load-Deflection Curves for Beam 4a, (Shear Type)

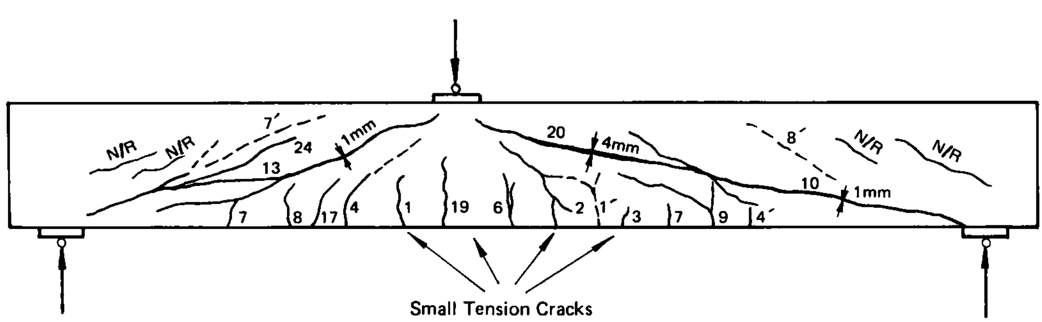

Figure 9 Crack Distribution in Shear Beam 4a

operated manually.

4. Automatic load cycling was then initiated and a number of cycled reverse loads applied at a maximum rate of about 3 cycles per minute. At various stages the cycling was stopped and stages 1 and 2 were repeated to obtain the load-deflection characteristics. This process was repeated up to 1500 load cycles.

The gradual increase in deflection with number of load reversals for a load of 20 kN is shown in Figure 10 for tension beam 5A and Figure 11 for shear beam 1A (cyclic load applied being 75 kN for the shear beams).

Vibrational Response of Fractured and Repaired Beams

A method of testing using oscillating low level stress waves ($\pm$ 0.5 per cent of yield) injected into a structure could provide advantages over point test methods since the low level vibration would excite the structure as a system. This test method also has the advantage of exhibiting clear unmistakable responses in the frequency domain, unlike the difficulties with small measurements obtained from static load-deflection tests. The method referred to is known as SHRIMP (refer section on Detection of Delamination).

Test Programme

The object of the programme was to evaluate the relative structural integrity of reinforced concrete beams, whole, fractured and repaired (1).

The results of vibration testing before and after static loading of a series of beams and after repair by injection of an epoxide resin (4, 5, 6) using SBP I are given in Table 1.

Table 1 Results of Vibration Testing Before and After Static Loading and After Repair for Both Tension and Shear Beams

BEAM TYPE	BEAM NUMBER	BEFORE CRACKING 3RD MODE, Hz	AFTER CRACKING ELASTOPLASTIC 1ST MODE, Hz	AFTER REPAIR 3RD MODE, Hz
Tension	2A	1445	36	1325
	2B	1419	37	1362
	3A	1400	36	1326
	3B	1440	37	1335
	5A	1448	NST*	1448**
Shear	1A	1700	NST*	1655**
	1B	1710	26	1508
	4A	1670	32	∅
	4B	1615	NST*	1595**
	5B	1690	NST*	1690**

* Not statically tested
∅ Repaired but not vibrated before 2nd static test
** Frequency of unloaded beam

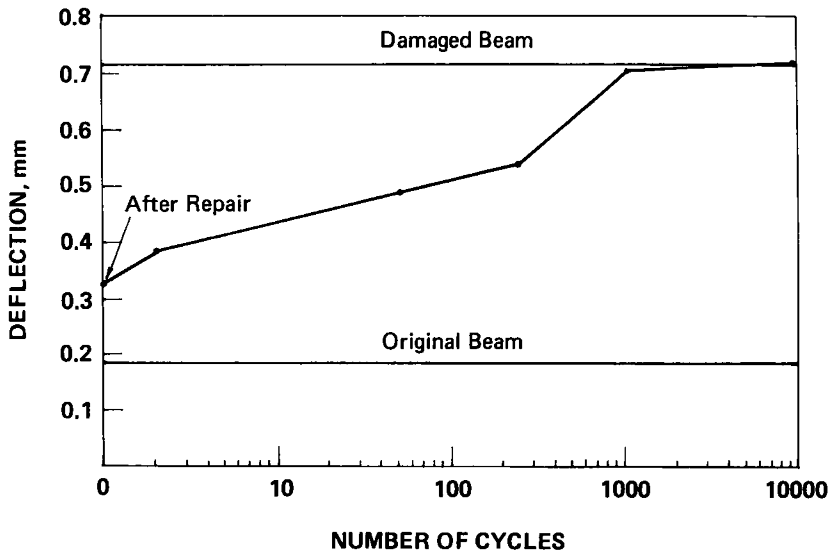

Figure 10 Deflection under Repeated Load Reversal for Tension Beam 5a (Imposed Static Load : 20 kN)

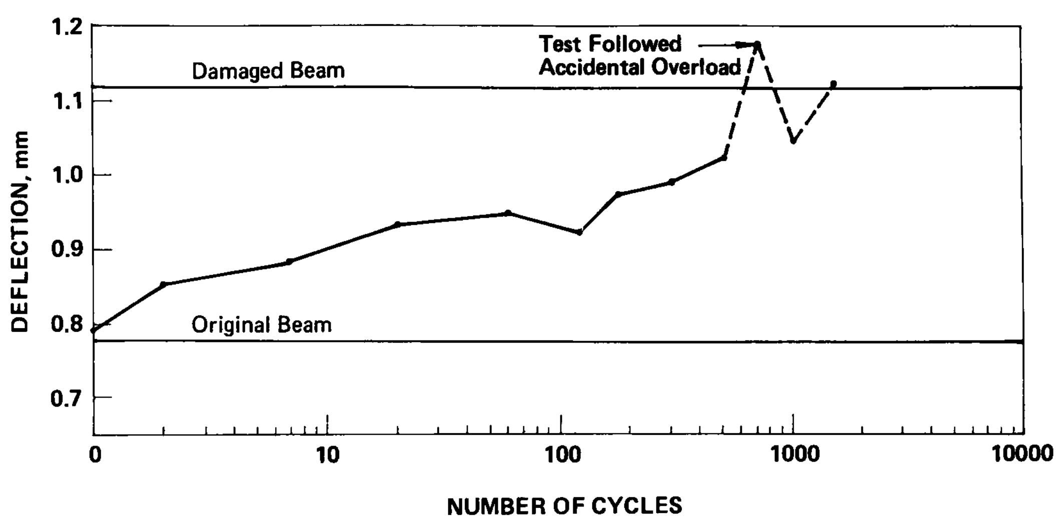

Figure 11 Deflection Under Repeated Load Reversal for Shear Beam 1a (Imposed Static Load : 60 kN)

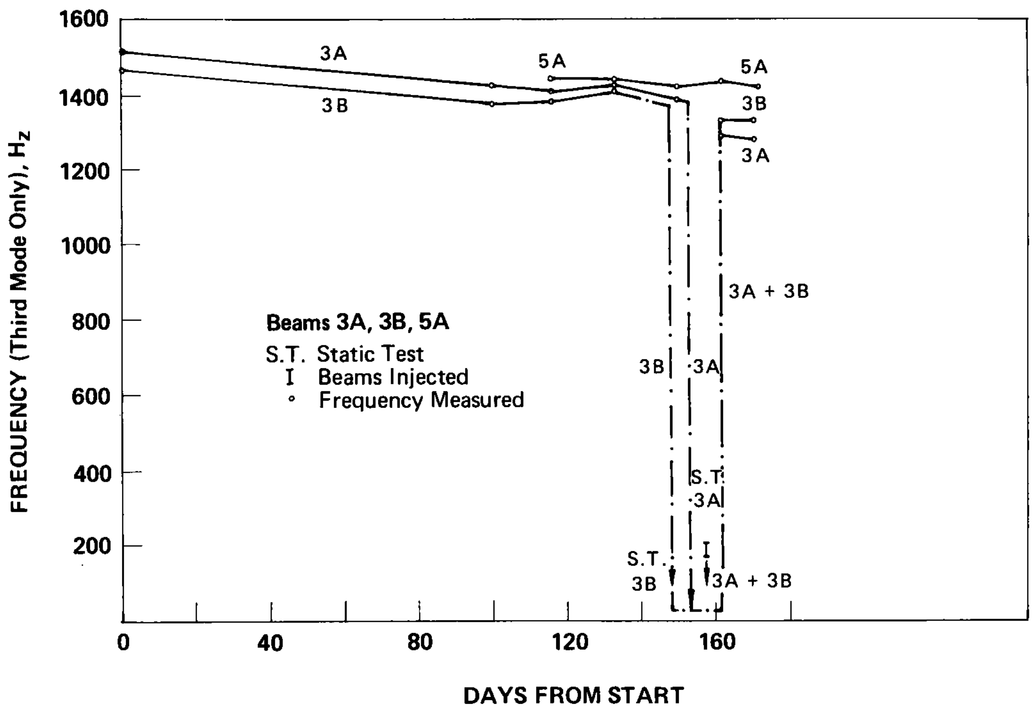

Figure 12 Frequency-Time Relationships for Statically Tested and Untested Beams (Tension Type)

Graphical results of several of these completely tested beams, all of which followed a similar pattern, are given in Figure 12 which shows the initial third mode frequency before the static bending test and the third mode recovery after injection. For the tension beams the level of third mode frequency recovery averaged about 94 per cent and for the single shear beam about 88 per cent. There appears to be a relationship between these recovery percentages and the tangent load-deflection slopes between the initial and repaired states of the beam. The level of oscillating stress induced by vibration is negligible and on the load-deflection (or stress-strain) diagram would be represented very close to the origin and, therefore, would not reflect the usual non-linearity shown by repaired beams.

Future tests will take into account these non-linearities, since it would be an easy matter to apply a vibrational input to an incrementally loaded beam and monitor the change in a given modal response for an induced static stress condition. Even so, results indicate that more than an order of magnitude difference in frequency occurs when a beam is severely damaged. The behaviour of the severely damaged beam can be represented as a damped torsional centre spring with two rigid-body masses attached to either side. The rigid-body masses are the intact outer parts of the beam; the spring is the damaged centre portion.

CONCLUSIONS

The results show that the structural integrity of components, and hence structures, can be assessed by the SHRIMP method of non-destructive testing. The method excites system response, which provides a characteristic fingerprint and against which future fingerprints can be compared for signs of deterioration. The same method can be used as a diagnostic tool once deterioration has been detected and, with carefully controlled calibration tests, strength, integrity and locked in stress of individual components can be assessed directly on the structure. A variation of the SHRIMP principle may be used to detect delamination as well as indicating the degree of repair achieved. As an extension to the vibration response work, a prototype delamination detector has been built and is undergoing evaluation in the field. Repair of delamination, using polyester resin injection, has been shown to be both practicable and effective. A novel resin injection pump is described.

It has been established that: a) epoxy resin injection repair is effective in restoring the structural integrity of failed and badly cracked articles such as beams, b) the ultimate strength of beams can actually be increased by the repair method, c) commensurate with the above, the repaired beam is somewhat stiffer than the unrepaired beam at loads approaching the original failure load, although at loads within the normal working range the repaired beam deflections are greater than in the initially uncracked beam in the same load condition, and d) the technique is effective also in conditions of reversed loading and the repairs do not themselves propagate further cracking by wedging.

It is possible to reinstate badly cracked reinforced beams by the resin injection process known as Structural Bonding Process I. Bending characteristics close to those of the original beam are found in damaged beams if satisfactory repairs have been effected, i.e. accessible by the injected resin. Resin repair is particularly effective in reinstating beams with diagonal shear cracks. Satisfactory repairs can be achieved in very badly damaged beams and the repaired beam is often stiffer and stronger than the original beam.

For beams with tension cracks, it would appear that satisfactory results can be achieved if the crack widths are not greater than about 1 mm. The stiffness of repaired beams in the working range is slightly less than originally but the repaired beams exhibit a higher failure load.

There are limitations to the effectiveness of the technique. If crack widths are very large, then the reinforcement within the beam or slab may be in a damaged condition and the resin repair will then not hold under load. Conversely, if the cracks are too narrow for proper resin penetration, there will be no improvement in the beam's stiffness but such cracks are not likely to affect the serviceability of the beam in any case.

There are no signs from the performance of either the tension or shear cracked beams tested that the resin repair itself worsens the situation by creating new cracks due to wedging. All the beams tested suffered a slow deterioration in stiffness after a large number of load reversals. Since no initially undamaged beams were tested as controls it cannot be established from the results of the tests carried out so far that this was due solely to a failure of the resin. Even though the deflections of the beams increased under repeated load reversals, there were no signs of any significant increase in the energy loss in each load cycle, as deduced from the load deflection hysteresis characteristics.

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REPAIR OF CONCRETE BALCONY SLABS

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ABSTRACT More than 200000 concrete balconies in Sweden are badly damaged by frost attack and corrosion of the reinforcement. Moreover, although many of these balconies have been repaired, the repair work, which normally consists of applying various types of plastic coating, has often failed within a few years. This paper describes a research project in which the durability of different repairing systems is being studied in the laboratory and in the field. The laboratory studies investigated the frost resistance, the thermal gradient sensitivity and the diffusivity of the repairs, while the field studies were directed at understanding the general behaviour of different repairing systems. In addition, the moisture state in the structural concrete slab beneath different repairs is being studied. Some preliminary results are given indicating that it is very doubtful if any repairing system can protect a low grade concrete slab from continued damage.

INTRODUCTION

During the years 1940 to 1960, about 800000 concrete balconies were built in Sweden. The balconies in most cases were designed as cantilever slabs with top reinforcement and were constructed normally as a two-course floor, the upper part being a thin concrete topping directly on the structural concrete or separated from this by a bituminous moisture membrane. Sometimes the slab was covered with ceramic tiles placed in mortar which was in turn placed on a moisture membrane. The three free sides of the balcony were provided with a barrier that was normally fastened by steel plates cast into the edge of the slab. A typical balcony is shown in Figure 1.

Since the balconies hang outside the facades they are exposed to severe climatic conditions such as direct precipitation, water from driving rain, water from melting snow and low freezing temperatures. The climatic exposure is such that it can result in a high moisture content throughout the slab in combination with low freezing temperatures. Moreover, air-entrainment was seldom employed, as a matter of fact it was not even prescribed in the then current Swedish concrete regulations, and the concrete was often of rather poor quality with a water-cement ratio above 0.70. The risk of frost damage was, therefore, very high in many balconies, with associated corrosion of the reinforcement taking place earlier and with greater speed than what would happen in a sound concrete. In many slabs the cover of reinforcement was insufficient, especially over the very frequently used, though unnecessary, bars running along the edge of the slab, see Figure 1. In some cases the concrete seems to have been damaged from the beginning due to early freezing.

Today, more than 200000 balconies are in such a bad condition that they need extensive repair, or even replacement. By far the most frequent damage is at the slab edges, consisting of spalling of the concrete cover and scaling and cracks in the structural concrete. In many cases the damage is so deep that the barrier has become loose. Other types of damage are also present, viz. i) corrosion of the top reinforcement in single-course slabs accompanied with spalling of the concrete cover, ii) severe frost damage of the concrete surface beneath the moisture membrane in two-course slabs or slabs covered with ceramic tiles, probably partly due to ageing of the membrane, iii) complete frost destruction of the mortar for ceramic tiles and iv) cracking and spalling in the lower part of the slab.

Many balconies have been repaired, for example, by the application of a plastic coating to the slab, but in many cases these repairs failed within a few years. The Swedish Cement and Concrete Research Institute has been involved in an extensive research project aimed at finding methods for analysing the type and degree of damage on balconies and recommending methods for repair*. One part of the project, which is concerned with the durability of different repairing systems and is currently in progress, is discussed in this paper and some interim results are presented.

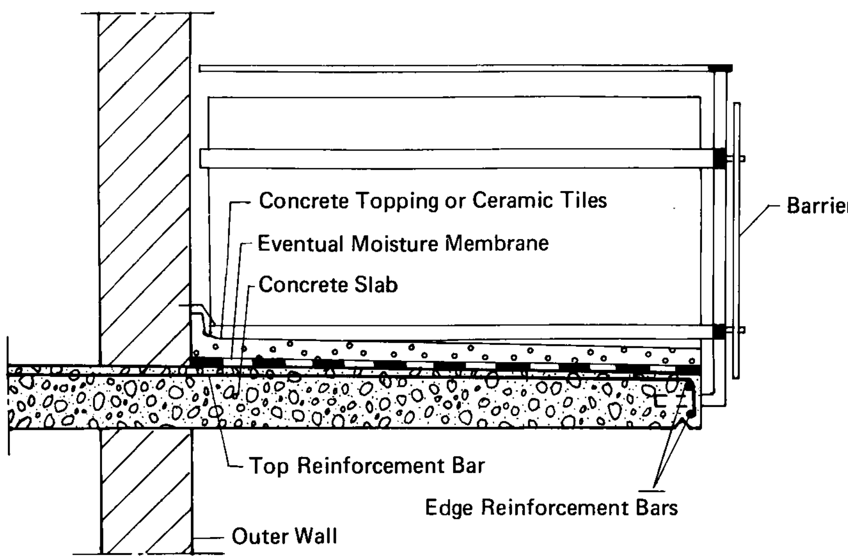

Figure 1 Cross-Section through a Typical Balcony

FUNCTIONAL REQUIREMENTS OF A REPAIR

A balcony slab in which the concrete cover over the top reinforcement is completely carbonated over a large area is not repaired but is totally replaced; the same is valid for a balcony slab that is severely frost damaged in its interior. Balconies

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*The leader of this project is Lars Johansson at the Swedish Cement and Concrete Research Institute.

with more local damage could be repaired, for example, balconies with frost destruction of the concrete topping but almost intact structural concrete and/or destruction around the edge of the slab.

Figure 2 shows a slab that has been provided with a new topping and a new edge. Although it is assumed that all bad concrete is removed, it is possible that the reinforcement may be surrounded by carbonated concrete in some places. The main durability problems to consider, see Figure 2 are:

1. Frost damage of the new repairing material leading to accelerated carbonation and/or corrosion of the structural reinforcement.
2. Frost damage of the structural concrete in its interior or in the old concrete cover leading to a reduced load carrying capacity of the slab.
3. Corrosion of the structural reinforcement leading to spalling of the concrete cover or to a slow reduction in the load carrying capacity of the slab due to reduction of the steel cross-section.
4. Corrosion of the cast-in steel plates carrying the barrier.
5. Aesthetic damage, such as discolouring, cracking, crumbling, or scaling of the new surface.

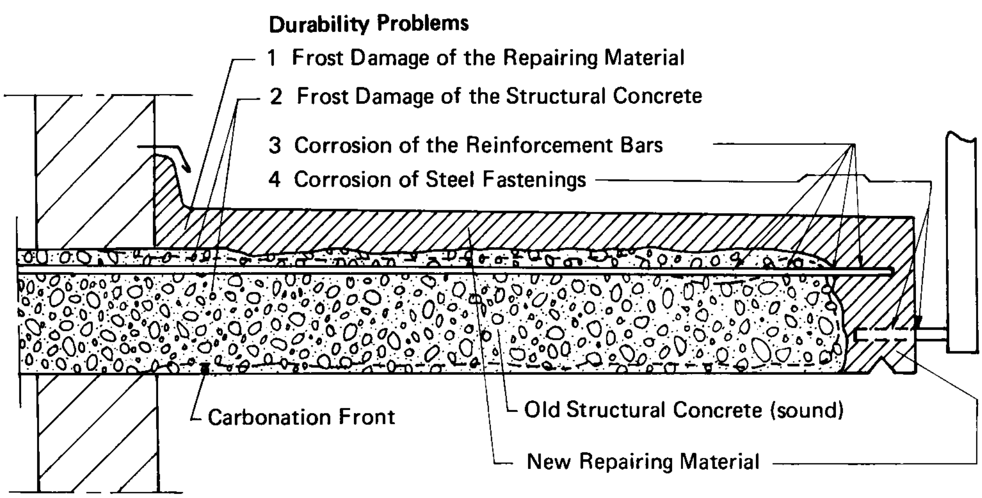

Figure 2 Types of Damage Possible in a Repaired Balcony

The functional requirements of the repair, apart from those allied to aesthetic damage, can be formulated after studying the different destruction mechanisms; Figure 3 shows schematically the conditions necessary for different types of attack.

1. Frost damage in a material occurs when the degree of saturation of the pore system, S_{ACT}, exceeds the maximum or critical degree of saturation, S_{CR} (1). The repairing material has free access to water from the surface. Thus, in order

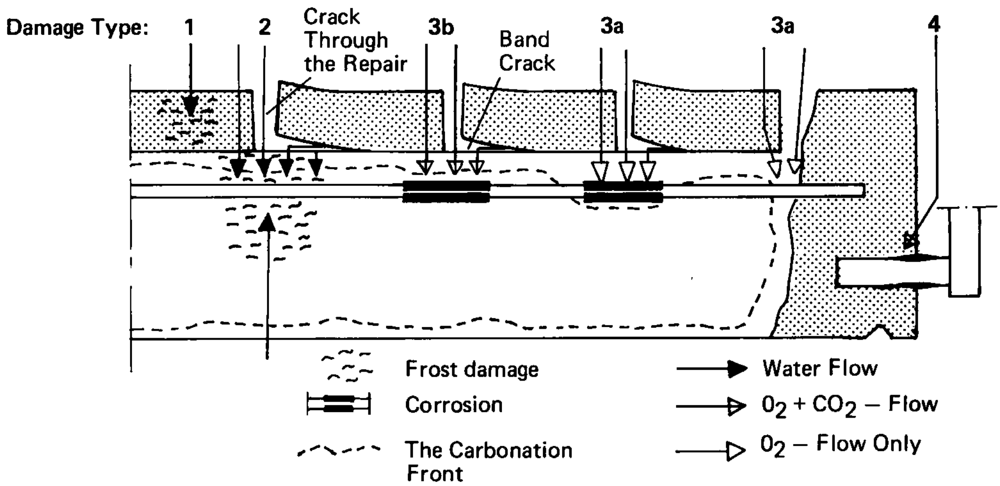

Figure 3 Different Flows Determining the Rate and Extent of Damage

to be frost-resistant it must be designed in such a way that its own S_{CR} value exceeds the maximum possible S_{ACT} value reached after a long time of free water absorption.

2. The structural concrete in the balconies often does have a high water-cement ratio, and in addition lacks air-entrainment. Thus, in the actual wet environment it must be assumed that all unprotected slabs will be saturated more than critically (2). The repair must, therefore, reduce the moisture intake of the slab to a level below that in an unprotected slab. High moisture contents can be expected when i) the repairing material has a high capillarity, ii) the repair cracks, iii) the bond between repair and slab breaks down and iv) capillary water condenses below the repair when the temperature drops. The last three mechanisms are especially pronounced for impervious repairs which hinder the slab from drying out once it is moistened.

3(a). A reinforcement bar that is embedded in a carbonated concrete will corrode when the relative humidity (R.H.) close to the bar is equal to or greater than 80 per cent. The rate of corrosion for high R.H. values ($\geqslant$ 90 per cent) will be governed by the rate of oxygen flow to the bar (3). This is in turn a function of the permeability and thickness of the repairing material, and, of course, of the water-cement ratio and water content of the concrete cover (3). At a certain stage of corrosion, the concrete cover spalls off, or the cross-section of the bar becomes too small. The life-time of the bar can be estimated when the oxygen supply to the bar is known (3). It depends on the following factors: i) the diffusivity of O_2 ions through the repair (which can be highly increased when the repairing material is frost damaged), ii) cracks through the repair and iii) cracks between the repair and the concrete slab.

3(b). When the concrete cover is only partly carbonated the reinforcement is protected against corrosion, unless, of course, no large cracks lead to the reinforcement or the Cℓ ion content is above the threshold value. Corrosion will initiate when the carbonation front reaches the bar. The rate of corrosion after that time will be governed mainly by the oxygen flow rate. The life-time of the bar can be estimated when the flows of CO_2, and O_2 are known (3), which will depend on the factors described in 3(a).

4. An ordinary steel plate cast into a porous material, for example, concrete, will corrode in all parts where the steel is not passivated. Thus, a steel plate cast into concrete will immediately corrode close to the concrete surface, the corrosion then spreading inwards as the carbonation front proceeds. The corrosion will cause cracks in the concrete around the steel plate and all fastenings in concrete must, therefore, be made of austenitic stainless steel. In other porous materials, with lower pH values, galvanized steel can be used. Ordinary steel can, however, be used in impervious materials since the rate of corrosion will be very low in such materials.

The functional requirements of the repair thus consist of the following:

a) The repairing material must be frost resistant in itself; that is, its actual degree of saturation reached in the actual environment must be lower than its critical degree of saturation.

b) The repair must protect the structural concrete from being critically water saturated with regard to frost damage during the whole required life-time.

c) The repair must limit the diffusion of CO_2 and/or O_2 into the structural concrete to values that can be accepted with regard to the maximum allowable corrosion of the reinforcement.

Requirement a) is in a way a direct consequence of b) and c), since it is improbable that a frost damaged repair should be able to protect the structural concrete and the reinforcement. In fact, requirements b) and c) imply that the repairing material is durable in many other respects, for example, against ultra violet radiation, against the high pH values in wet concrete and against variations in temperature and humidity.

Bond between a repair layer and the concrete slab is not a general requirement; in cases where the concrete cover is intact it could even be desirable to separate the new layer from the old concrete by an air-filled space making it possible for the concrete to dry upwards. On the other hand, separation of a coating which was initially bonded to the concrete slab could be disastrous since the horizontal crack so formed could lead water and oxygen into the interior of the slab at the same time as drying out is hindered.

PRACTICAL EXPERIENCE OF THE DURABILITY OF BALCONY REPAIRS

One part of the project consists of an examination of old repairs*. About 1000 previously repaired balconies and terraces belonging to 30 buildings were studied. The age of the repairs varied from 1 to 15 years and the examples were selected to include most of the commonly used repairing methods. The result of the inspections was rather disappointing; not more than 4 were considered to be intact while the other 26 repair types were damaged, some of them badly, with 15 of these being not more than 5 yr old. Typical forms of damage observed were: completely detached plastic or mortar coatings on top of the balconies or at their edge; cracks in these coatings; rust stains on the surfaces; and spalling of the concrete cover exposing the reinforcement bars. In some cases the damage could be attributed to bad pre-treatment of the old concrete. Many of the repairs were just cosmetic and did not significantly reduce the rate of destruction.

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*This part is carried out by the Swedish contractor STABILATOR.

FIELD STUDIES OF THE MOISTURE STATE IN BALCONIES

The durability of a balcony is highly dependent on its moisture content. Therefore, in a limited study the moisture level in 8 typical balconies belonging to 4 houses (B, F, G and H) were investigated over a period of 1½ years. Four balconies, in houses G and H, were repaired and four, in houses B and F, were unrepaired. The moisture measurements were made at 4 points on each balcony, one at the outer corner, one at the front edge and two at the central part of the slab, see Figure 4(a). The depth of the measurement was about 60 to 70 mm above the bottom surface at points 2, 3 and 4 and 40 mm at point 1, with the largest depths being just below the moisture barrier.

The measurements were made by inserting a relative humidity sensor into a plastic tube placed at a desired depth in the slab, isolated from the surrounding concrete by silicone rubber, Figure 4(b), and closed with a rubber stopper during the long intervals between measurements. The sensors have a high degree of accuracy for R.H. values below approximately 98 per cent, but above this level it was necessary to resort to a different type of measurement.

The results obtained were by no means unequivocal; no balcony was found to be significantly wetter than the other and the difference in wetness between two balconies of the same type on the same house was often very large. The micro-climate was evidently very important. In all houses except one (house G) the outer edge and the corner were, however, significantly wetter than the interior of the balcony, with the mean R.H. values for the whole period being approximately 94 and 83 per cent respectively. It is worth noting that the relative humidity very seldom was found to be 100 per cent which means that the slabs were normally not fully saturated. The risk for frost damage in the interior of the structural concrete should, therefore, not be so high. On the other hand, the R.H. values are such that a very high rate of corrosion could be expected (3). Moreover, the possibility cannot be excluded that the slabs were fully saturated intermittently, leading to frost damage, since the measurements were made under rather favourable conditions. Examples of the results obtained for point 3 are shown in Figure 4(c), in which the mean R.H. values for outdoor air in Stockholm are also shown. The moisture level was normally much higher in the slabs, presumably due to capillary suction during rainy periods. In two cases the slab was drier than the air which might be due to the drying out caused by solar radiation.

LABORATORY TESTS

Scope

The potential durability of thirteen principally different repairing systems was studied using laboratory methods. The following properties were investigated:

1. Sensitivity towards temperature gradients.
2. Frost resistance of the virgin repair (uncracked specimens were tested),
3. Frost resistance of the aged repair (pre-cracked specimens were tested),
4. Moisture diffusivity of the repairing materials and of the concretes, with CO_2 and the O_2 diffusivities being estimated from the H_2O diffusivities.

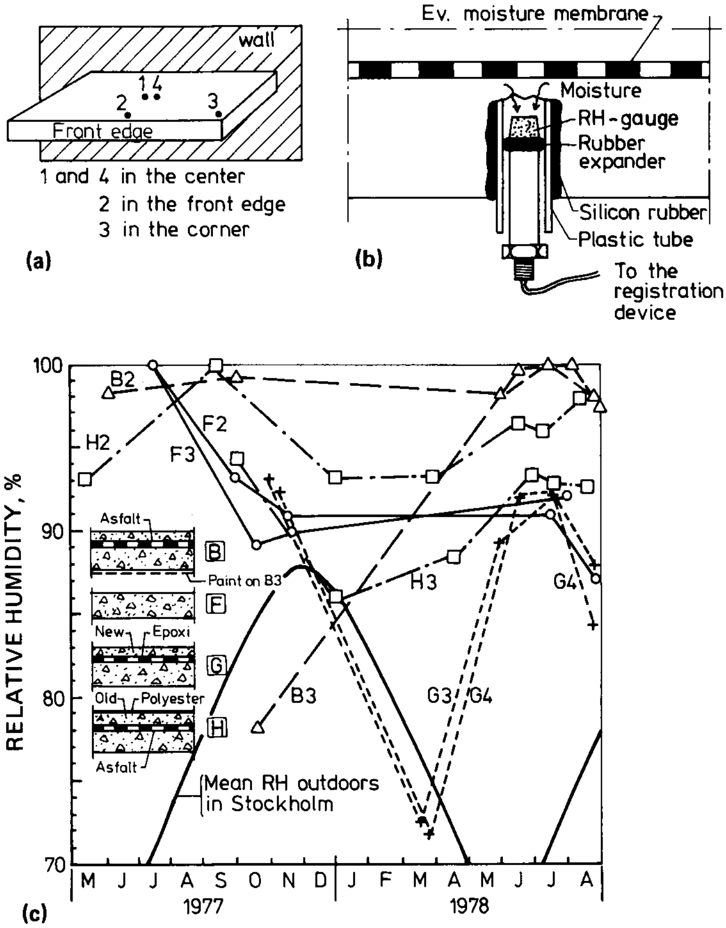

Figure 4 In-situ Measurements of Moisture in Balconies (a) Location of the Measurement Points (b) Method of Measurement (c) Results for Point 3

The repairing systems investigated were:

A. 35 mm concrete topping, water-cement ratio 0.45 and air content 8 per cent. Grout washed concrete.

B. Same as A but glued to the concrete with epoxy resin.

C. 40 mm polystyrene-polybutadiene-latex mortar topping, air content 7 per cent.

D. Coating of approximately 2 mm polyurethane modified metacrylic resin.

E. Coating of approximately 5.5 mm acrylic resin mortar containing quartz sand of unknown quantity.

F. Coating of approximately 8 mm modified metacrylic resin mortar containing approximately 70 per cent of quartz sand by volume.

G. Coating of approximately 1 to 1.7 mm epoxy resin mortar containing approximately 55 per cent of quartz sand by volume. The surface is coated with a thin layer of pure epoxy resin.

H. Coating of approximately 8.5 mm polyester resin mortar containing approximately 50 and 13 per cent by volume of sand and expanded polystyrene beads respectively.

I. Coating of 0.3 to 0.5 mm polyurethane.

J. Coating of approximately 0.5 mm water dispersed co-polymer of unknown composition.

K. Three-fold treatment of the concrete surface with silane.

L. 3 to 5 mm topping of latex-modified cement paste reinforced with a fabric of alkali resistant glass fibres.

M. Same as L but reinforced with a fabric of polypropylene fibres.

Every repairing material was used with two types of concrete, which were supposed to represent the limits of the range in qualities of the actual balcony slabs. The characteristics of the two concretes were:

K 150: cement, 190 kg/m^3; water-cement ratio, 1.0; without air-entrainment.

K 250: cement, 250 kg/m^3; water-cement ratio, 0.8; without air-entrainment.

A large number of specimens (100 mm x 100 mm x 400 mm) of each concrete type were cast, some with central reinforcement bars. After curing in water and air for some months, the beams were repaired with the 13 different materials. The beams are supposed to simulate the outer corner of a balcony which means that the top surface and two adjoining sides were repaired while the bottom surface was untreated. The two unrepaired sides were sealed with a membrane of epoxy resin and aluminium foil. The 'repairs' were made exactly according to the specifications given by the manufacturers. All concretes to be repaired were ground and the surface was normally treated with a primer before the application of a plastic coating. The beams were then subjected to the different durability tests.

It must be strongly emphasized that there are great difficulties in making durability tests where organic repairing materials such as plastics are involved; it is difficult to be sure of the exact chemical and physical composition and properties of the plastic, and it is impossible to simulate exactly all environmental influences on the material which make the behaviour and properties of an 'aged' plastic much different from those of a 'virgin' material. It was, therefore, decided to concentrate on repairing systems rather than repairing materials. The fundamental variables in the test were, therefore, the permeability of the material and its thickness, but not so much its chemical composition. The effect of ageing was investigated by making artificial cracks through the coatings before exposing them to the freeze/thaw action.

Results

The experimental work has not yet been completed and, therefore, only a part of the results can be presented; a full report will be published at a later date.

The Sensitivity towards Temperature Gradients

Three specimens of each type of repair were subjected to 96 cycles of unidirectional heating and cooling. The temperature was measured by thermocouples at different depths from the heated surface. The apparatus and a typical time-temperature curve is shown in Figure 5.

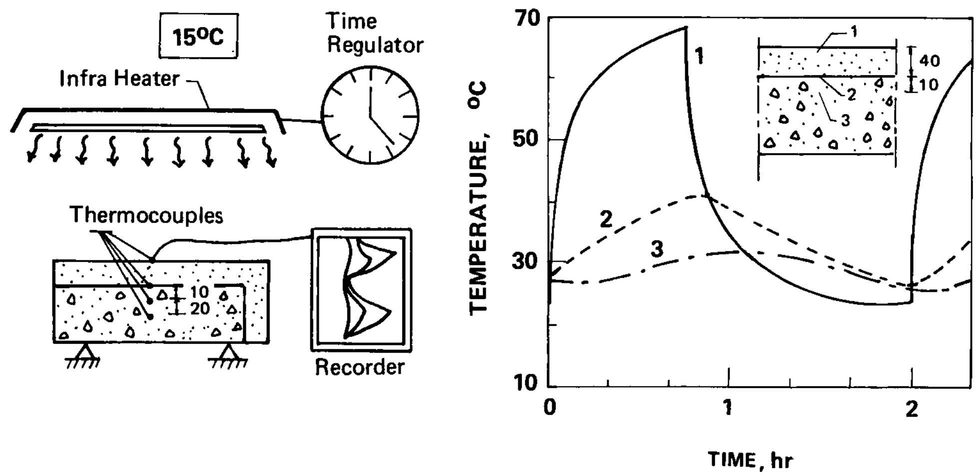

Figure 5 Test of the Sensitivity to Temperature Gradients
(a) Test Principles and (b) Time-Temperature Curves for a Repair of type A

In order to detect eventual damage caused by temperature stresses, the bond strength perpendicular to the repaired surface and the fundamental frequency of transverse vibration were determined afterwards and compared with values for the virgin material; some results are shown in Figure 6. Almost no destruction could be detected for any specimen and it can therefore be concluded that damage caused by thermal stress only is very unlikely in practice, especially since the temperature cycles used in the tests were at least 2 times as rapid as those to be expected in practice.

The Frost Resistance of Uncracked Specimens

The temperature gradient appearing in a porous moist specimen that is gradually cooled from the surface will cause a moisture diffusion outwards. Thus, if the surface of the specimen is covered with a thin impermeable coating this can result in an accumulation of moisture behind the surface layer during the cooling period. On the other hand, when the surface is coated with a porous material or uncoated, the moisture content of the surface layer of the sub-base material could be almost unchanged or even reduced. With a thick impermeable coating the temperature gradients in the sub-base material could be so small that no damage will occur.

In order to investigate the effect of eventual moisture redistribution, three specimens of all repair types were exposed to sealed freeze/thaw tests after being stored in water for exactly the same period. Every specimen was freeze/thaw tested three times. Before each test it was water stored for a fixed time; 32 days before the first test, another 14 days before the second test and another 75 days

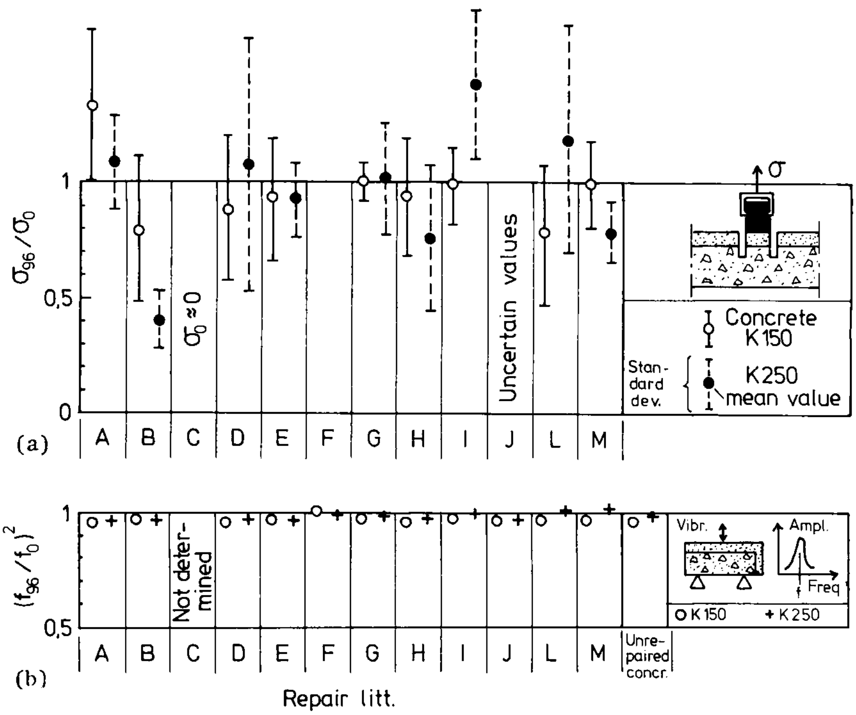

Figure 6 Results of the Temperature Gradient Test
(a) Relative Bond Strength where σ_{96} is the Strength after 96 cycles (the σ_0 Values are shown in Figure 7a; the standard deviation s is calculated from $s = [v(96)/m(96)^2 + v(0).m(96)^2/m(0)^4]^{\frac{1}{2}}$ where v(x) is the variance and m(x) the mean value for all determinations after x cycles
(b) Relative Fundamental Frequency of Transverse Vibration

before the last test. Hence, the water content was increased stepwise between every freeze/thaw test. The test consisted of 5 freeze/thaw cycles during which the specimens were stored in thick plastic bags, which effectively eliminated moisture exchange with the surroundings.

The natural frequency of transverse vibration was determined before and after each freeze/thaw test and the results are shown in Figure 7(c). The bond strength of the upper surface was determined after the final test and compared with the value for unfrozen specimens, Figure 7(b). It should be noted that the frequency values for most of the plastic-coated specimens are lower than for the uncoated concretes, especially for the high grade concrete K 250. This indicates that an impermeable coating might have a reducing effect on the frost resistance of the substrate. The frequencies for the concretes coated with normal cement mortars (types A and B) are much higher than for the plastic coated concretes. On the other hand, these frequencies are a bit difficult to interpret since it is possible that an overall high frequency can be obtained even with a damaged sub-base concrete due to a high frequency of the thick, highly frost resistant repair itself. Two vertical surfaces were sealed on every specimen, but the eventual damage caused by them will, however, have little influence on the measured frequency.

The bond strength values are reduced for all materials, in some cases to very low values, while in other cases to about 50 per cent of the original value indicating

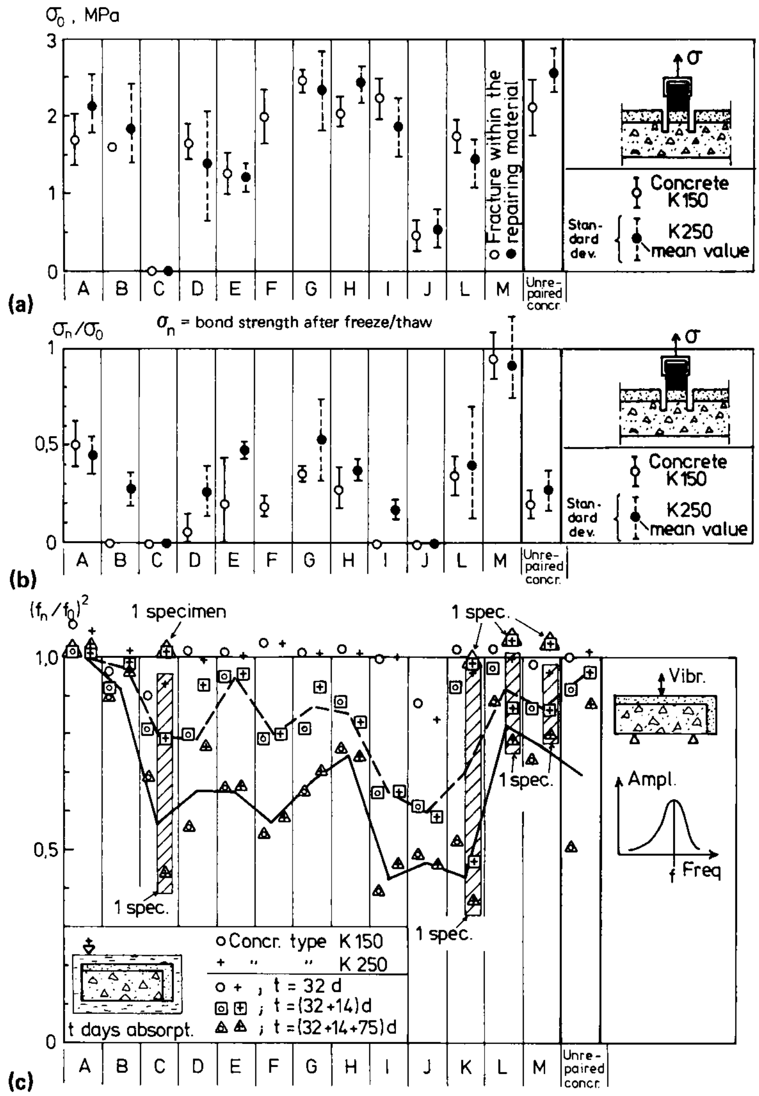

Figure 7 Results of the Freeze/Thaw Test with Uncracked Specimens, (a) Bond Strength of Unfrozen Specimens, (b) Relative Bond Strength after Freeze/Thaw and (c) Relative Fundamental Frequency of Transverse Vibration

a certain destruction even in the surface of mortar-coated and uncoated concretes although this was not indicated in the frequency measurements.

It is proposed that a sonic test will be carried out on all specimens in order to obtain a better picture of the internal destruction.

The Frost Resistance of Pre-Cracked Concretes

A repair in practice will probably have some cracks due to ageing effects in the repairing material itself or due to differential movements between the balcony slab and the repair. For practical reasons it was not possible to use such aged specimens in the tests. Therefore, an artificial ageing was produced forming one crack right through the coating in three specimens of each repair type. The beams were reinforced with a central bar and the crack was made by bending the specimen. The crack was, therefore, widest at the surface where it was about 0.5 mm.

The specimens were placed unsealed with the cracked side upwards in an automatically controlled climate chamber, consisting of a cooling coil for freezing, infrared heaters for thawing and water-hoses for moistening the specimens. The apparatus and a typical freeze/thaw cycle is shown in Figure 8. It should be noted that the specimens are allowed to dry out partly during the freezing period.

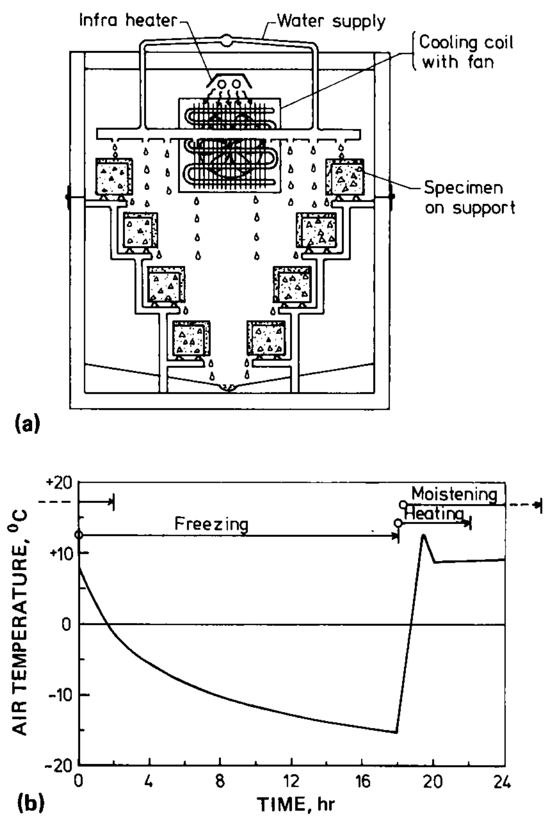

Figure 8 The Freeze/Thaw Test with Pre-Cracked Specimens
(a) The Apparatus, (b) A Typical Freeze/Thaw Cycle

The environment in the chamber was supposed to simulate an actual situation of water flowing along the aged balcony repair just before the temperature drops below zero. Thus, the water will be sucked into the concrete through the crack and spread beneath the coating, making complete drying out difficult if the coating is impervious. Water will also flow around the vertical sides and be sucked into the concrete from its bottom side.

The test is still in progress and no quantitative results can, therefore, be given. However, the following features have been observed: spalling of some thin plastic coatings at the crack and at the specimen edges and corners, delamination of the polypropylene fibre reinforced cement paste (type M) and severe damage of the latex-modified mortar (type C). The two ordinary mortars appear to be in excellent condition after more than 200 cycles. The bond strength will be determined and a sonic test will be made at the end of the test.

The Moisture Diffusivity

The moisture diffusivity of all repairing materials is to be determined by the well known wet-cup method using a relative humidity of 50 per cent above the specimen and 75, 84 and 100 per cent in the cup.

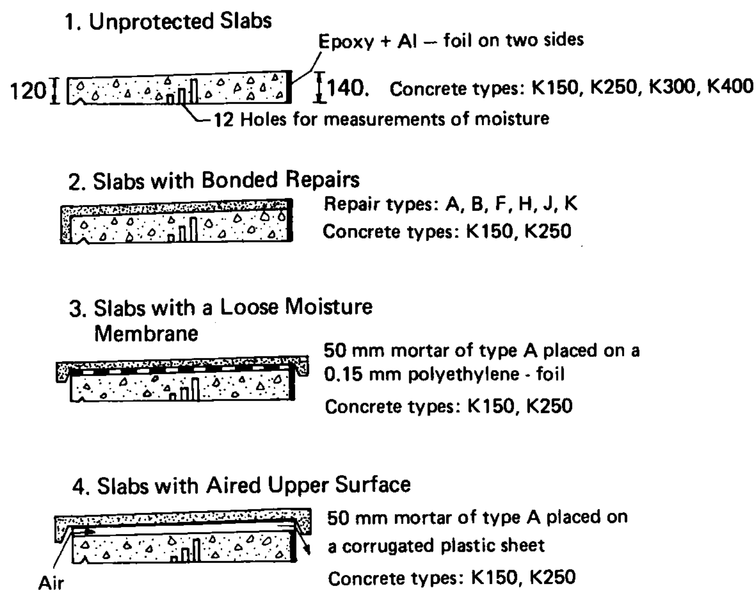

Figure 9 Different Repairing Systems Used in the Full Size Field Test

FIELD TESTS

The laboratory tests have been supplemented with a field test in which 20 full size concrete slabs were placed outdoors in a horizontal position about 0.5 m above the ground and without protection against rain, sun or frost. The slabs have a mean thickness of 130 mm and an area of 1.1 m^2. Sixteen of the slabs have been 'repaired' with different repairing systems; some of them exhibit bond to the slab, for two slabs a frost resistant cement mortar is separated from the slab by a polyethylene foil, for two slabs the frost resistant mortar is placed on a corrugated plastic sheet placed on the slab making it possible for the slab to dry upwards at the same time as it is protected against capillary water uptake. Two perpendicular sides of the slab are moisture insulated. The different repairing systems employed are shown in Figure 9.

Every slab is provided with 12 holes in different places and to different depths for the measurement of moisture profiles. The behaviour and the moisture content of the slabs is to be studied over a period of years, but measurements made to date indicate a rather high moisture content in all slabs; in many cases the relative humidity is 100 per cent indicating a high risk of frost damage. Only one slab, of type (d) in Figure 9, had a mean relative humidity lower than 90 per cent during the 4 month period studied so far.

CONCLUSIONS

Practical experience shows that the durability of different repairing systems used for the restoration of damaged concrete balconies is rather poor. The laboratory tests described tend to confirm this general conclusion. When the original concrete is poor, having high water-cement ratio and no air-entrainment, no repairing system was found to be able to prevent the concrete from being damaged by frost when subjected to the rather high moisture loads used in the tests. This is indicated by the bond tests made after freezing and thawing. In-situ measurements of the moisture content in real balconies and in full size test balconies indicate that high moisture contents can appear even in balconies with 'impervious' coatings.

The repair of type A (a highly frost resistant cement mortar placed directly on the concrete slab) was found to be the best of the repairing systems studied here. A well-designed repair of the type shown in Figure 9(d) should also behave well in cases where the concrete cover is not carbonated too deeply. If the cover could be designed in such a way that the risk of capillary water uptake in the slab is small it ought to stay quite dry, making the risk of further damage small.

ACKNOWLEDGEMENT The authors would like to thank Mr. Conny Rolén, who undertook a large portion of the experimental work, and the Swedish Council for Building Research which sponsored the project.

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FATIGUE RESISTANCE OF COMPOSITE PRECAST AND IN SITU CONCRETE FLOORS

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ABSTRACT The fatigue strength of composite in-situ and precast concrete slabs has been studied following the failure of a suspended slab at a local factory in the Midlands. Nine short (600 mm long) lengths cut from standard precast prestressed units (400 mm wide) have been placed side by side and used to cast transverse specimens 600 mm wide tested on a simple span of 3.18 m (i.e. normal to the main span direction of the standard floors).

The standard floor has been analysed using finite element and other methods to determine the predicted deflections and hence the central displacements which should be applied to the transverse specimens to reproduce the actual conditions in practice. The parameters which were studied included the thickness of the in-situ concrete topping (76.2 mm and 101.6 mm), bar size, bar spacing and type of distribution reinforcement.

Displacements were checked by LVDT linear transducers and by dial gauges and strains by demec and electrical resistance strain gauges. Static tests were carried out periodically to determine accurately the strain distributions and crack widths at various stages. Progressive damage under fatigue was indicated by the reduction in the slopes of the load-deflection curves for the transverse units and hence their decreasing ability to distribute the applied concentrated load to much more than one precast unit.

The S-N-P characteristics of the reinforcing bars have been determined from fatigue tests on individual bar specimens. Four transverse slab specimens have been tested and conclusions drawn concerning required depth of in-situ concrete and amounts of distribution reinforcement where fatigue loading can occur.

INTRODUCTION

Fatigue is the process of progressive, permanent structural change occurring in a material which is subjected to time fluctuating stresses and strains, culminating in cracks or complete fracture after a sufficient number of fluctuations. Interest in fatigue strength has increased in recent years for at least two reasons (1). First, ultimate strength design and the use of high strength materials at repeated high stress levels. Second, repeated loading may produce effects such as cracking between component materials or inclined cracking in prestressed beams, which alters the static load carrying characteristics.

Extensive research has been carried out by many investigators of plain, reinforced and prestressed concrete (e.g. 2-11) and some conclusions in Nordby's review (2) were as follows:

(a) The fatigue strength of plain concrete decreased with increasing number of cycles, at least up to 10 x 10^6 cycles, and therefore did not appear to possess a distinct fatigue limit as such.

(b) The fatigue limit (as given by 10 x 10^6 cycles) of plain concrete subjected to repeated flexural load was about 55 per cent of the static ultimate flexural strength, although actual variance was from 33 to 64 per cent, depending on other variables such as age, moisture content, curing and aggregate.

(c) Most failures of reinforced beams were due to failure of the reinforcing steel. The failures seemed to be connected with severe cracking and the possible stress concentration and/or abrasion connected with these cracks. Beams critical in longitudinal reinforcement seemed to have a fatigue limit of 60 to 70 per cent of static ultimate strength for 10^6 cycles.

Concerning the tensile regions of cracked reinforced concrete beams, Bresler and Bertero (3) concluded that with repeated load the average surface crack width approached the elongation of unbonded steel reinforcement over 8 inches (200 mm), and the relative contribution of the concrete to the stiffness of the tensile zone became negligible as the number of load cycles and the magnitude of peak stresses increased.

In 1974 Hawkins (11) tested at $4\frac{1}{6}$ Hz (250 cycles per minute) nine one-way, centrally loaded slabs reinforced with two layers of a square, smooth wire fabric consisting of No 2 wires at 150 mm (6 in) centres, inducing stress ranges in the wires varying between 0.30 and 0.46 times the tensile strength of the fabric. He concluded that the fatigue characteristics of the slabs were controlled by the fatigue characteristics of the welded wire fabric. The fatigue life value for fracture of the first wire in a slab could be determined with reasonable accuracy using fatigue characteristics measured from individual wire tests and a deterministic procedure to assess the appropriate level of probability. Conservative values for a fatigue life value for collapse could be determined based on first wire fracture and Miner's theory to predict cumulative damage effects. For a smooth wire fabric subjected to repeated loading the increase in deflection and crack widths with cycling was slow prior to first wire fracture even though there was extensive loss in bond, so that anchorage of the longitudinal wires was primarily provided by the transverse wires.

The present investigation aimed to determine satisfactory combinations of depths of in-situ concrete topping and amounts of distribution reinforcement in composite precast and in-situ concrete slabs subjected to repeated loading, the decreasing ability of individual precast units to distribute the applied concentrated load to adjacent units with progressive fatigue cracking being of especial interest.

EXPERIMENTAL PROGRAMME

Nine short (600 mm long) lengths cut from standard precast prestressed units

(400 mm wide), supplied by Bison Concrete Ltd., were placed side by side and used to cast each transverse specimen 600 mm wide, see Figure 1. These specimens were then inverted and tested on a simple span of 3.18 m which was normal to the main span direction of the standard floor as shown in Figure 2.

The concrete mix (12) for the in-situ concrete was 4.6 : 3.62 : 1 : 0.44 (10 mm Midland gravel : 5 mm down sand : OPC : water) by mass. The design concrete cube strength was 27.5 N/mm^2 at 28 days. A Losenhausen EHR1 universal testing machine was used for the fatigue tests.

The standard floor was analysed using a finite element method (13), Figure 3, to determine the predicted central deflection under a single concentrated load of 31.6 kN* and the corresponding central displacements which needed to be applied to the transverse specimens. Reductions in lateral stiffness (due to cracking between the precast units) as the number of cycles increased was allowed for in the computer programme by reducing the thickness of in-situ topping and simultaneously increasing the depth of off-set precast unit in order to maintain the same longitudinal stiffness. When the indicated load for the specimen subjected to the calculated central deflection for zero reduction in lateral stiffness (δ_0) reduced by a predetermined amount, a static test was carried out to determine the complete load displacement characteristics up to the new calculated central deflection. The amplitude was then increased according to the finite element analysis calculations.

All fatigue tests on slabs started by applying the required initial load, P_0, for the calculated central displacement for zero reduction in lateral stiffness, δ_0. The peak displacement, δ_0, was maintained constant either for a given number of cycles as shown in Figure 4 or until the indicated load reduced by a predetermined amount given by the initial static test and a given reduction (e.g. 10 per cent) in lateral stiffness. A further static test was then carried out to determine the new load-displacement characteristics up to the calculated central displacement for the given reduction in lateral stiffness (e.g. δ_{10} for 10 per cent reduction). The specimen was then subjected to the given displacement (e.g. δ_{10}) either for a further predetermined number of cycles or until the indicated load reduced by a predetermined amount corresponding to the next increment (e.g. δ_{20}) and so on. The rate of loading was 1 Hz. An external safety switch was designed to cut out the machine if the set displacement was exceeded. The main parameters for the transverse specimens involved in the test programme are shown in Table 1.

Table 1 Transverse Specimens

SPECIMEN	IN-SITU TOPPING		PLAIN ROUND MS BARS	
	Thickness mm	28 Day Cube Strength N/mm^2	Number	Diameter mm
1	76.2	31	1	6
2	101.6	31	1	6
3	76.2	34	2	6
4	76.2	28.4	1	6

* Total weight of loaded fork-lift truck used on factory floor which failed.

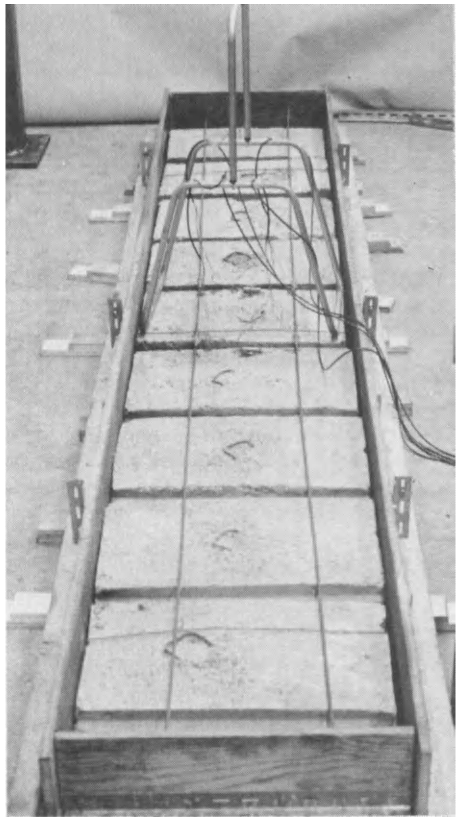

Figure 1 A Typical Transverse Specimen Before Casting In-situ Concrete

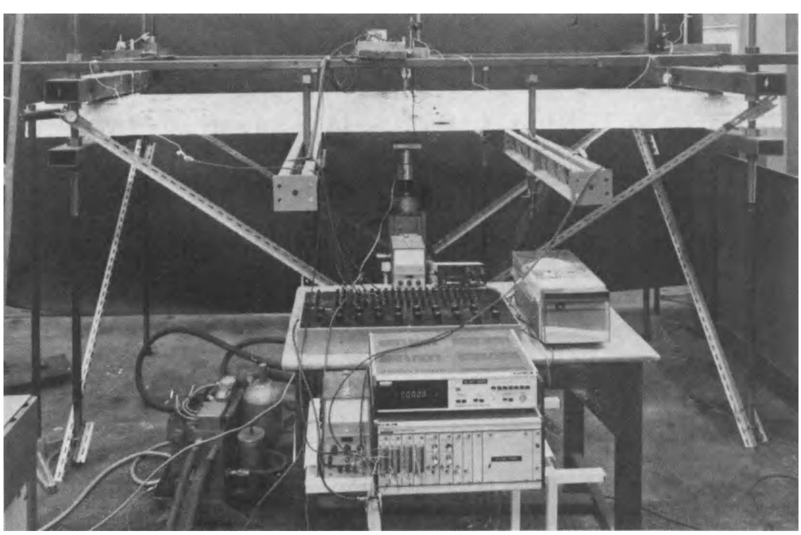

Figure 2 A Transverse Specimen in the Fatigue Test Rig

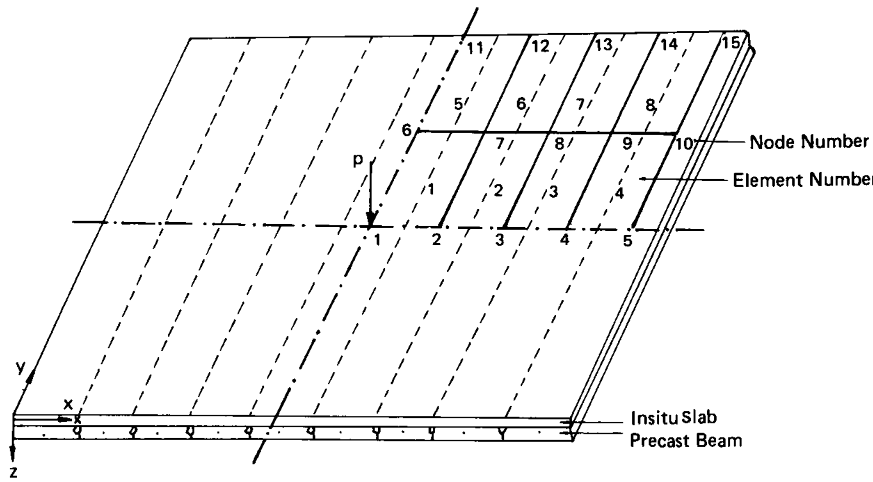

Figure 3 Finite Element Layout for Floor Analysis

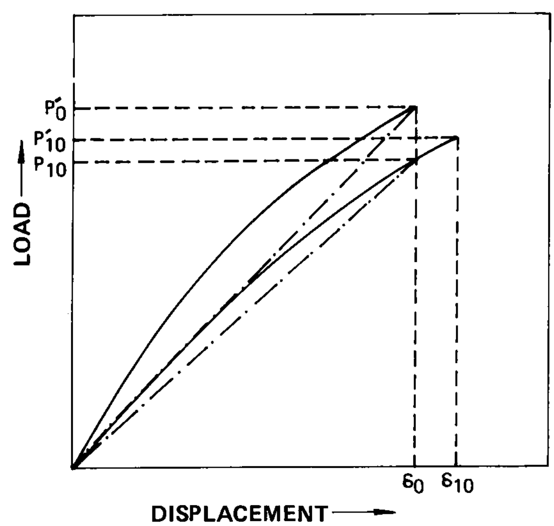

Figure 4 Displacement Control Test

EXPERIMENTAL RESULTS

Figure 5(a) was plotted from the results of static tests on three 6 mm diameter round MS (R6) specimens which gave an average failure load of 14.6 kN. Three fatigue tests were carried out at $4\frac{1}{6}$ Hz (250 cpm) for each load range and the resulting S-N curve plotted in Figure 5(b). The load-deflection and load-strain curves are shown in Figures 6, 7 and 8. Progressive damage under fatigue was indicated by the reduction in the slopes of the load deflection curves. The convex upward curve for specimen 1 gradually straightened under repeated load and finally became concave upward near failure, the degree of concavity depicting approaching failure as concluded by Van Ornum (15). Figure 9 shows the effect of a rest period on the strain measured at the top fibre of specimen 1, due to concrete creep recovery accentuated by the self weight of the specimen.

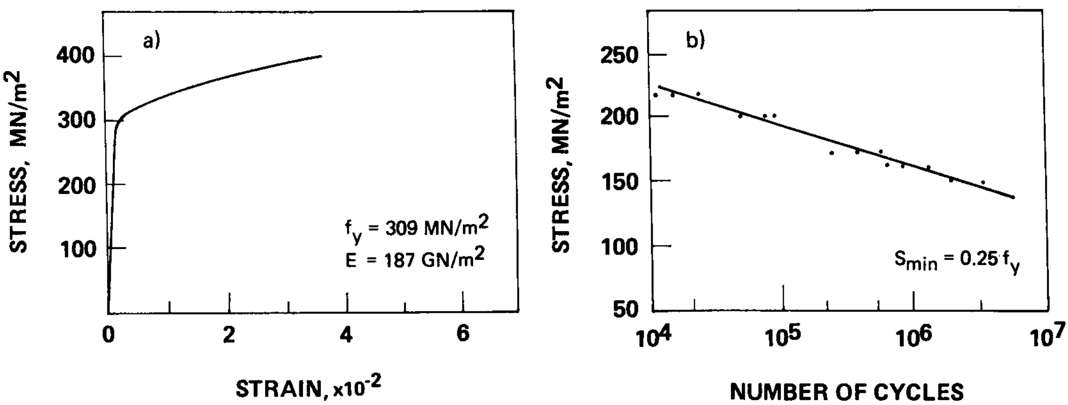

Figure 5 Characteristics of 6 mm Round MS Bars

The strain distributions across the specimens shown in Figure 10 indicated discontinuities at the interface due to relative movement (slipping) between the precast unit and the in-situ topping. As the number of cycles increased the distribution became more linear across the depth. Figure 11 shows the S-N relations of specimen 1 determined from the displacement control fatigue test. When the amplitude was 1.8 mm the predicted failure was at 10 x 10^6 cycles but increasing the amplitude to 4 mm gave a predicted failure at about 2 x 10^6 cycles. The first crack for specimen 1 was detected at 10^3 cycles at a joint adjacent to the loading area. The crack width increased gradually from 0.005 mm up to 0.02 mm at about 200 x 10^3 cycles for a constant amplitude of 1.8 mm. A static test was carried out at 350 x 10^3 cycles and the amplitude was increased to 4 mm. Figure 12 shows that the crack width then increased rapidly up to 0.65 mm at just over 10^6 cycles. The rapid increase was due to slipping and bond failure of the reinforcement. Figure 13 shows the crack at 500 x 10^3 cycles. Failure for specimen 2 was again expected at the joints of the precast units adjacent to the loading area, but the first crack occurred at the centre of the central precast unit, presumably because the extra thickness of mortar which escaped over the joint, as shown in Figure 13(b), made the transverse strength and stiffness at least equal to that elsewhere. There appeared to be very fine cracks at the joints which did not propagate. A static test was carried out each time there was a significant estimated (about 10 per cent) reduction in the stiffness of the slab and the applied constant displacement was increased accordingly. The crack width was measured at each of the nine static tests. Strain distributions were again determined across the depth of the specimen when first the top electrical resistance strain gauges and then the side gauges failed as the crack propagated completely through the section.

For specimen 3, electrical resistance strain gauges were fixed onto the reinforcement about 10 mm from the joint where cracking was expected and did in fact occur after a few thousand cycles. At 70 x 10^3 cycles, when the lateral stiffness had

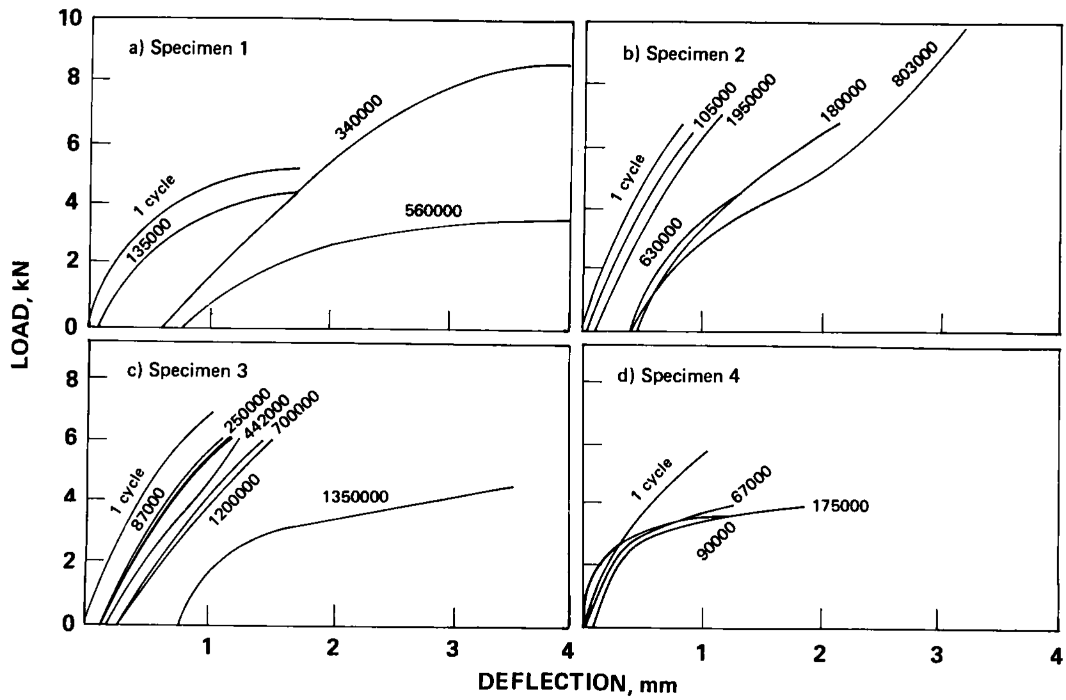

Figure 6 Load - Central Deflection Curves

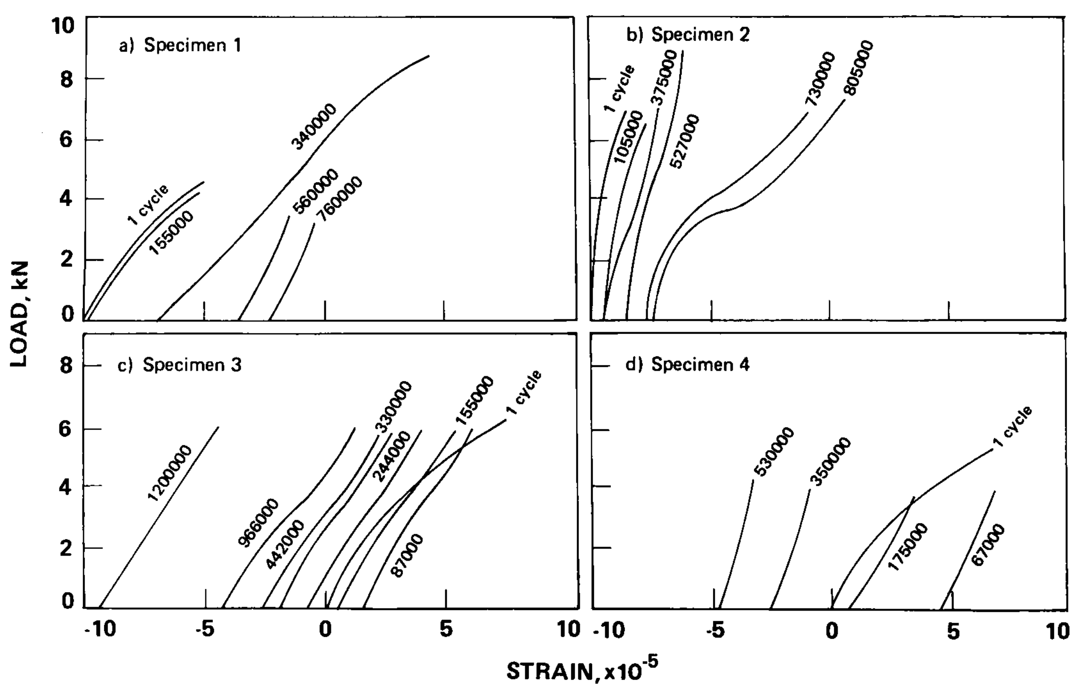

Figure 7 Load - Strain Curves (top fibre of in-situ concrete)

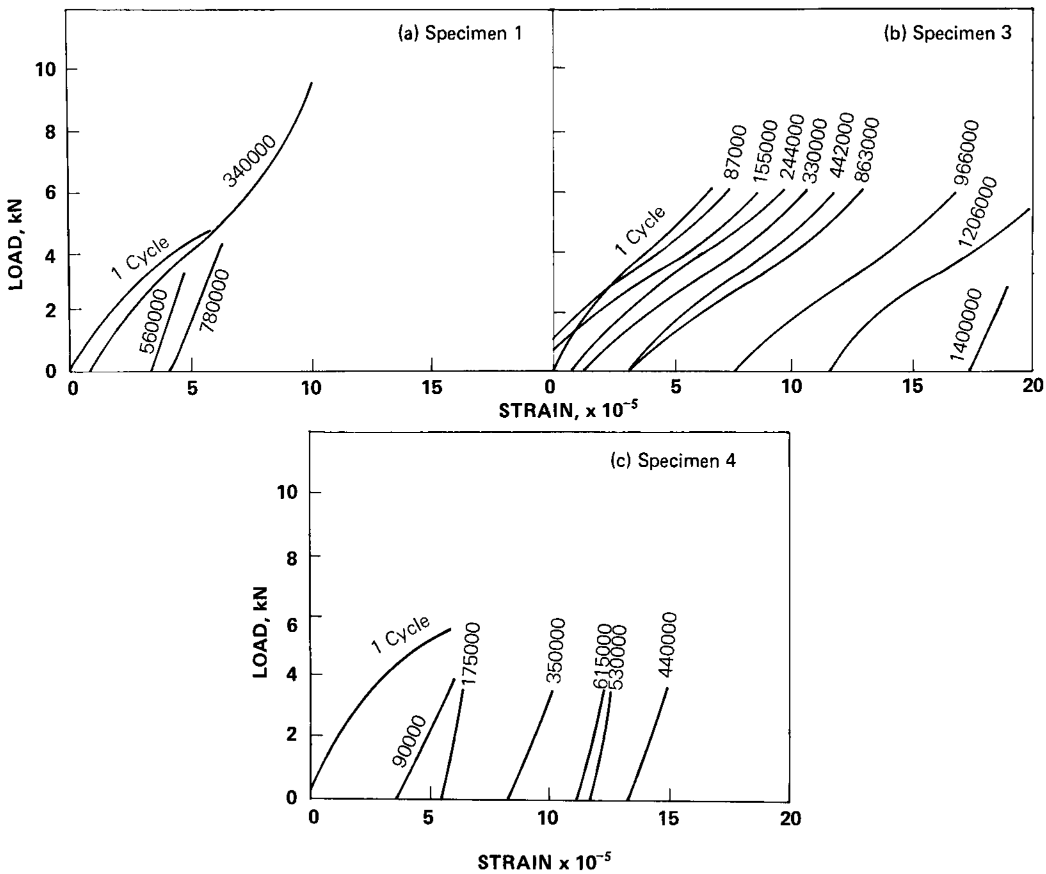

Figure 8 Load - Strain Curves (bottom fibre of precast unit)

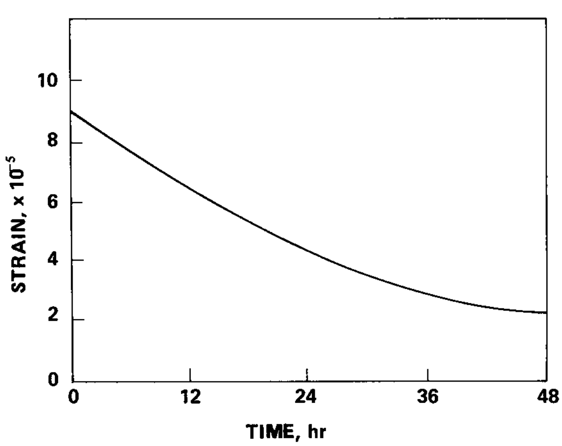

Figure 9 Rest Period

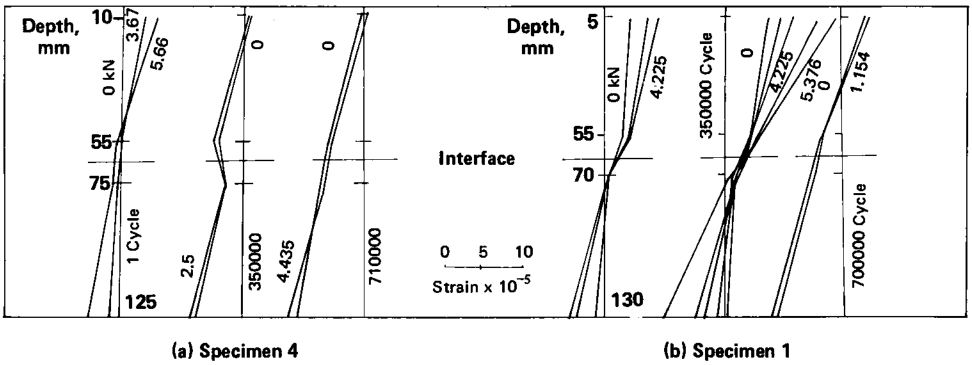

Figure 10 Strain Distribution Across the Depth

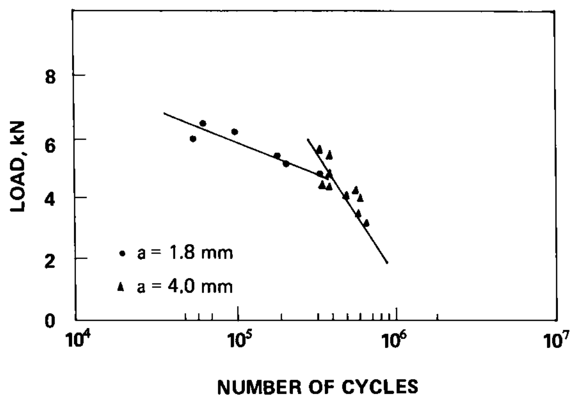

Figure 11 L - N Curves of Specimen 1 under Displacement Control

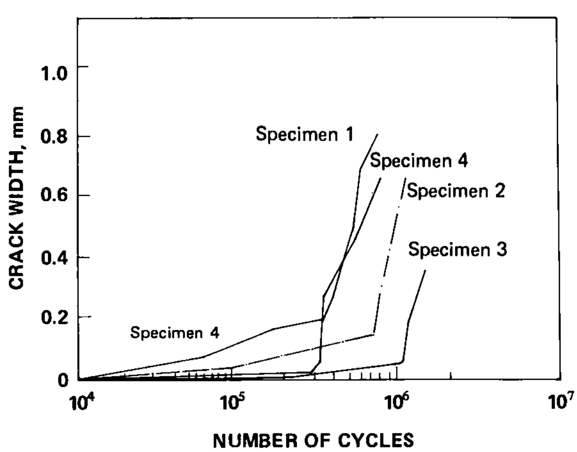

Figure 12 Crack Width - Number of Cycles

(a) In-situ Concrete Face

(b) Precast Unit Face

Figure 13 Joint of Specimen 1 after 500 x 10^3 Cycles

Table 2 Displacements Applied to Specimen 4

PROPOSED REDUCTION IN STIFFNESS AT CHANGE %	ACTUAL REDUCTION IN LATERAL STIFFNESS AT CHANGE %	NUMBER OF CYCLES AT CHANGE x 10^3	CENTRAL DISPLACEMENT mm
0	0	0	1.09
40	37.8	67	1.22
50	44.5	90	1.31
60	54.5	175	1.50
70	62	350	1.85
80	75	440	2.32
90	80	576	2.76
100	83	615	3.18

reduced by about 10 per cent, a static test was carried out and the amplitude was increased, according to the finite element analysis, to 1.12 mm. At 87 x 10^3 cycles the maximum crack width was 0.01 mm. Figure 6 indicated a gradual decrease up to about a 60 per cent reduction in lateral stiffness, followed by a rapid decrease with subsequent cycles. The strain readings on the steel bar increased gradually up to 863 x 10^3 cycles and then increased dramatically indicating that extensive bond slipping adjacent to the crack had progressed to the gauge. The crack width continued to increase steadily from 0.045 mm at this stage but at just over 10^6 cycles it increased rapidly to about 0.36 mm at 1.5 x 10^6 cycles. The load-strain curve was initially (Figures 7 and 8) parabolic gradually becoming concave and then S-shaped due to hinge formation at the joint.

Specimen 4 was similar in construction and behaviour to specimen 1, the main difference being that more frequent increases in the applied constant amplitude of displacement in accordance with the finite element analysis were used for specimen 4 as shown in Table 2.

CONCLUSIONS

The fatigue strength of composite in-situ and precast concrete slabs has been studied following the failure of a suspended floor. Four transverse specimens have been tested and conclusions have been drawn as follows.

Under repetitive loading the modulus of elasticity changes in various ways depending on the intensity of load. Progressive damage under fatigue loading was indicated by the reduction in the slopes of the load-deflection curves for the transverse unit.

Rest periods occurred for the first specimen due to accidental cutting out of the machine. Due to self weight the recovery in strain of the top fibre was as high as 45 per cent during the rest period of three days. These rest periods could therefore have increased the endurance of the concrete in this specimen.

The load-strain curve for top and bottom fibres of the specimen varied with the number of repetitions. The slope of the curve may decrease at the lower end and increase slightly at the upper end to become concave upward.

The width of crack increased suddenly if the number of cycles became sufficient to produce extensive local bond failure of the reinforcement.

Increasing the thickness of the in-situ concrete in specimen 2, combined with mortar leakage from the concrete which produced extra thickness over the joint, increased the joint strength and stiffness sufficiently for failure to occur in the middle of the precast unit at midspan of the transverse specimen.

The strain distributions were studied across the depth of the transverse specimens. Discontinuities at the interface appeared to be due to relative movement of the precast unit and in-situ concrete. The distribution became more linear across the depth with increasing number of cycles.

The crack width at the joint increased gradually for specimens 3 and 4 up to 1200 x 10^3 and 350 x 10^3cycles respectively, before showing a sudden increase and exceeding 0.2 mm. Assuming that for fatigue design crack widths should not exceed 0.2 mm after 10^6 cycles, then specimen 3 (with twice the usual recommended amount of distribution reinforcement for static loading conditions) would be satisfactory, but clearly more tests will be necessary to verify this result.

ACKNOWLEDGEMENTS The authors wish to thank Bison Concrete (Midlands) Ltd., for supplying all the precast floor units, and especially Mr. E.T. Hall for his help and interest throughout the investigation.

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STRUCTURAL REPAIRING OF THE ROOF-PLATE OF THE CHURCH L'EPIPHANIE AT SCHAARBEEK, BELGIUM

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ABSTRACT The paper describes the repair of a large, rectangular, reinforced concrete roof-plate which on removal of the formwork suffered an abnormally large deflection. The repair of the roof-plate was carried out after in-situ measurements and test borings and simulation on a model plate in micro-concrete. The behaviour of the repaired plate was measured and compared with theoretical predictions, the agreement between both being very good. This successful repair saved a time consuming and expensive demolition and rebuilding of the roof-plate.

INTRODUCTION

The load bearing element of the roof of the new church 'l'Epiphanie' at Schaarbeek is a rectangular reinforced concrete plate 500 mm thick with overall dimensions of 19525 mm x 25890 mm, measured axis to axis of the supporting columns, see Figure 1. The plate is lightened with cardboard tubes 350 mm in diameter at a mutual distance of 100 mm. In the short direction the reinforcement consists of 25 mm diameter bars every 200 mm ($A_1 + A_1' = 2454$ mm^2/m) and in the long direction of 16 mm diameter bars every 125 mm ($A_2 = 1608$ mm^2/m). In the corners, upper reinforcement of 25 mm diameter bars is provided every 200 mm in both directions. The steel quality is BE40, with permissible stress $\bar{\sigma}_a = 240$ N/mm^2. The concrete quality required is 35 N/mm^2 after 28 days, measured on 200 mm cubes, and the plate was designed for a total load of 9.65 kN/m^2. At the removal of the formwork an abnormally large deflection was noticed. In Figure 2 the deflections of the plate measured after the removal of the formwork are shown. The maximum deflection amounted to almost 190 mm, whereas only 50 mm was anticipated.

DIAGNOSIS OF THE REASONS FOR FAILURE OF THE PLATE

Possible causes for the abnormal deflections of the plate were identified as i) the quality of the concrete, ii) the homogeneity of the concrete, and iii) the positioning of the tubes in the plate. In order to gain an initial insight into what went wrong with this concrete plate, a number of cores were taken. The results of compression tests on these cores are indicated in Table 1.

The concrete quality is, on average, 12 per cent too low, while for core f the strength is impermissibly low. If the deflections were recalculated according to the average concrete quality, the sagging calculated for a concrete quality of

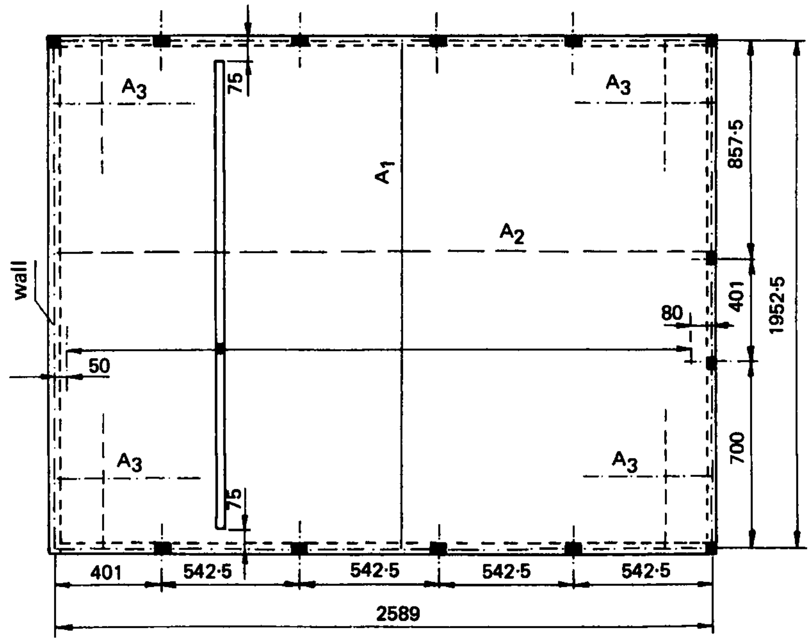


Figure 1 Disposition of Reinforcement and Tubing in the Plate

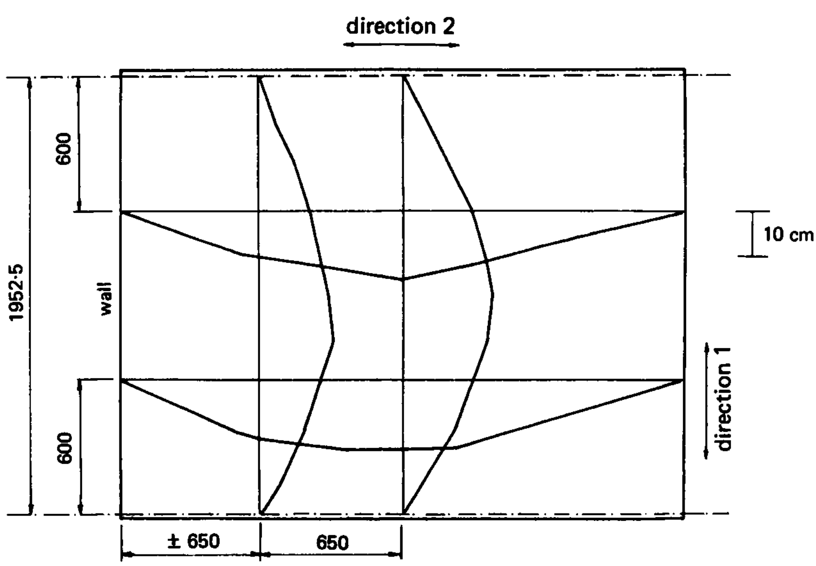


Figure 2 Deflections Measured in Relation to the Original State

35 N/mm^2 would increase by a factor such that:

$$\frac{(E_b') \text{ required}}{(E_b') \text{ real}} = \frac{5200\sqrt{35.0}}{5200\sqrt{28.9}} = 1.1$$

Disregarding the result for core f, an average strength of 31 N/mm^2 is obtained; the multiplication factor would then equal 1.06 only.

Table 1 Ultimate Compressive Strength of the Concrete Cores

CORE	POSITION IN SLAB	COMPRESSIVE STRENGTH, $R'_{w,28}$ N/mm^2
a	upper side	34.0
b	lower side	28.3
c	upper side	32.8
d	central part	29.0
e	lower side	32.7
f	upper side	15.6
g	upper side	28.9

It also appears from the results that the homogeneity of the concrete is only moderate. When the concrete cores were being taken it was noticed that the concrete cover to the tubes varied, generally between 25 and 50 mm, and was practically non-existent in some places. During the repairing of the plate it became obvious that most of the tubes had lifted when the concrete was poured. This lifting of the tubes has a double and important effect: the diminution of the rigidities both in the uncracked and the cracked state not only increases the deflections, but also the stresses. These greater stresses in turn result once more in a greater deflection.

THEORETICAL DEFLECTIONS OF ORTHOTROPIC PLATES

With a view to obtaining the most accurate possible calculation of the deflections to be expected, the influence of the orthotropy of the plate was studied.

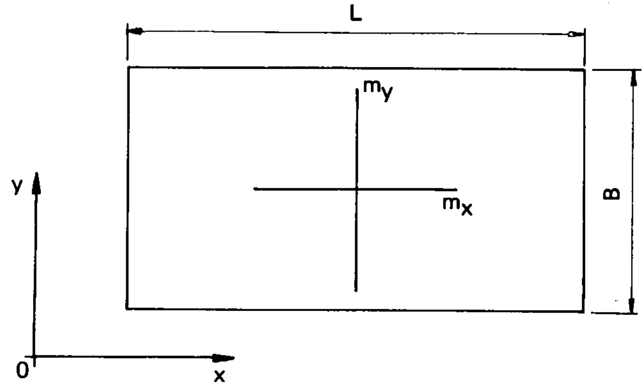


Figure 3 Diagram of a Plate

Assuming:

$$K_x = \frac{(EI)_x}{1-\mu_x\mu_y} = \text{unit bending stiffness in the x-direction}$$

$$K_y = \frac{(EI)_y}{1-\mu_x\mu_y} = \text{unit bending stiffness in the y-direction}$$

$$2C_x = GF_x = \text{unit torsional rigidity in the x-direction}$$

$$2C_y = GF_y = \text{unit torsional rigidity in the y-direction}$$

μ_x and μ_y = Poisson's ratio in x and y-directions,

it appears from the theory of orthotropic plates that the deflections are a function of two ratios of rigidity, viz.

$$\rho = \frac{K_x}{K_y} = \frac{EI_x}{EI_y}$$
$$\beta = \frac{C_x + C_y}{K_x} \quad \text{with } C_x = \frac{1}{4} E.F_x \text{ and } C_y = \frac{1}{4} E.F_y \tag{1}$$

In Figure 4 the influence of ρ and β is shown for a plate with a length-width proportion, L/B, of 0.75.

This diagram was drawn up by integrating the fundamental differential equation for orthotropic plates (1):

$$K_x \frac{\partial^4 w}{\partial x^4} + (2C_x + 2C_y + \mu_y K_x + \mu_x K_y) \frac{\partial^4 w}{\partial x^2 \, \partial y^2} + K_y \frac{\partial^4 w}{\partial y^4} = p(x,y) \tag{2}$$

The coefficients μ_x and μ_y are set to zero in the first approximation. From the deflection function follow directly the moment functions m_x and m_y, see Figure 5.

DEFLECTIONS OF THE ROOF PLATE

The rigidities of the plate can be calculated from the known formulas for hollow cross sections, see reference 2, Table XI. For the ideal plate (tubes with the same cover above and below) constructed according to Figure 1, one obtains values for stiffness and rigidity of:

$$\begin{aligned} K_x &= 8777 \times E \ cm^4/cm \\ K_y &= 6638 \times E \ cm^4/cm \\ 2C_x &= 6684 \times E \ cm^4/cm \\ 2C_y &= 6483 \times E \ cm^4/cm. \end{aligned} \tag{3}$$

In these calculations the round holes have been replaced by rectangular openings, which influences the results only slightly. For a plate in which the tubes have lifted (concrete cover above tubes equal to 25 mm), one finds:

$$\begin{aligned} K_x &= 7840 \times E \ cm^4/cm \\ K_y &= 3925 \times E \ cm^4/cm \\ 2C_x &= 3687 \times E \ cm^4/cm \\ 2C_y &= 4896 \times E \ cm^4/cm. \end{aligned} \tag{4}$$

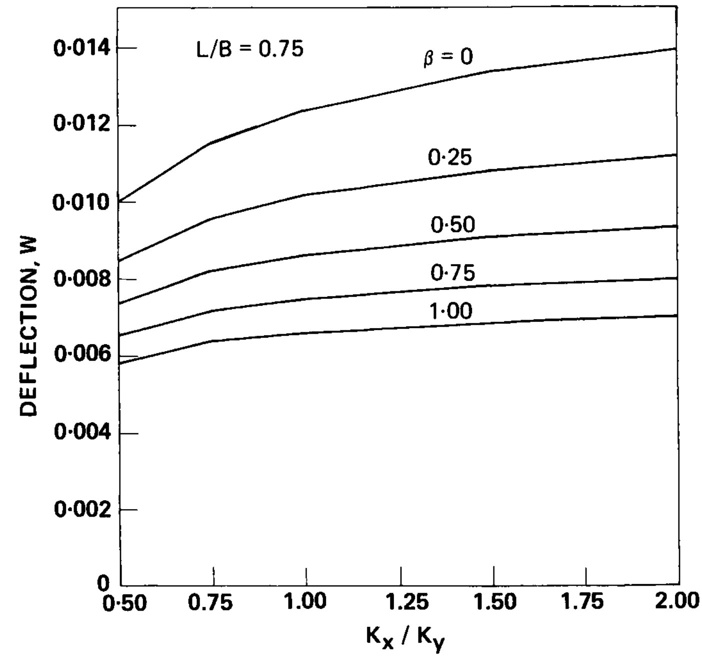


Figure 4 Deflections as a Function of ρ and β at L/B = 0.75 (w = reading x [PL^4/Kx])

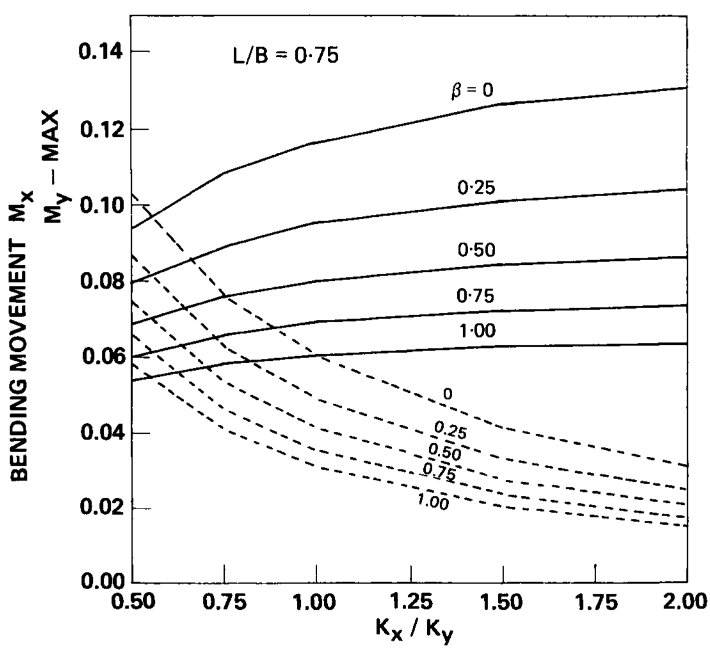


Figure 5 Bending Moments m_x and m_y maximum (m = reading x PL^2)

Although the torsional rigidities thus seem to become considerably smaller, the calculation of these torsional rigidities is based on such simplifications that an experimental check would be desirable. For this purpose two series of test pieces were prepared.

The first series consisted of two steel plates A1 and A2, of size 198 mm x 198 mm x 20 mm, see Figure 6. In one plate longitudinal holes 14 mm diameter were bored as shown in Figure 6. The ratio of the diameter of the holes to the thickness of the plate agrees with that of the real plate (14/20 = 35/50), as does the mutual distance of the holes (4/20 = 10/50).

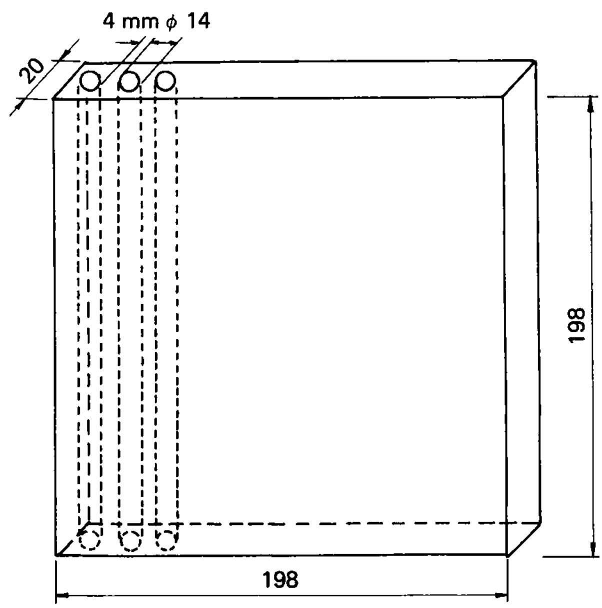


Figure 6 Steel Test Piece

A second series of test plates in polyester mortar made it possible to study the influence of the lifting of the tubes. The geometry of the test plates is given in Table 2, the scale factor to the real plate being 1/14.

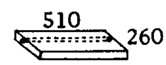
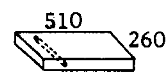
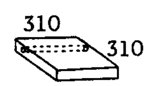
The model plates E1 and E2 are representative of the ideally executed plate (tubes not lifted). Model plates E3 and E4 simulate a roof-plate in which the cardboard tubes have a cover of 25 mm only.

The rigidities of the plate can be deduced (3) from the results of a torsion test and of a bending test in two mutually perpendicular directions. In Table 3 the experimentally determined rigidities are presented in comparison to those calculated for a full plate.

The agreement between calculated and experimental values is satisfactory for the ideally executed plate. When the cover to the tubes is small the differences increase, which may be due to the simplifications introduced into the calculations.

Henceforth, therefore, calculated quantities (t : theoretical) shall be distinguished from experimentally deduced values (e : experimental). With the use of the curves of Figure 4 and referring to the data given in Table 3, the deflections of the plate can be calculated.

Table 2 Test Plates of Polyester Mortar

TYPE	NUMBER	DIMENSIONS mm x mm x mm	HEIGHT AXIS/TUBE mm	CORRESPONDING CONCRETE COVER IN ROOF-PLATE mm	DISPOSITION OF TUBES
E1	1	36x260x510	18	75	
E2	1	36x260x510	18	75	
E3	1	36x260x510	15	25	I
E4	1	36x260x510	15	25	II
E5	1	36x310x310	18	75	
E6	2	36x310x310	15	25	III
E7	2	36x310x310	-	-	full plate
E8	1	36x260x510	-	-	full plate

With plates it is permissible to calculate the instantaneous deflections on the supposition that the section is not cracked (4, 5). For the ideally executed plate with L/B = 0.75, one obtains:

$$\rho_t = \frac{K_x}{K_y} = \frac{1}{0.756} = 1.32 \qquad \rho_e = \frac{1}{0.993} = 1.01$$

$$\beta_t = \frac{C_x + C_y}{K_x} = 0.749 \qquad \beta_e = 0.785$$

$$w_t = 0.0078\,\frac{pL^4}{K_x} \qquad w_e = 0.0074\,\frac{pL^4}{K_x}$$

Table 3 Calculated and Experimentally Determined Rigidity Ratios

RATIO	CALCULATED VALUES	EXPERIMENTAL VALUES	
		PLATES A1-2	PLATES E1-7
(a) Hollow plate with central tubes			
I_x/I-vol	0.843	0.765	0.843
I_y/I-vol	0.637	0.631	0.837
K_y/K_x	0.756	0.825	0.993
$\frac{2C_x+2C_y}{2K(1-\mu)}$	0.631	0.650	0.662
$\frac{2C_x+2C_y}{K_x}$	0.749	0.850	0.785
(b) Hollow plate with cover of 25 mm			
I_x/I-vol	0.753	-	0.799
I_y/I-vol	0.377	-	0.627
K_y/K_x	0.501	-	0.785
$\frac{2C_x+2C_y}{2K(1-\mu)}$	0.354	-	0.583
$\frac{2C_x+2C_y}{K_x}$	0.470	-	0.730

With the concrete quality required ($E_b = 5200\sqrt{35} = 30764$ N/mm^2), one obtains values for the deflection under its own weight as:

$$w_t = 36.3 \text{ mm} \qquad w_e = 34.4 \text{ mm}$$

With a quality of 31 N/mm^2 these deflections increase by 6 per cent. With the supposition that the concrete cover amounts to 25 mm everywhere and that the concrete quality averages 31 N/mm^2, the deflections become:

$$\rho_t = \frac{1}{0.501} = 2.00 \qquad \rho_e = \frac{1}{0.785} = 1.27$$

$$\beta_t = 0.47 \qquad \beta_e = 0.73$$

$$w_t = 0.0093\,\frac{pL^4}{E} = 48.5 \text{ mm} \qquad w_e = 0.0077\,\frac{pL^4}{E} = 37.8 \text{ mm}$$

These values are still much smaller than the measured deflection of 195 mm. The reason for this difference may be accounted for partly by the following factors:

i) the very irregular concrete cover and the quality of the concrete above the tubes; in some places the concrete cover was practically non-existent,

ii) the uplift of the reinforcement together with the tubes, in consequence of which the effective depth diminishes.

Similar factors, however, cannot reasonably be introduced into a calculation. For that reason it was decided to make a scale model of the roof-plate in micro-concrete. This model was intended to check the deflection of the ideally executed plate and at the same time to try out a possible repair method for the plate.

SCALE MODEL ANALYSIS

Scale Model of the Ideal Plate

The orthotropic roof-plate was executed to scale (scale factor 1/14) in micro-concrete. The mechanical characteristics of the micro-concrete were: compressive strength at 28 days, $R'_{w,28,}$ = 45 N/mm^2 (on 100 mm cubes); Young's modulus in compression, $E_{c,}$ = 29100 N/mm^2; Young's modulus in bending, $E_{b,}$ = 33800 N/mm^2. Burnt binding wire and galvanized binding wire of quality BE22 were used as reinforcement, with PVC tubes of 25.4 mm diameter. A plan of the reinforcement and a cross-section of the model are given in Figure 7. The theoretical moments of inertia of the model are:

$$I_x = 3.199 = \frac{8777}{14^3} \text{ cm}^4/\text{cm and } I_y = 2.419 \text{ cm}^4/\text{cm}.$$

The influence of the presence of the PVC tubes, which increase the bending rigidity, was examined by way of tests on small prismatic beams (140.9 cm x 10 cm x 3.6 cm). A decrease in deflection of 6 per cent was obtained.

The model plate was mounted in a metal frame, bearing on strips of plastic material. Lifting of the corners was prevented by clamps. The variation of the deflection was measured at several points, the deflections recorded at the centre of the plate are indicated in Figure 8. The line I is the theoretical deflection, calculated according to the theory of orthotropic plates (crack-free section), such that:

$$w = 0.0078 \frac{pL^4}{K_x},$$

but diminished by 6 per cent on account of the PVC tubes. The equation of this line is then: w = 2.287 p (for w in mm, p in tonnes/m^2). The line II is the theoretical deflection line based on the supposition that in both the x and y directions the plate is completely cracked and that in consequence the torsional rigidity is reduced to one tenth of the original value (4).

This line, however, is absolutely useless for checking the deflections under service load. The deflections in service agree very well with the theoretically calculated values of line I. From this model test a central deflection of 14 x 2.05 = 28.7 mm was obtained at a load of 865 kg/m^2.

Simulation of the Repair

In order to reduce the deflection of the roof-plate to normal proportions the following repair procedure was investigated.

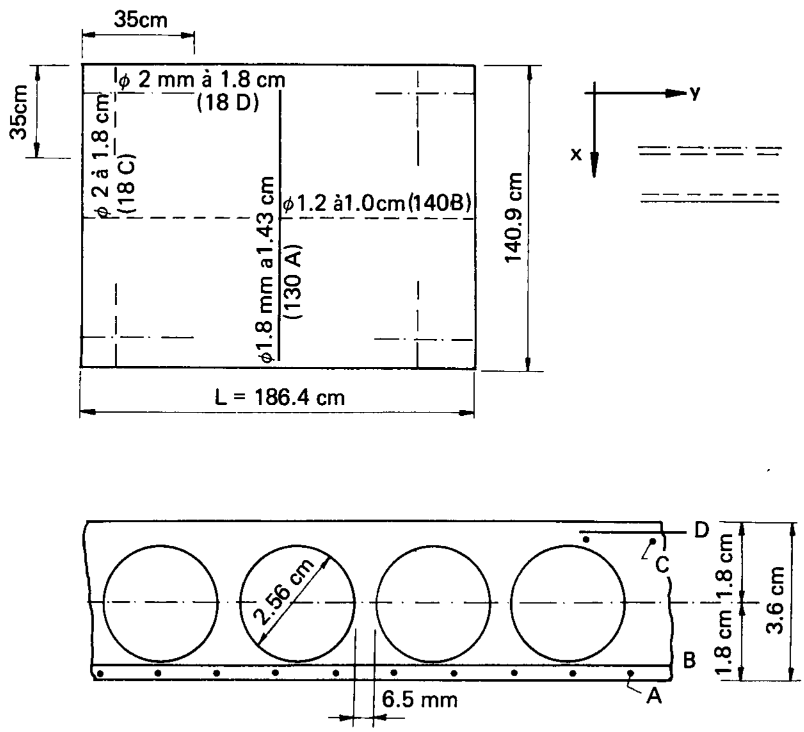


Figure 7 Plan of Reinforcement of the Model Plate

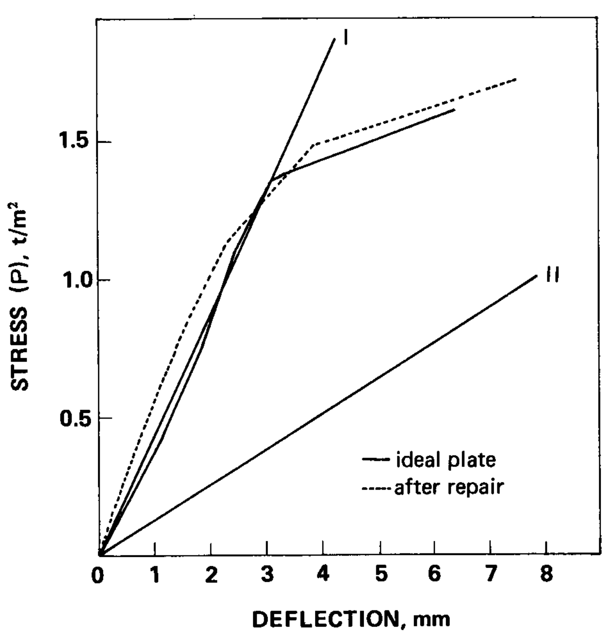


Figure 8 Deflection of the Centre of the Model Plate

Firstly, the plate is raised by means of hydraulic jacks up to the original level of the formwork (pre-camber of 5 cm). Thereupon, a new concrete compression layer, which is sufficiently anchored on to the old plate, is applied.

The new concrete compression layer is 200 mm thick and is applied over a central area of 10 m x 22 m. This concrete would in part replace the sloping concrete. Altogether some 50 m^3 of concrete would be applied and the total load on the plate, which originally amounted to 19.53 x 25.9 x 0.965 = 488 tonnes, would then be increased by an additional, almost uniformly distributed, load of 50 x (2.5 - 0.8) = 85 tonnes. An accurate calculation of the distribution of moments in the newly formed plate seems practically impossible, because of the great number of uncertainties which still exist in this new situation, viz:

did the cracks which existed in the plate before the jacking up close on lifting or not?

was the reinforcement steel already plastically deformed before lifting?

how did the inner cracks develop in consequence of the jacking up?

A reasonable estimate of the moments to be expected can be obtained by multiplying the original moments in the ideal roof-plate by a coefficient obtained as:

$$\frac{488 + 85}{488} = \frac{573}{488} = 1.172;$$

that is, the ratio of the final total load to the original load. From Figure 5 it can be seen that the moments of the ideal plate (ρ = 1.32; β = 0.75) amount to:

$$m_x = 0.072\ pL^2 = 0.072 \times 9.65 \times 19.525^2 = 265 \text{ kNm/m}$$
$$m_y = 0.028\ pL^2 = 0.028 \times 9.65 \times 19.525^2 = 103 \text{ kNm/m}$$

After the repair these moments would be increased to:

$$m_x = 1.172 \times 265 = 311 \text{ kNm/m}$$
$$m_y = 1.172 \times 103 = 121 \text{ kNm/m}$$

A quick check shows that the original reinforcement is sufficient to take this new force:

$$A1 = \frac{31 \times 1000000}{0.9 \times 600 \times 240} = 2399 \text{ mm}^2$$

$$A2 = \frac{12 \times 1000000}{0.9 \times 575 \times 240} = 974 \text{ mm}^2$$

However there remains the problem of the anchorage to take up the shearing force. For that purpose two provisions are made:

i) At the edge of the additional layer, the old concrete is partly hewn out down to a depth of some 70 mm, so that a buffer edge some 70 mm high is formed. On the assumption that this edge can resist a stress of 4 N/mm^2, a force of 4 x 70 x 100 = 280000 N/m can be accommodated. In the longitudinal direction there is a shearing force of 121/(0.9 x 0.575) = 234 kN/m (< 280 kN/m) which must be accommodated while in the transverse direction 311/(0.9 x 0.60) = 576 kN/m must be resisted. The buffer edge will take up 280 kN/m of this force, leaving a remainder of (576 - 280) = 296 kN/m to be resisted by other means.

ii) Every 900 mm, anchoring holes with a diameter of 350 mm are bored (every two tubes). In these holes two inclined anchoring bars of 22 mm diameter are placed which together can take a shearing force of $2[(\pi\ 22^2/4) \times 240 \times \sqrt{2}/2] = 129$ kN per hole or 143 kN/m.

In the next row of holes (further in towards the centre) the two anchoring bars are crossed. These bars can then take a total shearing force of 1.5 x 143 = 214 kN/m. The remaining 296 - 214 = 82 kN/m being taken up at the ridge of the longitudinal grooves of 600 mm width, which are sawn out on both sides in the longitudinal direction.

This programme of repair was tried out on the model plate. The disposition of the added concrete layer, the anchoring holes, the ridge and the reinforcement are shown in Figure 9.

Before the application of the reinforcements the plate was placed upside-down under the press, and by means of two distribution beams a pressure was exerted on the plate at the groove previously sawn out until it showed a deflection of 4 mm, corresponding to a pre-camber of the roof-plate of 50 mm.

To achieve this effect a force of some 20 kN was needed, which corresponds to a load of some 8 kN/m^2, being almost the self-weight of the plate. So for the jacking of the roof-plate itself a total pressure capacity in the jacks equal to the self-weight of the plate would suffice.

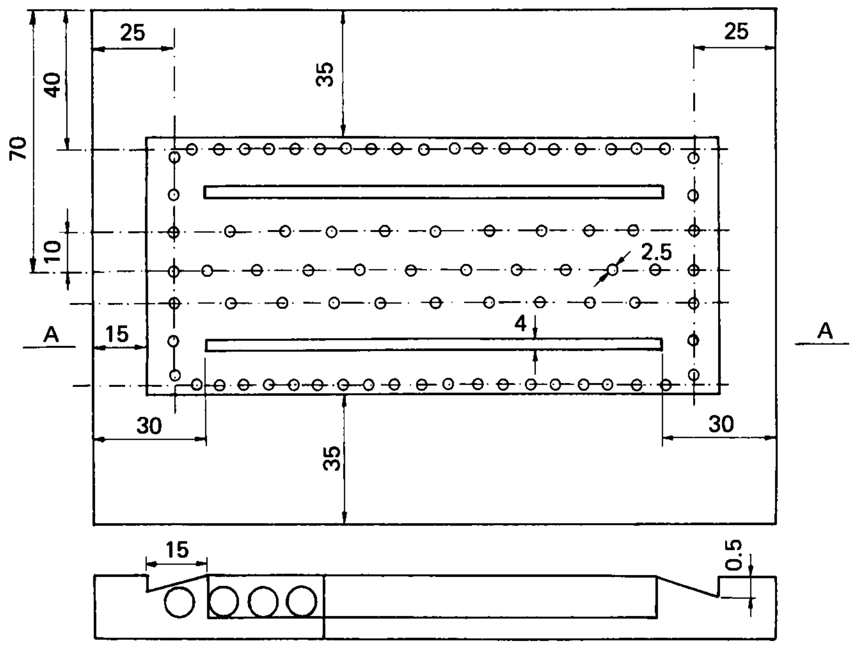
After placing the concrete of the new compression layer, which had the same composition as the plate itself, the repaired model plate was set up and loaded in the same way as the original model plate. The deflection of the centre of the plate was recorded and again plotted in Figure 8 (dotted line). Under a service load of 11330 N/m^2 (= 9650 + 1680 because of the concrete poured on) an elastic deflection of 14 x 2.3 = 32.2 mm may be expected. The repair proposed, as simulated on the test model, appears to be realistic. The repaired model plate behaved strictly as expected, which justified the decision to repair the real plate according to the programme proposed.

ACTUAL REPAIR

The programme of repair on the actual plate was, for practical reasons, slightly adapted according to the following scheme.

Preparation

A total of 18 jacks, each of 200 kN capacity (18 x 200 = 3800 kN ≈ self-weight of the plate) were set up. Each jack was placed under a prop (tripod), type Röro, which bore against the roof-plate through two longitudinal load distributing beams (HEA 120), see Figures 10 and 11. The disposition of the jacks in plan ($S_1 \ldots S_{18}$) is shown in Figure 12. Each jacking unit is composed of two tripods, only one of which rests upon the hydraulic jack. In plan the jacks were placed alternately



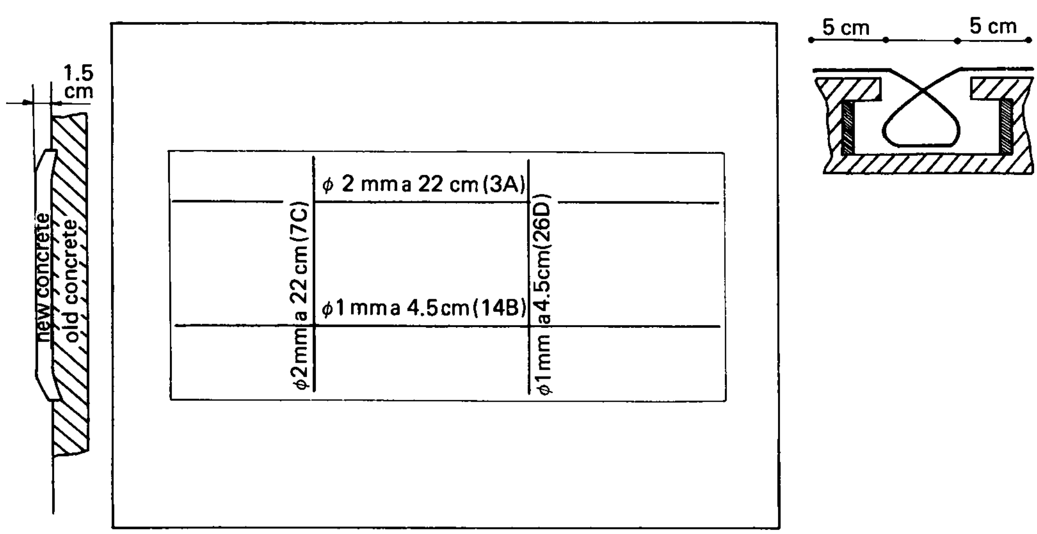


Figure 9 Preparation of the Plate for the Repair with a New Compression Layer

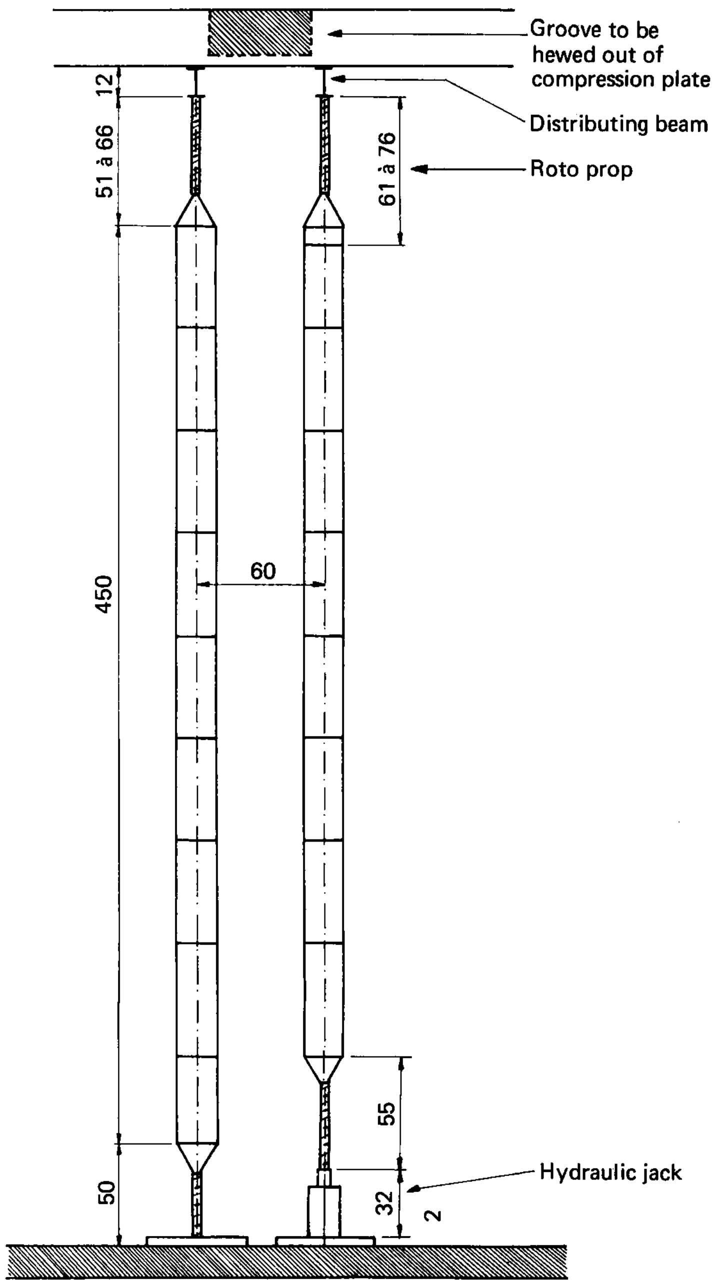


Figure 10 Arrangement of Props and Jacks

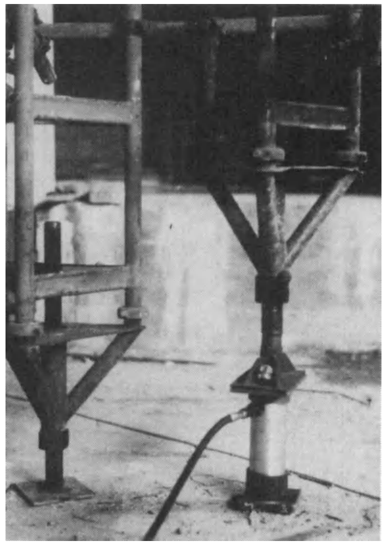


Figure 11 Detail of Prop Unit

under the left and the right prop of the twin prop support unit so as to distribute the load as uniformly as possible.

The reason for doubling up the tripod props is that during jacking the second prop must be readjusted immediately, so that in case of any defect in the hydraulic system the position of the plate should be maintained. The floor on which the jacks and the auxiliary props rested was propped from foundation level in order to carry the pressure of the jacks without overstressing the floor.

The jacking operation was controlled by way of two sets of hydraulic pumps and the movement of the roof was monitored during the jacking operation at 9 points, M1 to M9, see Figure 12. During the jacking, the plate cracked on the upper side, as was expected. Each of the two longitudinal cracks developed between the supporting beams, see plan Figure 12. A number of cracks also formed in the transverse direction; this was surprising and did not conform to expectations.

After the jacking was completed the two longitudinal grooves were sawn. This operation was carried out after jacking in order to give the plate a greater homogeneity during the delicate lifting operation. The risk that the longitudinal cracks might have developed beside the supports was relatively small.

After providing two saw-cuts of some 400 mm depth, the concrete between was hewn out, so that a longitudinal groove was formed between the distributing beams, see Figure 13. In the grooves it was clearly noticeable that practically 60 per cent of the tubes showed insufficient cover. In several places there was only a cover of 10 to 20 mm. At the same time it was noticed that in other places the tubes had become oval in such a way that the expected spacing of 100 mm between successive tubes was not obtained; in several places the tubes actually touched. To examine the formation of the unexpected cracks in the transverse bearing direction the lower reinforcement in the grooves was exposed. It showed that only 50 per cent

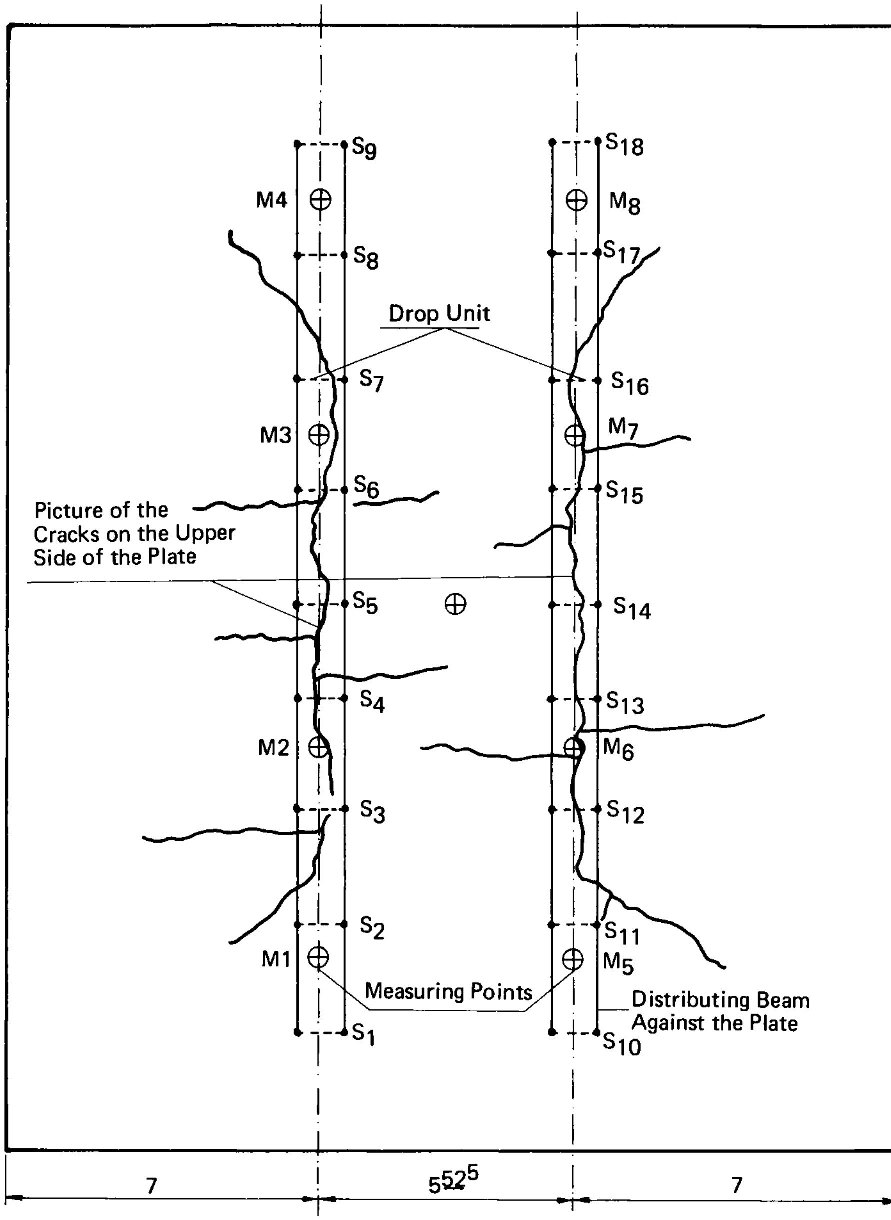


Figure 12 Plan of the Props S and the Measuring Points M. Picture of the Cracks on the Upper Side of the Plate.

of the longitudinal reinforcement (16 mm bars at 250 mm centres instead of 16 mm bars at 125 mm centres) was present.

This may explain why the plate showed cracks in the short direction during the jacking. Indeed, as only 50 per cent of the required reinforcement had to resist the total moment, the plastic threshold was crossed. An irreversible elongation

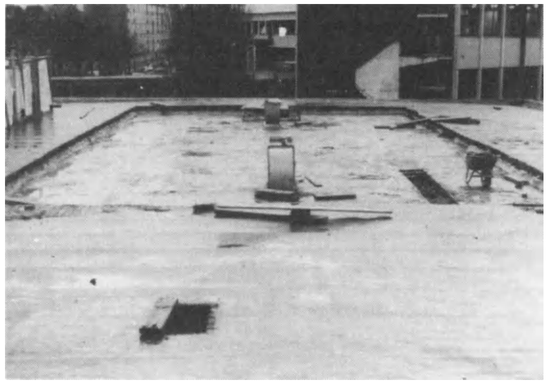


Figure 13 After Removal of the Roofing Material the two Grooves were Sawn

in the steel had been formed so that on jacking cracks were bound to develop perpendicularly to the 16 mm diameter reinforcement, that is to say, according to the shortest bearing direction of the plate.

Adaptation of the Scheme of Repair

To repair the bending rigidity as much as possible, the new compression plate was extended while its thickness was reduced to 150 mm so as not to increase the total volume, and thus weight, of concrete unnecessarily.

The grooves were each elongated by 4 metres closer to the edge and the number of anchoring holes was increased.

The missing longitudinal reinforcement was concentrated into the two grooves, after hewing the concrete from the bottom of the grooves. In each groove eleven 25 mm diameter bars were added. At the ends of the grooves inclined bars provided the necessary anchorage. Where the mutual distance of the tubes was insufficient, a tube was exposed and the cardboard removed. In this way a wide rib was formed from the groove up the edge. To ensure good adhesion between old and new concrete in the vicinity of the anchoring holes, the cardboard of the tubes was locally removed. A masonry partition ensured that no concrete could flow into the tubes during placing, see Figure 14. Figure 15 shows the detail reinforcement of the anchorages from the groove and also from the separate anchoring holes. A general picture of the reinforcement is given in Figure 16; this photograph was taken just before the concrete was poured.

Checking of the Repaired Plate

As a check on the efficiency of the repair, the deflections in the centre of the plate as well as the strains on the upper side of the new compression plate were measured after removal of the formwork.

After releasing the props, an instantaneous deflection of 78.8 mm was measured. Part of this deflection results from the shrinkage of the new concrete plate, the

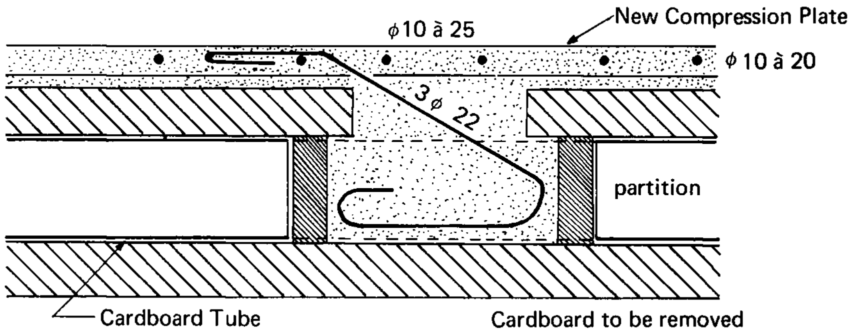


Figure 14 Detail of Anchorage Hole

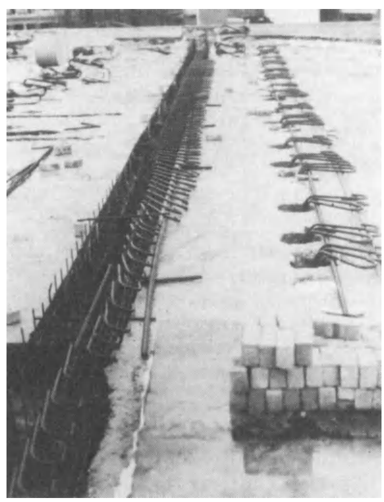


Figure 15 Reinforcement in Place in the Anchoring Holes

deflection due to shrinkage being approximately calculated from the formula:

$$y_r = \frac{1}{\rho} \cdot \frac{L^2}{8} = \frac{L^2 . \Sigma_r}{8h}$$

where ε_r is the concrete shrinkage in the newly poured upper layer. Due to the difference in length of the edges of the plate the deflection due to shrinkage is reduced in the longitudinal direction, whereas it is increased in the other direction.

That reduction or increase may be represented as a uniformly distributed load, p, which acts in one direction positively and in the other negatively. Ignoring the

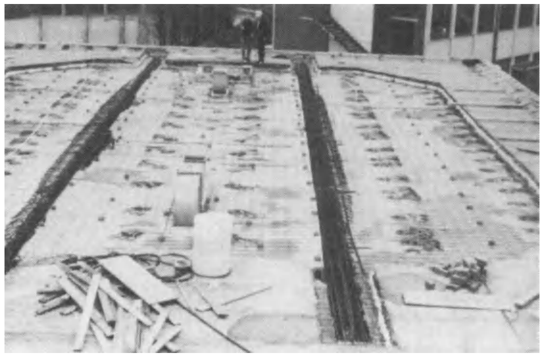


Figure 16 General View of the Reinforcement

torsional rigidity, the beam action of the plate is made compatible, as far as the deflection in the centre is concerned, as:

$$\frac{L_1^2\ \varepsilon_r}{8h} + \frac{5\ pL_1^4}{384\ EI_1} = \frac{L_2^2\ \varepsilon_r}{8h} - \frac{5\ pL_2^4}{384\ EI_2}$$

$$p = \frac{384}{5}\ \frac{\varepsilon_r L_1^2}{8h}\ \frac{\left[\left(\frac{L_2}{L_1}\right)^2 - 1\right] EI_1}{L_1^4\left[1 + \left(\frac{L_2}{L_1}\right)^4 \frac{I_1}{I_2}\right]}$$

with ε_r = 0.0003
h = 450 mm
I_1 = 842787 cm^4 (cracked state)
I_2 = 600841 cm^4 (cracked state)
L_1 = 19600 mm
L_2 = 25800 mm,
one obtains $(y_{max})_r$ = 36.5 mm

The resulting instantaneous elastic deflection then computes as y_e = 78.8 -36.5 = 42.3 mm.

On the scale model an elastic deflection of 32 mm (converted to the real plate situation) was measured. The influence of the reduction of the compression plate from 200 mm to 150 mm may be introduced into the calculation by way of the ratio of moments of inertia, corresponding to a compression plate of 150 or 200 mm respectively. The 32 mm deflection measured would then increase to

$$32 \times \frac{1007091}{842787} = 38.3 \text{ mm}$$

Comparison of this elastic deflection, calculated from measurements on the scale model, with the real deflection shows a difference which may be taken as insignificant. As a matter of fact, it must not be forgotten that:

i) the grooves were elongated,

ii) the missing longitudinal reinforcement was concentrated into the grooves,

iii) the longitudinal reinforcement was deformed, in consequence of which an initial load acted on the plate.

The relative deformations in the directions x and y give a measure of the bending moments occurring. The ratio of the deformations measured was:

$$\frac{\varepsilon_2}{\varepsilon_1} = \frac{0.13 \times 10^{-3}}{0.23 \times 10^{-3}} = 0.5652$$

which agrees very well with the ratio of the moments:

$$\frac{M_2}{M_1} = \frac{118}{228} = 0.518$$

The relatively small difference between $\varepsilon_2/\varepsilon_1$ and M_2/M_1 is an additional guarantee of the correct functioning of the repaired plate.

CONCLUSIONS

The successful repair of this roof-plate at Schaarbeek shows that besides demolishing and re-building there still lies fallow for the structural engineer a very exciting domain, namely the repair of faulty structures.

Any repair has to be based on a detailed investigation into the causes of the faults.

If an important structure is to be dealt with, a simulation of the repair on a scale model combined with a far reaching theoretical study, if necessary, seems to be the obvious route to come, with some certainty, to the right remedial treatment. If the measurements agree with expectations, the structure can be deemed serviceable.

To justify repair the cost of study, repairs and testing has, of course, to remain lower than the cost of demolishing and rebuilding of the faulty structure. The repairing of constructional elements which exhibit faults is a captivating and promising technique, both from an economical and a scientific point of view. A recently developed technique which can contribute a good deal to this is the technique of glued-on external reinforcement (6). Perhaps this path in the field of research has been followed too little so far, but it is the authors' hope that this contribution may rouse interest in this only sporadically exploited field!

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THE EARLY AGE BEHAVIOUR OF A MASSIVE REINFORCED CONCRETE SUSPENDED SLAB

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ABSTRACT Temperatures and strains resulting from the heat evolved in hydrating concrete have been recorded to observe the early age behaviour of a massive, cement-rich, reinforced concrete, suspended floor slab. Values of temperature rise and thermal movement have been compared with predicted values and show good agreement. The significance of predicting the temperature rise and temperature variations within a concrete pour together with the level of external restraint is discussed in relation to the likelihood of cracking during its early life. The importance of adequate curing is highlighted.

The high steel content, the low level of restraint and the use of surface insulation resulted in virtually crack free concrete in the slab examined.

INTRODUCTION

Queen Annes Mansions is a twelve storey office block with an adjacent sixteen storey tower situated in Petty France, London, see Figure 1. The building was constructed in reinforced concrete by Taylor Woodrow Construction Limited for the Department of the Environment. To accommodate architectural requirements for a change in floor configuration, massive, heavily reinforced beams (with sections up to 2.60 m x 1.07 m) were constructed at the third floor level. In view of the size of the beams and the necessity for a high cement content it was felt that problems may have arisen due to the expected high temperature rise which would occur as a result of heat generated by hydration of the cement. In particular, consideration was given to the likelihood of cracking which may have occurred due to,

a) Differential thermal strains within the massive beams;
b) Differential thermal movements between the beams and a 125mm lightly reinforced slab spanning between the beams;
c) External restraint to thermal movements.

A preliminary analysis of thermal strains, based on estimated values of temperature rise, indicated that extensive cracking was likely to occur if precautionary measures were not taken. It was, therefore, decided to insulate the top surface in an attempt to achieve compatibility of movement between the beams and slab and to reduce thermal gradients in the beams themselves. The soffit formwork consisted of 20 mm plywood which itself provides good thermal insulation.

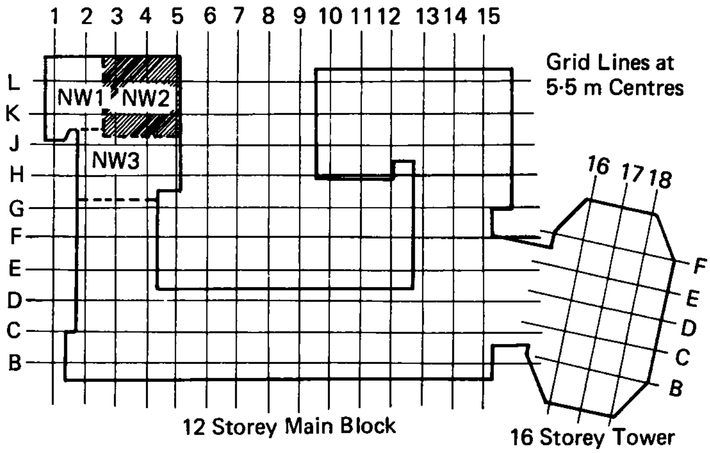

Figure 1 General Plan of Queen Annes Mansions showing Location of Instrumented Bay

To substantiate the analytical results and provide additional information relating to early age concrete behaviour it was decided that a typical bay of the third floor should be monitored, Figure 1. The subsequent programme of in-situ instrumentation was carried out jointly between the Taylor Woodrow Research Laboratories and the Cement and Concrete Association to provide information on the following:

i) Temperature variations through the pour;
ii) The effectiveness of surface insulation;
iii) Thermal movements within the beam and slab;
iv) The degree of restraint to early thermal movement;
v) The behaviour of construction joints;
vi) The degree of column movement.

Bay Size and Configuration

The configuration of the instrumented bay, NW2, is illustrated in Figure 2. External restraint to thermal movement was offered by the supporting columns and soffit formwork as well as by previously cast concrete in bays NW1 and NW3 along two adjacent sides. The bay covered an area of 9.5 m x 10.7 m and comprised approximately 75 m^3 of concrete.

Concrete Mix Details

The concrete, supplied by Greenhams Ready Mixed Concrete Limited, was designed to achieve a 28 day characteristic cube strength of 41.5 N/mm^2 with a slump of 75 mm. Concrete mix proportions are given in Table 1. A sea-dredged gravel aggregate was used together with a zone 2 natural sand. The cube strengths achieved are also included in Table 1.

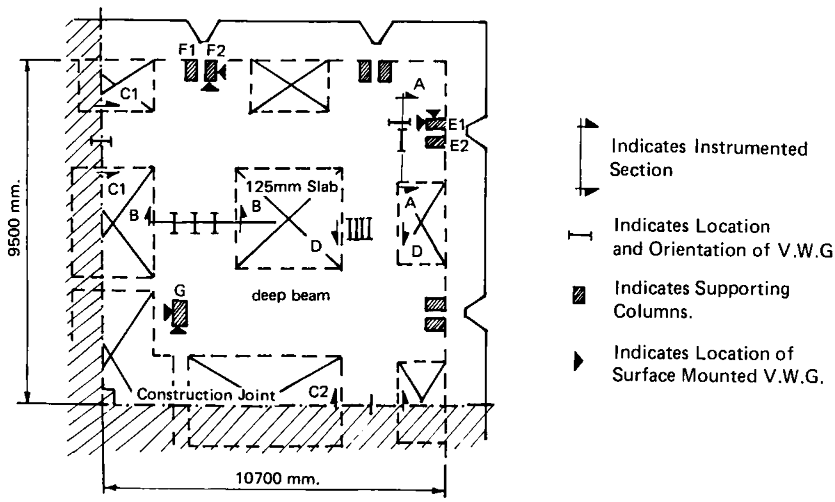

Figure 2 Configuration of Bay NW2 showing Location of Instrumented Sections

Table 1 Concrete Mix Details

MATERIAL	BATCH QUANTITIES kg/m^3
Ordinary Portland Cement	480
Sand	650
Sea-dredged Gravel	
20-10 mm	730
10-5 mm	390
Water	175
Plastocrete Admixture	0.875
Compressive Cube Strength, N/mm^2	
7 days	38.8
28 days	51.6

Concrete Placement and Curing

Concrete was placed using both pump and skip over a period of 6 hours. The surface of the concrete was sprayed with a curing membrane and covered with 50 mm polyurethane-ether-foam blankets which were subsequently saturated with water. The blankets were removed after 4 days, the area being required for storage.

STRAIN GAUGES AND THERMOCOUPLES

Copper/constantan thermocouples were used to monitor temperature variations and embedded vibrating wire strain gauges (VWG's) were used to monitor movements within the concrete. Of the 27 embedded VWG's, 14 were of the acrylic type supplied by Taylor Woodrow and 13 were steel, supplied by the Cement and Concrete Association. In general there was little difference between the performance of the two types of gauge once the concrete had hardened. During the hardening of the concrete the acrylic gauges, being less stiff, were more responsive to thermal movement. A total of 18 surface mounted VWG's were used to monitor surface strain in supporting columns. Demec gauge readings were also taken to compliment the VWG results. Thermocouples and strain gauges were monitored automatically for a period of 16 days.

The location of the five sections through which embedded strain gauges and thermocouples were cast is shown in Figure 2, in which instrumented supporting columns are also indicated. The beams were, by necessity, heavily reinforced as shown in Figure 3. Reinforcement details through typical instrumented sections B and D are illustrated in Figure 4.

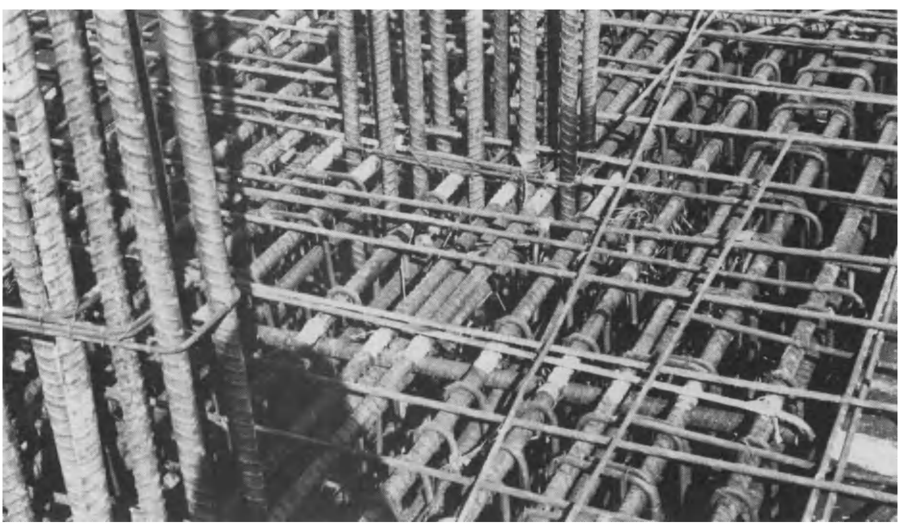

Figure 3 View of Reinforcement Close to Section D

The specific locations of embedded thermocouples and VWG's within each section are illustrated in Figure 5. At each of three sections A, B and D remote from construction joints the temperature variation was monitored over approximately half the beam width using a grid of up to 25 thermocouples. Strain was only monitored on one or two axes through the centre of each section. Only two thermocouples and VWG's were cast across each of the two construction joints at the centre of sections C1 and C2.

One thermocouple and two VWG's (one acrylic and one steel) were also cast into an insulated 300 mm cube of concrete to monitor the free or unrestrained thermal movement during the early age temperature cycle. Surface mounted VWG's were located at

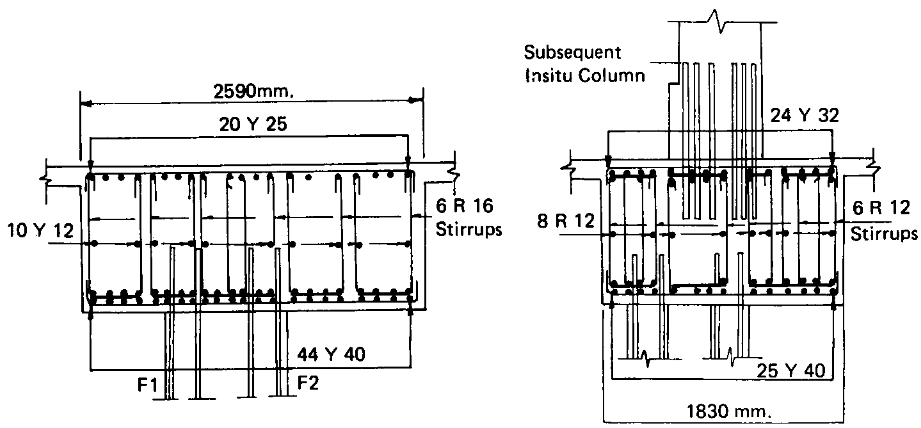

Figure 4 Details of Reinforcement at Section D (left) and Section B

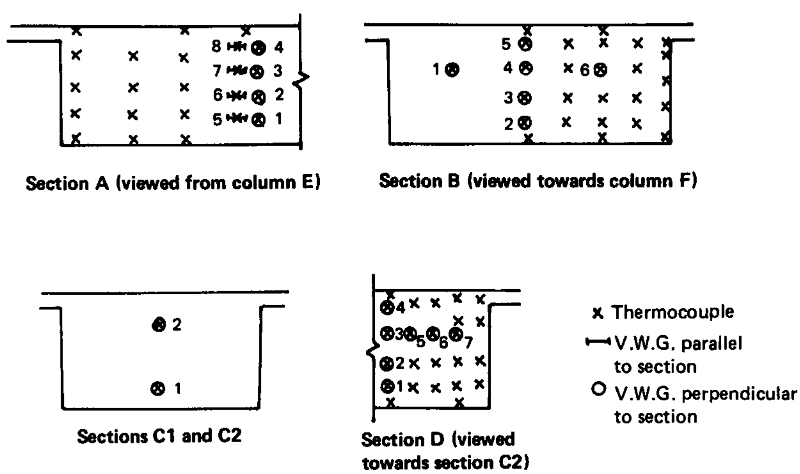

Figure 5 Location of VWG's and Thermocouples within Each Section

three levels, 1220, 1600 and 2440 mm above second floor level on two faces of each of the three instrumented columns E, F and G. Demec studs were located on either side of each surface mounted VWG to provide supplementary data.

THERMOCOUPLE RESULTS

Values of temperature were recorded throughout the duration of the early age temperature cycle at each thermocouple location. Typical values recorded at the centre of sections A, B and D are illustrated in Figure 6. At section A, the most massive volume of concrete in the bay, the maximum recorded temperature rise was 57°C occurring at 36 hours after casting, which is equivalent to 11.9°C/100 kg cement. This value is close to the value of 12°C/100 kg suggested by Fitzgibbon (1) and commonly used for estimating the temperature rise due to hydration in mass concrete. In the smaller sections B and D, values of maximum temperatures rise were 53°C and 47°C respectively. The initial concrete mix temperature was generally 16 to 20°C.

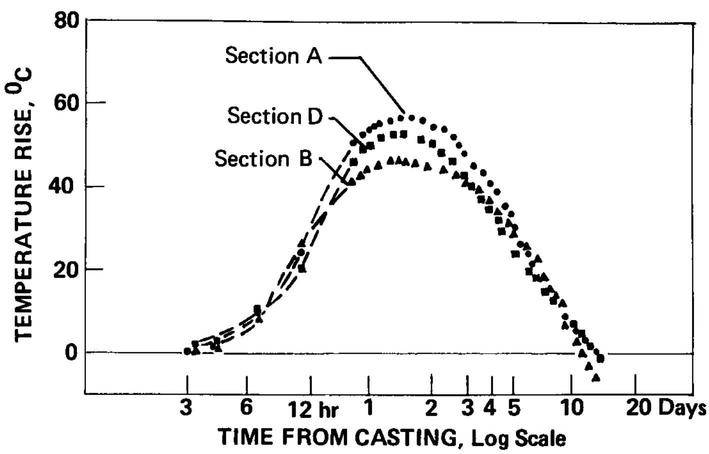

Figure 6 Temperature Rise Curves Recorded at the Centre of Sections A, B and D

TEMPERATURE GRADIENTS

Temperature rise in itself does not result in thermal stressing and cracking. It is differential or restrained thermal movement which, if excessive, can be harmful. Isotherms based on the thermocouple results at sections A, B and D at the time of peak temperature rise have been constructed and are illustrated in Figure 7. The irregularity of the isotherms at sections A and B is believed to be due in each case to the presence of a concentration of vertical reinforcement from columns, the specific heat and conductivity of steel being high in relation to that of concrete. It can be seen from Figure 7 that the maximum recorded temperature differential within each section was never greater than 20°C and differentials between the centre and the surface were generally considerably smaller, being of the order of 12 to 14°C. A maximum acceptable value of temperature differential of 20°C has been recommended to avoid cracking (1) and values within the beam sections were certainly below this level.

Had surface insulation not been used, however, temperature differentials in excess of 40°C would have been likely to occur as shown by calculations carried out to establish the temperature rise and thermal gradients through the beam at section A. The results are shown in Figure 8 compared with the recorded temperature gradient. The benefit gained from the use of insulation is clear, the temperature differential between the centre and surface of the beam being reduced by 50 per cent.

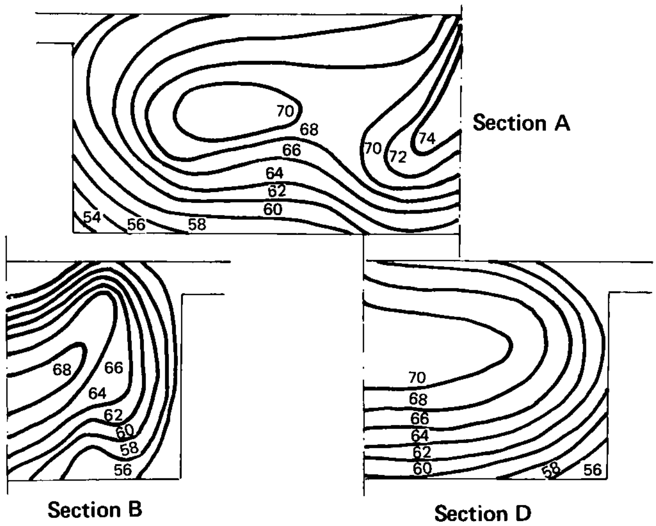

Figure 7 Isotherms Recorded at the Time of Peak Temperature Rise

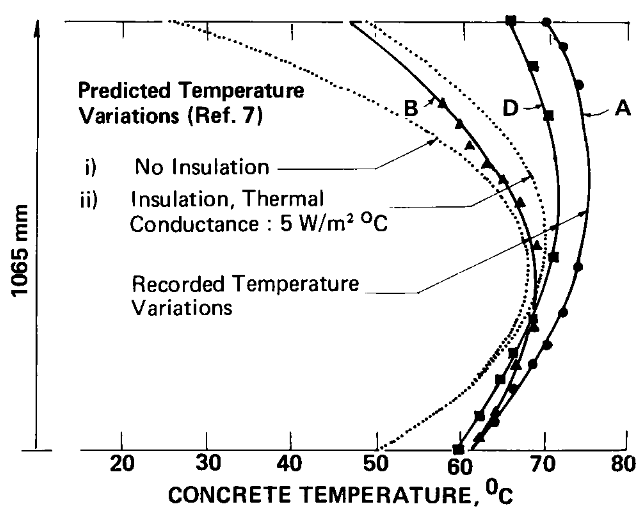

Figure 8 Estimated Thermal Gradients Through the Deep Beams

PREDICTION OF CRACKING RESULTING FROM THERMAL GRADIENTS

In a concrete pour totally free from external restraint, cracking will only occur at early age as a result of thermal gradients. Consider a typical pour in which the temperature distribution through the thickness at the time of peak temperature is approximately parabolic. The tensile stress-causing strain developed at the surface, ε_s, will be proportional to two-thirds of the difference between the surface temperature, θ_s, and the temperature at the centre, θ_c. Because concrete is relatively soft, however, with a high rate of creep during its early life the stress-causing strain component does not fully develop. In-situ measurements have indicated that less than half the expected value occurs (2). The stress causing strain is, therefore, unlikely to exceed:

$$\varepsilon_s = \frac{1}{2} \times \frac{2}{3} (\theta_c - \theta_s) \alpha$$

where α is the coefficient of thermal expansion of the concrete. To avoid cracking, the value of ε_s must be less than the tensile strain capacity of the concrete, $\varepsilon_{s(max)}$, hence:

$$\varepsilon_{s(max)} > \frac{1}{3} (\theta_c - \theta_s) \alpha$$

Rearranging:

$$(\theta_c - \theta_s) < 3 \times \varepsilon_{s(max)}/\alpha$$

In this particular case a flint gravel aggregate was used with a measured thermal expansion coefficient, α, of $13.2 \times 10^{-6}/°C$ and an estimated tensile strain capacity of about 75×10^{-6} (3).

Hence:

$$(\theta_c - \theta_s) < \frac{3 \times 75}{13.2} \simeq 17°C$$

This value is close to the recommended limiting value of 20°C. The fact that it is lower suggests that creep relief is more effective than was assumed or that the temperature distribution is not perfectly parabolic.

The Use of Alternative Aggregate

If a limestone aggregate had been used in place of the flint gravel the limiting temperature differential could have been increased. The α value of concrete containing limestone is about $8 \times 10^{-6}/°C$ (4) and the tensile strain capacity, $\varepsilon_{s(max)}$, is of the order of 90×10^{-6} (3). Hence:

$$(\theta_c - \theta_s) < \frac{3 \times 90}{8} \simeq 34°C$$

A change in aggregate alone can, therefore, significantly increase the acceptable level of temperature differential. Recommended values of maximum temperature differential for different aggregate types are given in Table 2.

Various methods exist for predicting the temperature rise in concrete resulting from hydration, based on a knowledge of the heat generating properties of the cement, the cement content of the concrete, the size of pour and the environmental conditions (5, 6, 7). However, to achieve a first order estimate, the temperature rise can be assumed to be directly proportional to the cement content and influenced primarily by the minimum dimension of the pour. A relationship between lift height and specific temperature rise is illustrated in Figure 9, based on the results of thermocouple measurements in a range of pours undertaken by Taylor Woodrow (2, 8, 9, 10) and other workers (11, 12). The results generally apply to uninsulated concrete; an estimated curve for concrete with insulation having a thermal transmittance of $5 W/m^2 °C$ is also included.

Table 2 Acceptable Temperature Differentials using Different Aggregate Types

AGGREGATE TYPE	THERMAL EXPANSION* COEFFICIENT OF CONCRETE x 10^{-6}/°C	TENSILE STRAIN† CAPACITY OF CONCRETE x 10^{-6}	LIMITING TEMPERATURE DIFFERENTIAL °C
Rounded Quartz	12.1	67	16.6
Crushed Quartzite	12.1	70	17.4
Granite	9.6	80	25.0
Limestone	8.6	90	31.4

* See Ref.2 † See Ref.3

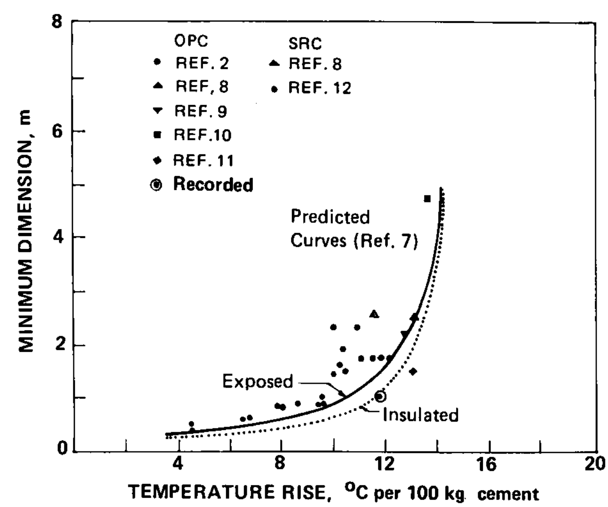

Figure 9 Relationship Between Lift Height and Temperature Rise

In large pours the effect on the maximum temperature rise is insignificant. In smaller pours the effect, in terms of temperature rise, is to increase the pour size. The measured values agreed closely with the estimated relationship.

THERMAL STRAIN

Values of strain were monitored at each VWG location throughout the duration of the early age temperature cycle. Typical values are illustrated in Figure 10, recorded at the centre of sections A, B and D. It is not the measured strain, ε_m, however, which causes the development of thermal stress, it is the restrained strain, ε_r, this being the difference between the free thermal strain, ε_f, and the measured strain at a particular point. The free thermal expansion of the concrete was measured in the 300 mm insulated cube and found to have a value of 13.2 x 10^{-6}/°C, which is a typical value for concrete with a flint gravel aggregate having a relatively high silica content (4).

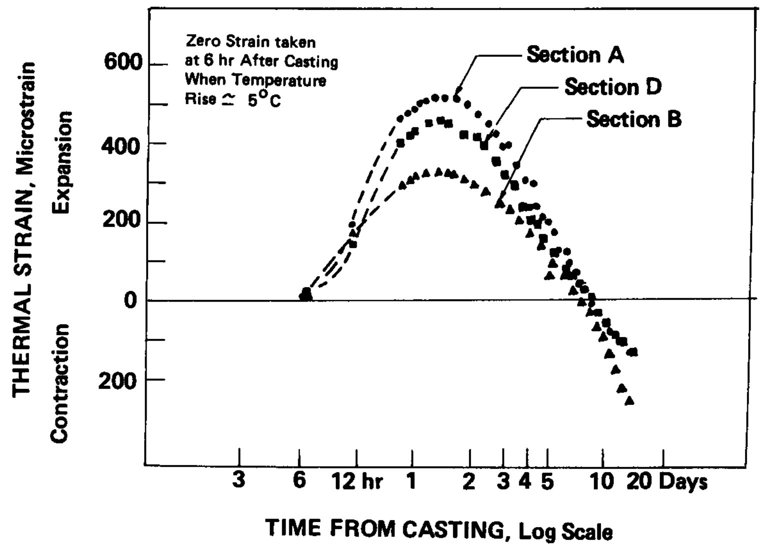

Figure 10 Thermal Strain Recorded at the Centre of Sections A, B and D

RESTRAINT FACTOR

The restraint factor R, used to establish the level of thermal stress and the likelihood of cracking, is calculated using the equation:

$$R = \frac{(\varepsilon_f \times \varepsilon_m)}{\varepsilon_f} \times 100\%$$

Values of R determined at each VWG location are given in Table 3, together with the values of thermal expansion coefficient obtained at each point from the temperature-strain relationships. Some typical relationships are shown in Figure 11. External restraint to thermal movement was generally of the order of 15 to 30 per cent resulting from a combination of the following:

a) Localised restraint from supporting columns,
b) Edge restraint from adjacent hardened concrete in bays NW1 and NW3,
c) Base restraint from the soffit formwork,
d) Internal restraint from reinforcing steel.

The precise degree of column restraint can be calculated very simply by relating the force-deflection relationship at the top of the column to the restraining force-expansion relationship for the beam, to achieve compatibility of movement at the column head. In this instance the column restraint was insignificant, the maximum stiffness of a pair of columns being only 1/200th of the axial stiffness of the massive heavily reinforced beams.

Edge restraint was significant, the restraint factors determined at section B, close to the adjacent bay NW1, being up to 20 per cent higher than values in sections A and D remote from adjacent bays. General base restraint from soffit formwork and internal restraint by reinforcing steel (which had a lower thermal expansion coefficient than the concrete) resulted in a restraint factor of about

Table 3 Values of Restraint Recorded at Each VWG Location

SECTION A			SECTION B			SECTIONS C1 & C2			SECTION D		
GAUGE NO.	α	R	GAUGE NO.	α	R	GAUGE NO.	α	R	GAUGE NO.	α	R
1	11.8	11	1	-	-	C1-1	12.3	7	1	11.1	16
2	10.6	20	2	10.6	20	C1-2	9.6	28	2	10.6	20
3	10.6	20	3	9.9	25				3	11.3	15
4	11.4	14	4	9.3	30	C2-1	11.4	14	4	19.5	-48
5	12.0	9	5	8.8	33	C2-2	12.0	10	5	11.5	13
6	10.6	20	6	10.6	20				6	11.0	17
7	10.0	25							7	11.3	15
8	-	-									

α Thermal movement coefficient recorded in-situ, $x10^{-6}/^{\circ}C$.
R Restraint factor (%) related to free α = $13.2x10^{-6}/^{\circ}C$.

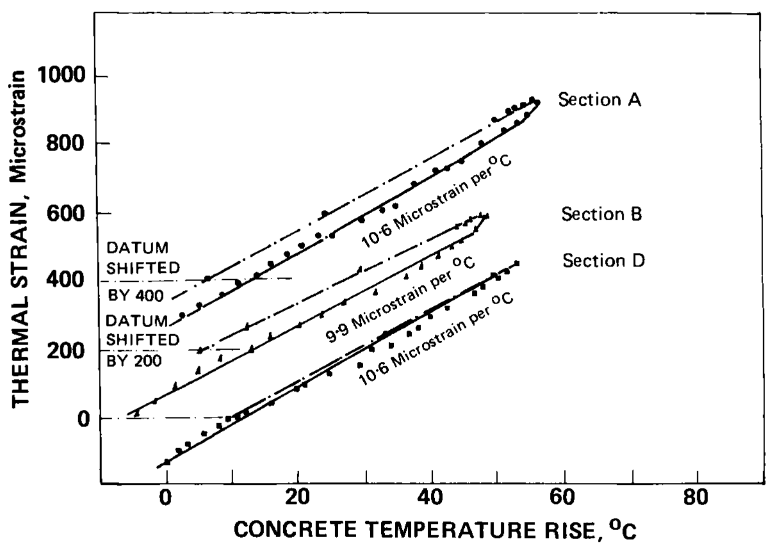

Figure 11 Temperature-Strain Relationships
Recorded at the Centre of Sections A, B and D

15 per cent. The values of restraint were, however, generally low. Some typical values of R recorded for various pour configurations are shown in Table 4. Overall restraint was not, therefore, considered to be a major problem.

Table 4 Values of Restraint Recorded for a Range of Pour Configurations

POUR CONFIGURATION	RESTRAINT FACTOR %
Thin Wall Cast onto Massive Concrete Base.	70 to 100 at base 10 to 20 at top
Massive Deep Pour Cast onto Blinding.	10 to 20
Massive Deep Pour Cast onto Existing Mass Concrete.	30 to 60 at base 10 to 20 at top
Suspended Slab	20 to 40
Infill Bays	80 to 100

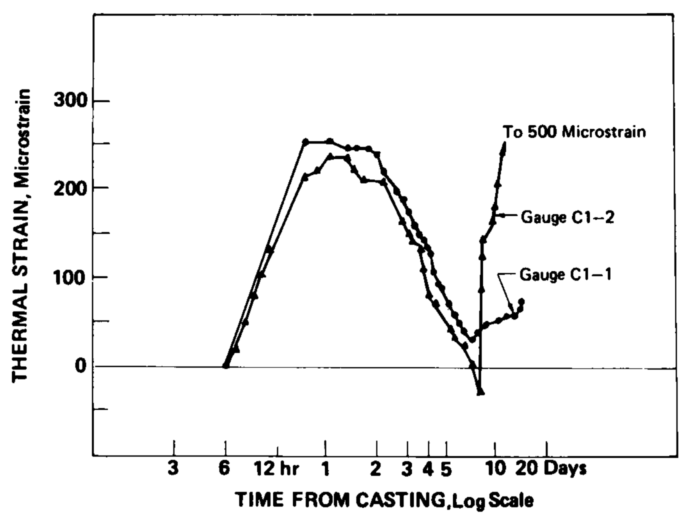

Figure 12 Strain Behaviour Recorded at Construction Joint C1

JOINT OPENING

It is clear from the results of VWG's cast through the joints at sections C1 and C2, that joint opening occurred. This was indicated by a sudden increase in strain at about 7 days after casting as illustrated in Figure 12. The magnitude of joint opening was of the order of 0.05 mm reducing towards the heavily reinforced lower part of the beam.

The mechanism of joint opening is influenced by both the temperature gradient and overall restraint to thermal movement. When a new pour is cast against mature concrete, some of the heat developed is transferred into the adjacent bay. As a result there is a tendency for the mature concrete to expand into the new less rigid bay, more so at the centre where the temperature rise is greatest. On cooling down, not only does the new pour contract, but the adjacent concrete does also, causing the joint to open initially at about mid-height, propagating slowly towards the upper and lower surfaces. This behaviour has been observed on each occasion that construction joints have been monitored (8, 9) and it is clear that eliminating external restraint alone will not eliminate joint opening. This can only be achieved when both external restraint and the thermal gradient through the depth of a joint is small.

CRACKING

Although precautions were taken some fine cracks were observed on the surface of the beams at about 2 days after casting. These were believed to be thermally induced, this being confirmed when the cracks closed completely when the concrete cooled down. One or two plastic shrinkage cracks were also observed, probably due to the curing membrane not being applied soon enough after tamping. These were not considered to be of structural significance, however, and with time tended to knit. No cracks were observed on the 125 mm slabs.

CONCLUSIONS

In considering the effects of temperature rise due to hydration of cement, the following points should be noted:

i) Temperature variation within the concrete should be estimated and excessive temperature gradients avoided.

ii) The constituents of the concrete, primarily the coarse aggregate can substantially effect the thermal behaviour of concrete and should be considered in any calculations. Limestone aggregate is generally most desirable.

iii) The magnitude of restraint should be estimated. External restraints are usually of most concern and can often be accurately predicted. Where restraints are high, steps should be taken to reduce them.

Despite the high cement content and the configuration of the third floor at Queen Annes Mansions, cracking was maintained at an acceptable level by the use of a curing technique comprising of a sprayed on curing membrane followed by 50 mm thick foam blankets. Whilst the temperature of the concrete in the most massive section reached 74°C, the temperature differentials were maintained at less than 20°C, this being the maximum recommended value if cracking is to be avoided in concrete containing gravel aggregate.

Restraint to thermal movement was generally low, the largest part resulting from adjacent concrete. Construction sequences should, therefore, be arranged such that each bay has at least one free face in each direction to permit unrestrained thermal movement.

Restraint from the supporting columns was insignificant. Large area pours encompassing a number of supporting columns may therefore be cast when the slab is stiff in relation to the columns. This would also reduce joint opening by minimising the number of joints.

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DISCUSSION

Paul Poitevin. I have a question for Mr. Gauld about epoxy glued reinforcing steel flats. The idea of bonding steel plates on concrete was developed and patented by L'Hermite more than ten years ago. Some applications have been carried out in France, very few at first but becoming more and more frequent now. My comment is that this process has some limitations. First the eveness of the surface of the concrete must be perfect so we have to do grinding. Second, the steel flats cannot be applied on screeds, they must bond on the original concrete; I think this was the problem for your slab. Thirdly, there is the very important limitation that, to be effective, the steel must be perfectly bonded and so the thickness of the steel flat should be less than 3 or 4 mm, if it is necessary to have greater reinforcement you have to bond another flat of steel on the first. In addition there is a very important limitation for indoor construction, as yours was, which is the fire hazard. Above 100°C the epoxy will collapse, so this technique is most generally applied on the soffits of bridges. My question to you is: you had to repair a beautiful building, but it was an internal repair, have you taken protective measures against the fire hazard?

George Gauld. Thank you for your question. Yes I think many of the points you have made are recognised in France and Switzerland and I would not disagree with any of them. In respect of the protection from fire you are quite correct that both the steel and more particularly the adhesive are vulnerable in the event of fire and protective screeds and blankets would have to be added over the top to provide the necessary minimum fire resistance required by building regulations to prevent the temperature at the steel becoming too high within the required fire period.

Dionys A. van Gemert. I would like to direct a question to Mr. Petersons. What would you do to prepare the surface of the concrete before glueing such plates? Since you have to clean the concrete, what would be best, flame cleaning or mechanical cleaning?

Nils Petersons. Well, we have not compared flame cleaning with mechanical cleaning. Both methods are very suitable, but when you have to repair industrial floors which are oil stained and are subsequently to be covered with some new coating, I think you will have great difficulty finding a better method than flame cleaning. This is because the oil is burnt away and you can at once cover with a new coating. I think both methods are good but flame cleaned surfaces do not need any further treatment.

George Barnbrook. I am interested in the same subject. When considering the preparation of concrete floor surfaces, the topic of flame spalling is obviously of interest. I remember one particular job where a consulting engineer wanted to prepare the surface of an old concrete roof slab where another storey was being added to a building. He wanted to achieve good bond between the slab and a new concrete topping to be added to the surface. Equally he wanted the occupation of the building on the floor below to continue and he was looking for a quiet preparation process rather than mechanical scabbling or the like. I suggested flame spalling, and the people who carry out flame spalling in this country were called in and did some of the work. Sadly, however, it was very ineffective in this particular case; the spalling being very limited. The reason put forward for the ineffectiveness was that the type of aggregate used for the concrete, which was not known at the time, was such that good spalling would not occur. So in the event mechanical scabbling was used and the people on the floor below had to suffer the noise. With this background then, I would welcome comments on the effect of type of aggregate on spalling characteristics.

Nils Petersons. We have made quite a lot of investigations on behalf of the Swedish firm AGA who supply the equipment and gas and we have, among studies of different methods, also studied the influence of aggregates. We used a very broad field of different aggregates and found that the type of aggregate had only a very slight influence, I could say no influence at all. There are some other factors which influence the scaling effect; for example, the moisture content of the concrete is very important and you must use very skilled personnel who are accustomed to the equipment. The results then would be much better I think.

Barry P. Hughes. Just a quick question on the same point, Dr. Petersons. Have you any very approximate comparative costs between mechanical and flame cleaning?

Nils Petersons. No I have not; I am not in the contracting business. I can say that the price in Sweden is about £1.50 per m^2, which I think is about the same as a good mechanical treatment would cost.

John M. Rolfe. I would like to put in two quick questions if I may. One to Dr. Petersons: does your experience of aggregate include aggregate such as limestone which reacts rather fiercely under flame treatment? Secondly to Mr. Gauld: I have encountered this type of treatment before. I have little in the way of reservations about epoxy bonds carrying transverse perpendicular stress but as I understand it epoxies are inclined to creep under sustained stress, the rate of creep being influenced by temperature change within ambient range. I am not talking about fire, what I mean is perhaps 20°C ambient here and 30°C ambient at home which could make a difference to the rate of creep. When the epoxy bond is acting in shear stress along an indefinite length it certainly worries me. Could we have some comments on that please?

Nils Petersons. Well the tests included limestone and there were no problems. One big problem we had was when the laitence layer was very thick, say 5-10 mm, but if the equipment is adjusted and you have the right moisture content in the concrete you can flame clean at a speed of about 1½ m/minute with a good result.

Allan A. Lilley. My query is going to be directed at Mr. Fairweather, but before doing so, I ought to explain that I am a highway engineer, some people say a Highwayman, and I know very little about floor slabs. I would like just briefly to go through the history of why we groove concrete roads at all. The major problem is skidding accidents. Skidding accidents happen mainly in the wet and the problem we have is to get good contact between a rubber tyre and the road surface. At slow speeds you can do this very easily with a good micro-texture, a sort of sandpaper like, gritty texture is ideal. By slow speed I would think of something of the order of 50 kilometers per hour. When a vehicle is travelling at a high speed a great deal of water has to be displaced to ensure contact between the tyre and the road. This is why we have a tread pattern on the tyre and, also, a 'tread pattern' on the road, in the case of bituminous surfaces, with stones projecting above the road, and in the case of concrete, with a grooved surface. The total object of grooving, as I see it, is to aid the displacement of surface water. I have been round a few factories and I have never seen forklift trucks driven at very high speeds, and I am a little concerned with the idea of grooving within the factory, grooves could worsen a situation as far as the floor is concerned. I certainly would not consider grooving any floor which was to be dry because of the possibility of the grooves filling with oil and detritus which could create a hazard. That, if you like, is the bad news. The good news is something which I only heard a few days ago which concerns some research being carried out by the Dunlop Tyre Company, which as far as I know has not yet been published, but which indicated that for a vehicle running on a concrete road surface, with transverse grooving, the fuel consumption compared with a vehicle running on any other type of road surface would be reduced by about 20 per cent. Perhaps Mr. Fairweather is wrong to promote grooving for the avoidance of skidding on floors, but possibly he would be better to pursue the idea for fuel economy.

Ernest S. Fairweather. Theoretically you are entirely right and when we started our work I agree with you we looked on transverse grooving as being the answer for wet weather at high speeds. You have raised two points, speed and wet and dry. Dealing with the question of wet or dry may I quote to you from collision data before and after grooving an old pavement, data from the Ministry of Transportation and Communication of Ontario, Canada, who did very detailed research. The result was that grooving on wet pavements reduced accidents by 59 per cent and grooving on dry pavements reduced accidents by 49 per cent. Therefore I cannot accept the theory that grooving is only effective in the wet. As to the question of speed, no we have done nothing theoretical about this, but you make me feel a little bit like the bumble bee which all scientists can prove cannot fly. We have grooved a ramp on which they had an average of two accidents a month, a slow speed ramp. They grooved it about two years ago, their speed of movement has not increased but they have never had an accident since. I do not think the accident reduction is due to the fact that it looked nicer when grooved. I can only assume it is due to the grooving, so again I do not quite accept the bumble bee theory. Finally I may say that one of our major fields for grooving at present is agriculture and particularly cow sheds. Cows do not move at a rate of knots but where a farmer for whom we have done grooving reckons previously to have lost a cow every six weeks due to slipping, in the ten months since he had grooving done he has not lost a cow.

Roger D. Browne. Two questions. First, I would repeat a question by a previous speaker from the floor who asked Dr. Hewlett's people about the effects of temperature on sustained creep of repaired beams. Secondly to Dr. Petersons: in the case of structural slabs do you not get a thermal shock effect on the concrete in the top of the slab, where you showed 180°C in the top few millimetres? If you take a limestone concrete, at 95°C you can get a strength reduction of 25 to 50 per cent.

Peter C. Hewlett. I think the original question from Mr. Rolfe concerned primarily bonded reinforcement and its response, as opposed to the response of an in-depth crack repair, but I will try and answer both queries from the same materials property. Organic materials do degrade with temperature. Above a sustained temperature of 55 to 66°C they will soften but probably still sustain about one fifth or one tenth of their normal ultimate bulk property. Usually these materials are being used in stress situations which probably represent a tenth to a hundredth of that ultimate property. The work that has been done with respect to resins subjected to a high temperature situation with very little cover, and that I think probably would be the case with externally bonded reinforcement, shows that below the heat distortion temperature, which is 55 to 60°C, creep at most working stress levels is insignificant or certainly tolerable within the normal working range. Above that temperature, and sustained, the situation would be very dubious. I must say, however, that there are alternative ways of fastening reinforcement so that one is not simply relying upon adhesive shear connection in order to maintain a composite contribution.

In the case of crack repair you have quite a different situation. The aspect ratio of a crack is very high in that it goes deep and yet its exposed profile is usually very narrow. Concrete also degrades when subject to fire and above an external temperature of, let us say, 500°C significant degradation starts and goes on progressively. Now we have been interested in this aspect for some little while and on a small scale it would appear that even subjecting small articles with just a narrow exposure to the resin and an in-depth use of the resin, one can sustain, certainly for an hour or more, ambient temperatures in excess of 600°C without significant degradation of the resin itself. Now that we have several beams previously in a state of failure, then repaired and then a stress/strain plot on the repaired article taken, it is part of this programme now to subject those beams to the typical BS 476 fire test and to see whether the load deflection profiles do in fact change. As matters stand, we do not know the extent of resin downgrading but we have reason to think that it probably will not be significant to any depth into the concrete.

Nils Petersons. I will be very short with the answer. We sawed out prism specimens and tested them for internal cracks. There were none. The flame passes over the surface very fast, surface spalling occurs and the temperature gradient occurs only in a very thin layer.

John D. Peacock. One quick question for Mr. Fairweather and a comment to follow it. Can he please tell us that we do not need the saving on petrol to pay for the tyre wear. I am pleased to know from the safety point of view that the road users do not suffer, but I am a little worried about nearby residents. Now I must declare an interest because I live in Gerrards Cross, not very far from the M40. I have to tell you that the residents of Gerrards Cross would be very happy, because they do not use that part of the road very often you understand, but they would be pleased for other people to wear their tyres out and fill these grooves up so that we do not have the terrible noise the road makes. It is nice to know perhaps that a major part of the grooving business now is where the cows are walking, making a more acceptable noise.

Ernest S. Fairweather. To the residents of Gerrards Cross may I strongly suggest that you read the Proceedings of the Institute of Civil Engineers, part 1, 1979, and read paper 8144 on Skid-resistant Road Surfacing and Tyre Noise by George Salt. I think the answer is that a few people who live an awful long way from that road, claim to suffer terrible noise in their large mansions. This is an argument that has been going on long enough and is one that I do not want to enter into. As far as the effect on tyre wear and tyre noise is concerned I can assure you that the grooves do not fill up due to rubber wear nor do they fill up for any other reason. The action of traffic forms a vacuum and clears the grooves so the grooves remain and safety remains. Lives are also important as well as people's comfort. The question of saving fuel is a matter which has been raised by Dunlop's recently. I know nothing about it, but I will be reading the paper when I return to London this weekend. I am not making the fuel economy claim, but I do know that so far, nobody has been able to prove excessive tyre wear due to grooving.

Hans Gesund. How are you finally repairing the balcony slabs, Dr. Fagerlund?

Goran Fagerlund. That is a problem we have to face now. The solution must depend on their state together with many other things. What we have suggested is that if they are deeply carbonated then we have to replace them by new concrete; similarly if they are severely damaged by frost. The problem is if there is sound concrete in the main part and deteriorated concrete only at the edges, with perhaps some points where there is too deep carbonation. The best repairing material we found was an ordinary cement mortar, 4 cm thick, with 8 per cent of entrained air. Of course the slab must be able to take that extra load. I think myself that the repair material should have a certain porosity, it should not be too dense because then you can get moisture collection beneath the impervious coating which could be dangerous for the frost resistance.

George Gauld. I should like to ask if your findings on the balconies have led to any radical changes in the construction of new works. Are you still going for cantilevered balconies? What changes, if any, have been brought about in new works?

Goran Fagerlund. Today we have better concrete standards in Sweden. We have to use air entrained concrete with a rather low water-cement ratio, below 0.6, for the actual types of structures. So if people follow the new regulations it will be okay; we hope they do.

George Barnbrook. I am interested in two points in Professor Hughes' paper. First of all, how did the original failure present itself in the actual contract that he was involved with? What sort of failure mechanism was occurring that brought about his investigation? The second point relates to the type of bonding and preparation of the units in the tests: was it similar to the preparation and treatment that occurred on the actual site?

Barry P. Hughes. Well the actual failure that occurred was in fact progressive cracking between the units, as I indicated. I think there were about three forklift trucks which were using the suspended floor and it had already 'failed' and temporary measures had been taken. It was simply that the operatives on the floor were becoming increasingly alarmed by the increasing vibration of the floor as the forklift trucks moved over it. It was after that stage, when it was no longer serviceable at all, that I became involved. As regards our reproduction in the laboratory, we were then more concerned with coming up with proposals for composite precast and in-situ floors in general, trying to define what minimum amounts and types of reinforcement, and perhaps different thicknesses of topping, that would be necessary. For this reason we were not attempting to reproduce exactly the conditions in the original failure. We were simply casting the concrete onto the sections of precast units without any special bonding agents between the precast plank units and the in-situ concrete or between the sections. We understand that there was none used in the original construction, but as with so many of these things, when it is a historical investigation of a failure, you cannot always be sure of the facts. For example, the in-situ topping that was actually used in the factory floor was not that as shown on the drawings; there are always these sorts of differences. So our investigation was initiated by the failure, but we were more concerned with floors in the future rather than reproducing exactly what had occurred on that particular factory floor.

George Barnbrook. My question on preparation was prompted by the fact that in one of the tests some grout had penetrated between units, which affected the results of cracking of the total beam section.

Barry P. Hughes. That is right, some slight seepage of grout underneath the abutting units occurred with all our castings in the laboratory and since this was fairly typical of what could happen on site we did not take special measures to prevent it. As far as the test results are concerned, then certainly in one of our specimens we obtained fracture not between the units but in the centre of a unit, which was a type of failure we did not anticipate. However, this was largely because of the slightly increased depth of the section at the joint.

Robert J. Savage. Regarding the state of the art in fatigue testing, fatigue science, people deal mostly with metals, which by comparison to concrete are very homogeneous materials. I was wondering if there was any attempt on Professor Hughes' part at Birmingham to follow the linear cumulative damage laws that Miner produced quite a long time ago. Also whether there was any attempt to establish some endurance fatigue limit? In metals, for example, limits are quoted such that if you can achieve 10^8 cycles at a given level of stress, it is called infinite life, you are below the fatigue limit. Is the concrete fatigue programme following similar lines to that established for metals?

Barry P. Hughes. There is not a fatigue limit as such for concrete although the achievement of a nominal number of cycles, e.g. 10^7 or 10^8, is adequate we think for many applications, including ours. There has been plenty of fatigue work done on metals, a considerable amount done on concrete in compression, and so on, but the main problem in this case is the breakdown of bond between the reinforcement and the concrete and not all that much work has been done on, say, standard reinforced concrete members, specifically on the bond, although there has been some. For the situation we are studying, with composite action, then as far as I am aware very, very little indeed has been done on this. So my answer is, yes, we are of course aware of the work done on the individual materials, but it is the combination here which we think is of particular interest and we are only at the stage now of being able to make some suggestions. For example, in broad terms for our particular application if the amount of distribution reinforcement for the static design is doubled then for our dynamic loading problem and from our preliminary tests we would suggest that this should be safe. To take this to closer limits, well, we simply have not done enough work to say anything further.

Paul Poitevin. Professor Hughes made a remark about slabs on the ground and thermal cracking. I have read your very important contributions on thick slabs and of the thermal stresses and I agree with you that in thin slabs also there is a problem of thermal cracking. However I am not sure that I have properly understood your remarks. Am I right in thinking that you said that for outdoor slabs the danger of thermal cracking was greater?

Barry P. Hughes. Yes. For outdoor slabs the early thermal effects are greater than for indoor slabs because the early thermal drop is both greater and sooner (i.e. the indoor slabs are protected from the extremes of solar gain by day and radiation losses by night). The point I was making earlier was that if the slab is designed to take care of the early thermal contraction in the normal way for outdoor slabs, then for indoor slabs, where the temperature drop and the early thermal contraction are going to be less (because your ambient conditions are more favourable) but your drying shrinkage strain which occurs over a long time is going to be much more severe, the net effect is very little different. Thus if you design indoor slabs as you would an outdoor slab you should satisfy either conditions in the same way. The timing of the cracking of course would be different and this has certain effects because the tensile strength of the concrete when the steel has got to crack it at new sections will be higher. I think that this is a relatively small effect, however, and the main thing is to reinforce it to crack satisfactorily for the outside situation in which case you should be more or less all right for the indoor situation also.

R.V. Iyer. I must say I was fascinated by the paper by Dr. van Gemert on the repair to this roof plate. I remember five or six years ago in Edinburgh I had occasion to inspect a similar case when I was working as a Building Control Officer. This was a very big reinforced concrete slab at first floor level. When I realised it had deflected about 8 inches I made a dash for the door. However my recommendation was to demolish the building and rebuild, which was also the consulting engineer's opinion. Two questions to Dr. van Gemert: first, who footed the bill and second, did you carry out a cost comparison? Why did you take the trouble to repair it? Might it not have been cheaper to demolish and rebuild?

Dionys A. van Gemert. First the second question. I think that the cost of this repair was nearly one third of the cost of the demolition and re-building of that plate because now we did not need the formwork, we used the old plate as a formwork. As to why it was allowed, it was a compromise between all the parties involved. It was agreed to make a study of the repair and if the results in the laboratory on model plates were good then the repairing procedure could be followed, so it was a compromise.

Khalafalla B. Musa. To my knowledge, and this is supported by tests carried out in the laboratory, although using curing temperatures as high as 57°C increases the strength at the early age of three days compared to normally cured concrete, at later ages, from 7 days onwards, the relative strength will decrease progressively. I should like to ask Dr. Bamforth whether he has carried out tests either in the laboratory or using core specimens to ensure that the strength, for example at three months or six months, is within the specification. Curing temperatures as high as 57°C have an adverse effect on strengths at later ages.

Philip B. Bamforth. We have in fact done some work looking at that specifically and found that in certain circumstances heating the concrete at a very early age will impair the long term strength development. The concrete will achieve its peak strength at about seven days and not increase at all thereafter. However, we have got varying results varying from compatible strength at 28 days, that is the early strength development was so fast that it achieved the 28 day strength relatively quickly and stayed thereafter, to 30 per cent reductions in strength as a result of this early heating in concrete. We did not know about it at the time when we cast this particular pour which was about five years ago. I think that if we were to have a similar problem facing us now we may have opted for an alternative solution by perhaps putting fly-ash or blast furnace slag into the concrete to cut down the heat which had developed rather than try to control the heat which was in there. From our initial studies it would appear that if you use either of these replacement materials the degradation to the strength at 28 days is not as bad as if you are using a neat Portland cement concrete. There are conflicting results at the moment, however, and I think some more work needs to be done before we can make any definite conclusion in that respect.

WORKSHOP SESSIONS

CHAIRMEN

Session 1: Cyril Hobbs

Managing Director
John Laing Research & Development Ltd., U.K.

Session 2: D. N. Trikha

Professor of Civil Engineering
The University of Roorkee, India

Session 3: Geoffrey C. Mays

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The University, Dundee, Scotland

Session 4: John M. Rolfe

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Session 2
*

Session 3

J. Maxwell
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Session 4

P. C. Hewlett
*

* Due to technical problems the recordings of Workshop Session 2 and the major part of Session 4 were of such poor quality that no transcription was possible.

WORKSHOP SESSION 1

Concrete and its Constituent Materials

Ravindra K. Dhir. I would like to start with a comment on your keynote address Mr. Hobbs. While it was gratifying to hear that you and others in industry are beginning to recognise that concrete no longer consists only of four conventional materials, but that it is now possible to see 5, 6 and 7 phase concrete, nevertheless I felt that perhaps you over-stated your case in stating that all seems to be well with conventional concrete. My own feeling, having worked for some years now with both conventional concrete and also non-conventional concrete, is that conventional concrete itself is not really well understood. I find that the design engineer tends to limit himself quite severely to what he finds in the text book. With contractors, I find that the feeling generally is that concrete is still a simple material, anyone can make concrete, there is nothing to it, so why worry if problems such as bleeding, plastic shrinkage cracking, etc., tend to manifest themselves from time to time. While I think that both in this country and elsewhere there is an obvious need to move away from the conventional types of concrete, and I think one should welcome the introduction of other materials to concrete, I feel that the emphasis on improving and understanding conventional concrete should not be forgotten in the process.

Alan A. Lilley. I believe that concrete is a simple material to be placed by simple people. I do not support the idea of making concrete so complex that we cannot handle it on site. That is not to say that we should not go on with further research on trying to utilise the material better.

Paul Poitevin. The idea raised by Dr. Dhir is very interesting. In France for ordinary slabs for industry, for warehouses, generally we have to order concrete from ready-mixed concrete suppliers. Ready mixed suppliers are unwilling to supply specially designed mixes, as for flexural strength specifications, the bulk of their production being used in general construction work. So where ready mixed concrete has to be used, it is difficult to obtain anything but conventional concrete.

Peter G.K. Knight. I would like to view this from a slightly different angle. Concrete is not necessarily a complicated material to produce but the producer must know what he is doing. Unfortunately there is an attitude in some quarters that any fool can do it. He can, but often with disastrous results.

Adequate training and supervision are basic necessities,all the more so when dealing with new materials. The nightmare of any promoter of a new material is the character who considers it a substitute for good concrete technique. The latter is almost invariably wrong. New materials or new techniques may improve a basic material but they are no substitute for good workmanship.

R. Colin Deacon. I would certainly like to agree wholeheartedly with that. I think the problem really lies with the attitudes of our industry. Some years ago in the Northern Region of the Concrete Society we organised a debate one evening and Mr. D.A. Stewart, known to many of us for his very strong views on this sort of problem, proposed that there should be a trade of Concretor. We had a couple of trades union people to oppose this idea and it was a damn good evening. It was a very good debate but at the end of it, it was completely clear that the industry just would not have it, neither the contractors on the one hand nor the unions on the other, so that is out right away. Our attitude to training is that we will not train operatives because the industry likes to have an unskilled man to handle concrete today, to dig a hole tomorrow, to lay a drain the day after. What worries me beyond all this is that I see a trend on sites for less and less supervision. Young engineers when they do come onto the site are spending nearly all their time on man management and cost control, they are not standing at the end of a concrete pump or under a concrete skip learning what concrete is all about and how to handle it. So our supervision is getting less skilled and we have an unskilled labour force and so our expertise in the material I believe is going down and down. I thought the gentleman from Rhodesia really showed how far down the road we had gone compared with what he was able to do in Rhodesia because of the situation that he has. He says he is in a backward country, but I think it is we who are backward.

Hugh Stewart. I have got to agree with Colin Deacon on this point since I have been experiencing this on site for some time now. I think the BRMCA method of approving ready mixed concrete producers means that the young site engineer has lost his responsibility for going to a concrete plant, checking the materials, etc. Concrete now comes on site as just another material; it is something which the engineer orders just as he would order a piece of steel reinforcement. When it comes on site he has no guidance as to what materials have been used or anything, and very briefly I think that there is a need for education, and for continuing education.

John M. Rolfe. I think I should reply to Mr. Deacon's comments. I thank him for his kindness, but I must say my philosophy on this is that if you cannot teach people to mix cement, coarse and fine aggregates and water and produce concrete, you are still less going to be able to teach them to combine those four materials and another fifth or sixth material and produce a better product. Until they know how to handle the basic materials, they are going to assume, exactly as was suggested, that the additional ingredients can be taken as miracle ingredients to relieve them of responsibility and not to improve the concrete. My view is that we cannot turn bad concrete into good concrete and good concrete is so good a material that it hardly needs anything else; not to say that we could not do with a bit more tensile strength and a bit less shrinkage and things like that, but on the whole good concrete is a really good material. I would also like to comment on this question of the BRMCA because our ready-mixed manufacturers use the BRMCA standard specification, which I understand has been criticised in this country as being a little bit dictatorial. I have found it so unsatisfactory that in my standard specification I state that ready-mixed concrete shall not be used except with the express approval of the engineer and under such conditions as he may specify, because I have found that attempts to produce a good concrete using ready-mixed concrete have been almost impossible; the technique I described in my paper when attempted with ready-mixed concrete was a total failure.

George Barnbrook. I am not with a contractor, Mr. Chairman, but just a point about the contractor and his problems. I think they have relied so much on ready-mixed concrete for use in construction for so many years that recent trials by some sub-contractors to go back and mix their own concrete have shown that in fact they have lost the necessary expertise. They do not understand materials and they do not understand the problems of the more difficult aggregates; grading, for example. In fact a recent job I was involved with was associated with problems of plastic cracking because of very poor aggregates. This contractor had lost all his knowledge of putting the materials together properly, and particularly his knowledge of what is good aggregate. This is a big problem.

William P. Liljestrom. I would just like to make a comment on quality control. One State in the U.S.A. has recognised the fact that a building inspector who obtained his licence, say, fifteen years ago may not be keeping himself abreast with advances in technology yet he still holds his licence. Now we have a law in the state of California that these men can no longer carry what we call a 'grandfather's' licence. Every two years they have to go back and be re-examined and tested on new techniques and materials. This has put a new responsibility on those people who can not now just sit back, but have to keep abreast with new technologies. However, that has only happened in one State in fifty-two so far.

R. Colin Deacon. Can I just put the other side of the ready-mixed concrete industry case or perhaps look to the future. Nils Petersons is not here, but he was saying to me the other day that in Sweden apparently, quality control in the ready-mixed concrete industry has advanced to such a stage of competence and confidence that the need for taking control cubes on site is no longer mandatory in their new codes which are coming out. Now one feels that this is a remarkable achievement; whether it is going to lead to problems or not I do not know but I think this is what we have got to aim at because ready-mixed concrete will not go away, we have got to build up its confidence and its competence. There are certainly great problems in achieving the sort of standards that we would all like but perhaps Sweden points the way and we may get there some time.

Chairman. The picture you all paint to me is quite foreign and quite different from the picture that I see from within Laings, for example, and I think my colleagues in firms like Wimpeys and Costains would equally agree with me on this one. When I joined Laings thirty odd years ago there was hardly anybody at all in the Laing organisation who even had the remotest knowledge about how to design concrete mixes or how to control them or what was important. There was not even a consciousness that the water-cement ratio had any effect. The total knowledge of concrete technology within the Laing group at that time was probably confined to about half a dozen people, and the problem on sites was that people did things without any kind of consciousness of what they were doing. You spent a lot of your time in those days trying to persuade men to use gauge boxes for volume batching, and even that was a struggle. Now if you look at Laings today, if you look at Wimpeys and all the other big contractors and I think this is probably true overseas too, there are certainly as many people on every site who fully understand concrete technology now as there were in the whole of the Laing group thirty years ago. So that the knowledge about concrete and concrete technology in the contracting side of the business is vastly greater and vastly more widespread right across the whole country than it was when we began. Against this you are all saying that somehow or another quality has gone down. I think what has happened is the opposite, I think your expectations have gone up. What has happened is that you are now looking for concrete of a quality you would not have dreamed of asking for thirty years ago. Certainly our performance has not gone up as fast as your expectations and I accept this and we all therefore want to see quality go up a lot more. However, I still say that certainly with all the bigger major contractors in U.K., and probably overseas too, on the average you get considerably better concrete nowadays than you used to get thirty years ago. The performance has gone up and not down, but because you are all the time increasing your expectations and stretching the material further it is still falling short of your wishes.

Now on the ready-mix concrete side of things, for example, to us one of the achievements that comes out of ready-mixed concrete is that you can now, with a good ready-mixed concrete company, specify a very sophisticated weigh-batched mix and most times get it done pretty well. It is weighed out and it is accurate, the water is right and the sand is right and the corrections for moisture content and what have you are right and you do get better concrete delivered to your site than you used to get with site mixing. By doing this we have removed the need to have operatives on site to do this, we have taken one of the operations away from the site situation and removed it from a position where it can be spoiled by bad workmanship. I think by doing a number of other things, by getting vibration and other techniques under control for example, we are making the site operation less sensitive to bad workmanship. So that we see that we are getting to a stage where although there was a need for a trade of concretor at one time, it is becoming less necessary now, not more necessary, because we are making it easier to do the job correctly without having to rely upon the skill of the craftsman on the spot. We are taking the control out of his hands and we are putting it where it ought to belong. So we see this problem turned entirely the other way round. I entirely accept Dr. Dhir's point that, of course, we do not know the answers to all the problems on concrete, there are a great many things to be done to get it right and there are many problems in conventional concrete still to be solved. However, the really big problems, and this is the point I was trying to make in my keynote speech, are just understanding the main parameters which affect the properties of concrete and making sure that they have been fully understood in the last thirty years and are now automatically applied. The improvements we shall get in future will not be the big steps we have made in the last thirty years, they will be a process of continual refinement. I think the next big step, if there is going to be one, will have to be some new breakthrough which will come from adding a fifth or a sixth or a seventh material to the four used to start with. There is a lot to be done with the existing four constituents certainly but you are going to have to bring something else in for the next big step. I only put these points forward as a view from the contractor's side, but I would like to hear whether I am right. It is not that our performance has gone down but that your understanding and expectations have gone up.

Alan A. Lilley. I am not interested in what the large contractors can do, because it is not they who build the majority of floors. It is the small firms that undertake the bulk of work, but too often lack expertise. I think there is a gigantic education problem regarding the small contractor and I do not believe there is much chance of controlling the small contractor until he is given sensible and readable specifications. I come back to a point that I made yesterday which is that we have a lot to do in improving specifications and a university is not a bad place to start this task. To the best of my knowledge nothing is done in universities to teach engineers that a specification must be realistic and usable. Certainly in my formal education this aspect of specification never came through. I think part of the universities' job has got to be to teach people to understand specifications, write them clearly and then apply them.

Ravindra K. Dhir. Could I, Mr. Chairman, have another go, partly at you and then partly at the C&CA and then mainly at the consulting engineers, and try to reassure you that the universities, and this is not simply because I am a member of a university, do try to stress the need for clarity in specifications. For my own part, I can certainly say with confidence that I try to indicate the need for a good look at specifications. I try to indicate why one has to start with the standard specifications which are available in C.P.110 and elsewhere and go on to form one's own judgement based on one's experience. Where one is horrified is that no sooner do students go to consulting engineers than somehow they are enclosed in their narrow shells surrounded by various books from which specifications are then taken, specifications which often are not fully understood and not properly written, certainly not clearly written. I have seen many bad specifications, I can give examples if you wish, but very rarely do I find that the consulting engineers train these young men so that they can develop independent thinking. I came across one problem recently in the course of my research on shrinkage of Scottish aggregate, and I called in a few consulting engineers and asked for their views as to how would they approach the shrinkage of concrete in general, not going into detail about the shrinkage contribution of the aggregate. I was horrified at how often they said to me that this can be dismissed as long as you have got a good concrete. So I asked them what is the definition of a good concrete: is it that it has a good workability, is it that it has good strength? Nobody could answer and that is just about where we stopped and I think that is generally where we tend to stop. That is why I made my opening remark to the effect that I do not think that conventional concrete is well understood. While people tend to think that it is a simple material, I take the view that it is a complex material. I once was faced with a problem in my own Ph.D. work on the development of a stress measuring technique using glass. I tried to remind my supervisor that while it looks a simple technique there are many complications with it and it is only honest to recognise both its simplicity and its limitations and complications. He did not take any notice of my advice but extolled only the good points of our technique and as a result people, as they used this instrument, found out that after all it was not as simple as it was claimed to be. This to me is the stage we are at with the conventional concrete. As you rightly said Mr. Hobbs I do not think it is the contractors' fault, the ones I have met certainly, both small and big, are doing good jobs and they are capable of doing much better jobs. The fault really lies with the designer. He does not give his specifications clearly and the reason he does not do it is because he does not know himself.

Christopher Coward. I would like to disagree with Dr. Dhir. I accept that possibly he has come across some bad consulting engineers but I do not think he should judge them all by the examples that he has come across. In our firm we do have standard specifications, but they are meant to be used as a guide to the specifier, not just as pieces of paper which go unconsidered into documents. We ask our engineers to look at the job that they are doing and define the specific requirements of each job and then write in additional clauses to the standard specifications. I think Dr. Dhir is being a bit unkind to say the least.

To move on, one of the points that Monsieur Poitevin raised on the flexural strength of concrete is interesting. In highway work the cylinder splitting test is used to specify strength. Could this not be enlarged and used in other branches of construction, particularly for floor slabs?

Alan A. Lilley. Heaven forbid!

Chairman. I would like to come in here because Mr. Lilley made a remark which is so common in our industry and I think it is time we asked ourselves why. He said that we are not interested in what Laings and Wimpeys and so on can do, you can do it all right we know that. Who we are interested in is all the little people who actually do the work but who cannot do it properly. Now I have to ask in what other industry do you choose people to do your work who you know before you begin cannot do it, and why do you choose them? You choose them for any reason other than the technical reason, in fact. You do not look to see whether a man is competent, you seem to choose him for quite different reasons. If you know that Laings and Wimpeys can do this work, and you know that these little fellows cannot, then surely this should be a signal that you choose people who are competent to do the job in the first place. Surely the only way you are going to get competence throughout the industry, and make the little man learn how to do things properly, is to let him discover that he will not be allowed to do work unless he knows how to do it properly. So long as he can go on doing it badly, and yet continue to get work, what incentive is he going to have to train his people, to get his technology right, or anything? So where we in our industry have gone wrong is that so often right across the board, not just in concrete but everywhere, the common complaint of our clients including the consulting engineers who are employers, is that often they employ people who they know cannot do the job when they employ them. Now what are they playing at? I think this is a question which consultants do need to ask themselves.

George Barnbrook. I think the problem is that clients often just do not understand the difference between good consultants and those consultants who do not fully understand concrete. It is fine for major consulting firms and the Wimpeys, Laings and McAlpines of the country. They have their concrete laboratories and their specialists so there are no problems. The difficulty is with the smaller contractors who are not in fact experienced contractors; they may be newspaper owners and only managers. It is just a money making business to them. They employ people to run jobs with sub-contract labour and the supervising expertise just is not there. That is the big problem, Mr. Hobbs. Your company is in a select group.

Chairman. But we have a whole host of professional organisations, consulting engineers and architects and so on, who act on behalf of the clients, that is,the ultimate building owners who would not claim to know anything about building technology. These consultants are there to help the client to get a proper building put up. Now they have a job to do and apparently they think their job is executed by employing a lot of incompetent people to do the work. I am asking where they fall down on their job?

Peter G.K. Knight. I think there is another factor here. Architects were mentioned by Mr. Hobbs but I understand that an architect is not required to serve any time on site during his training. If he has not been on site how can he know anything about the way concrete is handled and treated there. I know that on big jobs he may employ a consulting engineer but this may be for design only. Often, therefore, concrete is supervised by people who have no experience of it and in such cases your chances of getting good concrete appear poor.

Christopher Coward. I would just like to take up a point that Mr. Hobbs raised to the effect that he is employed by consulting engineers. In fact, he is not employed by consulting engineers, he is employed by clients and there is an important difference. However, I think there is a way round this problem we are discussing, and it is the way that a responsible consulting engineer gets round it, which is to advise his client only to go to contractors that are responsible, that are experienced, and that he knows can do the job, not to go to the cheapest contractors that are available. If the clients listen to that advice then it goes some way towards eliminating this problem. Unfortunately clients do not always listen to that advice and they put people on tender lists who they know are cheap. So I do not think it is just the fault of consulting engineers, I think the clients bear a big responsibility as well.

Chairman. We are speaking here rather in the context of the United Kingdom. We have people from many other countries here. Can they tell us whether they have this same problem in their country?

Paul Poitevin. In France very often specifications are written for the tender document, not by consultants but by administrative bodies which are content to copy the old ones. Each administration has its own traditions, there is the Railways, there is the Electricity Board, there is public roads, and there is no common corps of doctrine on concrete. We lack some authoritative state of the art publication such as the U.S. Bureau of Reclamation Concrete Manual for concrete dams. It would be easier to draw good specifications referring to a kind of Bible, or state of the art, periodically reviewed. We are trying actually to develop this kind of book, but the French Standard Institute is a meeting point between the industry, the administrations and the producers, and for concrete we have a very difficult problem because of the opposition between the ready-mixed concrete producers and the administrations and contractors so the standard is a compromise. The standards are written by people motivated by conflicting interests so many of their recommendations are deliberately vague. It is very difficult to draw good specifications in a compromise spirit. European agencies and international agencies like F.I.P. are trying to promote international recommendations and I think personally it will be a good thing to have a common language on concrete.

R.V. Iyer. I do not propose to speak on behalf of any overseas experience. It is just that I noticed that no one has said anything on behalf of the client. Let me make it clear that I do not propose to defend the client. As employers of contractors, consulting engineers and academics, the Local Authority Organisations are the largest users of concrete, being the largest builders in the country. May I point out that much that has been said today is very true and I think everbody has to take his share of the blame. If the design is correct, the specifications are good, and the concrete delivered at site is absolutely right, things still go sour because you have the small contractor who can mess things about. The small contractor naturally does not have proper supervisory personnel on the site and this is a major cause of why things do go sour. A client himself, generally, is uninformed, but now the tendency is for local authorities to employ engineers, so they have some measure of control on the handling of contracts. I think that unless we strengthen the engineering content of the client's organisations we are going to be in trouble. This is one aspect which does not seem to be appreciated by the engineering community at large. There is far too much competition between consulting engineers and local authority engineers, and consulting engineers are not happy to see local authorities employing their own engineers because this means loss of business to them. They do not realise, however, that if local authorities have a strong engineering content, their own profession is strengthened. You will get local authority engineers specifying that concreting jobs must be controlled by engineers, by consulting engineers, and not just by contractors who do not employ engineers. We in Local Government insist that contractors have properly qualified engineers on their staff to supervise, that jobs be designed by consultants, that architects employ consulting engineers; and this is why I maintain that there is plenty of support from the local authority engineers for the profession. This is also why I would blame the rest of the engineering profession. They do not put forward their image strongly enough; it is always the architect and the quantity surveyor who take the lead in developing any building project. The engineer comes in at a very late stage. It is high time that the engineering professions stand up and speak out and insist that the proper procedures be followed in developing a scheme.

Chairman. I am glad that you have made that point. My view of this is that the client is the poor innocent who is finding the money and buying the product of an industry which in this case is the construction industry. The construction industry, to me, starts with the architect and consulting engineer and goes to the contractor and all the sub-contractors and it is our duty as an industry to make it unnecessary for the client to have his own specialist people. If he is doing that it is because we are failing in our job. The industry as a whole should be able to give him what he wants, which is a jolly good building at an economic price, and he should be able to count upon that. If we are getting to a stage where, because of internal squabbles between the designer and the contractor or something, we are actually putting up buildings using people who we know within the industry are incompetent to do so, as a result of which we are giving the client a bad building, then the industry is failing the customer. It seems to me that this is a matter for the industry as a whole. We have to sort out our own procedures until such time that the client can say: 'I don't any longer need to have my own people to check what these fellows are doing. I can go to this industry, tell them what I want and be satisfied that I am going to get a good job'. Now until we get to that stage I think we are, as an industry, somehow not making things click.

R. Colin Deacon. I think in a throw-away line there you have just touched on something which I think is really important and that is our procedures: our procedures on site of having the consultant, the main contractor and a series of sub-contractors and the main problem is communications. Very frequently I come across a situation, and it is not always absent just because there is a big contractor or big consulting engineer, I know some small contractors who are much better in their performance than an individual site of a big contractor, for instance, but I very frequently come across a situation where there is a problem with concrete. The consulting engineer's representative's knowledge of concrete is poor, the contractor's representative's knowledge of concrete is poor, and the sub-contractor supplying the concrete probably has got more knowledge than either of them put together. The problem is that the consulting engineer cannot talk to the ready-mixed concrete supplier because he is a sub-contractor of the main contractor. Nobody gets together and they get in an awful tangle. This I think is a very real problem.

David Little. I feel I should make a point on behalf of the clients. I have listened this morning to the conversation which has ranged from having a go first at the operative, then at the contractor himself, who then kicks the consulting engineer and the consulting engineer blames the client. At the end of the day it is the client who finds himself with the problem and having to make a decision. I agree wholeheartedly with you. I think it is time the industry sorted itself out. As a client we are forever having to interpose between contractor and consulting engineer, virtually arbitrating on all the many problems you have. I would have thought it refreshing if we could sit down and write ourselves some good guidelines to work on.

Alan A. Lilley. Could I take this up on a slightly different tack. Dr. Dhir and his staff, I think everyone in this room would agree with me, have organised an excellent conference, but they have failed because they have failed to attract a single architect or, to the best of my knowledge, a single builder. I am left with the impression that architects and builders do not want to learn. Why are we not getting architects and builders more interested in the basic materials they are using?

Pritpal S. Mangat. It seems to have been a constant feature of the past few years that when people talk about concrete the battle between the contractor, the designer and architect and so on seems to continue. I have reached the conclusion that apart from a lot of research and development that we need in theoretical aspects of concrete research, as Dr. Dhir mentioned, I think that aspects of management and production of good concrete are perhaps equally important. I would, therefore, propose that the next conference which is held at Dundee should perhaps consider this aspect, dealing with the problems of concrete manufacture on site and the various underlying problems which arise because of the different people associated with it. Let us clarify once and for all who is to blame for bad concrete. I propose that the Concrete Society should promote some research into that.

Barry P. Hughes. If I could perhaps make one comment. I have listened with interest to the discussion so far. I must confess that I have more sympathy for the small contractor, and the inexperienced agent with the large contractor just starting up on a new job, because you half expect him not to know the answers and you can be sympathetic. I think it is much more exasperating when fellow engineers incorrectly write specifications, or indeed incorrectly implement specifications. If I can give just two examples: C.P.110 attempts to carry through the principle of designed mixes and standard mixes, but how often do I come across a consultant who says 'well, I specify the properties of the concrete that I require, but of course I am not going to let the contractor get away with a lower cement content, so I specify the cement content as well'. So we have got engineers mixing up the concepts. If we consider the new B.S. specification for concrete, it has in one part scrambled up statistical principles in an unfortunate way. C.P.110 is bad enough, but with just a slight modification it could be made to follow through logically the concept of the acceptance of five per cent defectives as far as the specified strength of the concrete is concerned. With the new code the better the control of the concrete the more the producer is being penalised! It is inappropriate to go into further details here (they are given elsewhere*). However, I do think that it is highly unfortunate when specifications, especially British Standard Specifications, include, as in this case, testing plan clauses which are at variance with their own basic specification clauses (in this case the acceptance of 5 per cent defectives).

Chairman. What we seem to be saying, if I understand it correctly, is that we have learned over the last thirty years quite a lot about what we ought to do with concrete so we now know the rules, if you like, but we are not yet very good at applying them. We have not yet found the mechanism which must exist between the designer, the consultant, the contractor and so on, to get the information passing up and down the line properly, to get the supervision right

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*Ref: Hughes, B.P. Limit State Theory for Reinforced Concrete Design. Pitman Publishing Ltd. Third edition, London 1979.

and to achieve the results that we in fact are now capable, in scientific terms, of achieving. We seem to be saying that we have got the science well ahead of the practice. Can I move on if that is the case and look at some of the new papers that we looked at earlier this week. For example, we were always limited in concrete technology in the past in that we wanted on the one hand easy workable concrete that could be placed and compacted properly, the only way to do that was to put more water or more cement in it and every time we did either of those two things or did them together, we somehow produced a harmful effect on the finished concrete. So everything that was beneficial to the green concrete tended to be damaging in one way or another to the hardened product afterwards; it either put the strength down or the shrinkage up or something. Now have we with superplasticizers begun to move in a direction where we can start separating these two stages and have changes in workability to eliminate placing problems without having to accept the corresponding harmful change in the finished concrete? What about fibre reinforcement?

Ravindra K. Dhir. I wonder if someone can put me right. In order to use these new materials, these new types of concrete, one obviously needs to know the performance of such materials properly so that one benefits and one does not lose from it. Where does the responsibility actually lie for this data to be made available? Is it the manufacturer of the new material, is it the C&CA, is it SRC, is it a university, who? That is what I would like to know? Perhaps someone can help me on this matter.

Peter G.K. Knight. May I complicate the issue slightly? It is often said that it is the task of the promoter of a new material to disseminate information on it. This is of course true so far as it concerns the information in his possession. However as any such material gains acceptance, others move into the field and obtain additional information from their own work. Such work may duplicate that already done or alternatively for one reason or another may not be published. An example of the second case, which always causes me concern, is the unpublished university thesis.

Chairman. Well I can perhaps make a comment from the point of view of SRC because I have in the past served on SRC committees. For the benefit of our foreign friends the Science Research Council is a British organisation that provides funds for university research, so it is a rather special kind of funding body for research. Their attitude has changed quite a lot over the years and whilst they still regard it as their job to make sure that the research in the universities is of the fundamental character, getting to know the information without having to have a commercial pay off, they do nevertheless, in stimulating university research, ask that the university people should now pay attention from the beginning of a project to how and where it will be applied ultimately and who the users would be and to give information to SRC about what efforts they will make later on to propagate the results and get them applied in some sort of way. So the SRC tries to do two things: they try to say this is not commercial research, that is for industry to finance, we are interested in the underlying fundamental knowledge, but please do not just collect it and put it on the university shelves, tell us, as you get the money for doing it, what you are going to do to make people aware afterwards that you have done it so that they can pick it up and apply it. So SRC in that sense has, in my experience, changed a lot in the last fifteen years, it has become much more aware of the need for application of research findings.

William P. Liljestrom. In the U.S.A. we are trying to get the academic world to stimulate new products and new thinking. The educational institutions, especially the graduate schools, have encouraged researchers, maybe from the standpoint of their own financial benefit as well as that of the university, such that a graduate student or a professor would bring a piece of research to the point of having a patent issued, then the university like to have industry take this over, with the university collecting royalties from any money that is obtained from these new products. They then reward the graduate student or professor with part of that royalty, which is extra income to him. I think that they have found that, maybe because of the financial benefit to the researcher, this has stimulated patents and also gets money coming back to the university.

Chairman. I would like some comments here from our British academic friends because I think you, Mr. Liljestrom, are better at this in your country than we are. How does it work in South Africa or in Rhodesia? Do your university professors act as consultants to industry, being paid for this and getting royalties on any applications of research that is done?

John M. Rolfe. At my university we have what we call an industrial liaison committee, the purpose of which is to bring home to industry that the university regards itself as part of the community and is available for doing research work to fulfil the needs of industry. We are a very new establishment, the faculty of engineering is only six years old and is only just starting to produce graduates now, and most of the staff are still fully occupied getting the new faculty going instead of doing research, but I think we have a much closer relationship with industry than appears to be the case in the U.K. Our system is modelled on South Africa. For example, in my case the university only requires me to work four days a week so that I can have a fifth day available for consultancy work. To my mind I think it is a very effective way of arranging things.

Turning back to the topic of concrete materials, I did ask a question at the practical session seeking information about the effect of cements on superplasticizers, something which has been a problem in southern Africa. I think it might be of interest to the meeting to know that I have, in private discussions at this conference, to some extent found the answer to that question, which appears to fall into two parts. First of all southern African cements, although technically complying with the British Standard specification for Portland cement, are, in fact, much more alkaline than cements in the U.K. Apparently the alkalinity has an effect on the action of the superplasticizer. Secondly, southern African cements are much more finely ground than are cements in the U.K. and although they do not quite comply with the B.S. specification for rapid hardening cement, they come very close to it and I believe the experience here is that superplasticizers work less well with rapid hardening cements and hardly work at all with the extremely rapid hardening cements that are now being developed. So in fact the cement does have an effect on good plasticizer action.

Chairman. I would be interested to hear some comment from consulting engineers about their attitude towards allowing some of these new materials to be used on their sites. I am thinking of their long-term responsibility to things like durability and creep and so on. What would your attitude be?

Hugh Stewart. I think at this stage I could not recommend their use because I would not be happy recommending them. We have a long-term responsibility and although I would like to see them used, I think they could be of great benefit, how can I recommend their use to a client with the risk that sometime in the future they may cause a failure? Will anyone give me a guarantee that I will be covered in the event of a building suffering distress?

Chairman. Going back to Dr. Dhir's point which he made earlier, from whom would you look for information to reassure you? Who would you trust enough? Would you want this to come from the manufacturer or an institution such as the BRE or a university, who would you count upon for your evidence?

Hugh Stewart. Probably an institution like the BRE or universities, definitely not from the manufacturer since he has too much vested interest in it and would tell me anything. As you say, one person comes up with something such as a superplasticizer and suddenly there are a dozen manufacturers supplying competing superplasticizers. At the end of the day I am responsible to a client and it is no use going along to him if there is a failure and saying that I am very sorry, it was a new material, it was a good idea but it did not work. That does not help the client who is having trouble with his building.

Chairman. You asked earlier whether someone would give you a guarantee. Normally the place you would look to for your guarantee would be the manufacturer or the supplier of the material, but you were saying nevertheless that you would not accept his information.

Hugh Stewart. That is true.

Chairman. Are you saying that you would still accept no information even if you had a guarantee? Or are you wanting information and a guarantee?

Hugh Stewart. I do not think I could accept his guarantee. A guarantee is only useful in the light of that supplier. If the material fails he is not going to be around to give you much guarantee, so there is no point in having the guarantee.

Chairman. The guarantee is not enough?

Hugh Stewart. No, it is not. I would need independent advice and information which will allow myself and others to assess the material, especially the long-term durability of the material and any long-term consequences of its use.

Ravindra K. Dhir. I think this really brings me back to my criticism of the consulting engineer. After all what is the consulting engineer trying to do? Is he trying to make his life as easy as he possibly can and thereby always play safe, or is he trying to present himself with a challenge and thereby make it his business to seek further information, further data, which perhaps is already available and moreover in fact to try to set up his own small investigations, so that on large projects initially, and then medium size projects thereafter, and then small projects at the end, these new materials can be utilized. As I said earlier, I happen to be working on Scottish aggregates and when I came on this scene I was horrified at how little the consulting engineers knew about the Scottish aggregate and how happy they were to continue to play safe and simply say the aggregate shrinkage should not be greater than x, y or z. I think the responsibility to some extent does lie on the shoulders of the consulting engineers to make it their business to get the information they need and then design the structure around that information.

Christopher Coward. Regarding the use of superplasticizers I would take issue with Dr. Dhir again that we do not play safe without good reason. I understand that superplasticizers can work extremely well in the right conditions, but they have to be very carefully controlled. There are two aspects about their use about which I think consulting engineers are very rightly wary. The first aspect being that the initial concrete must be properly designed and carefully controlled and, with the increasing use of ready-mixed concretes, this is becoming much more difficult. Secondly there is the fact that the superplasticizer must be added at the point of discharge. These are two problems which frankly I do not think the ready-mixed concrete industry have overcome. In fact I understand that a recent demonstration at the C&CA, where they attempted to show the benefits of superplasticizing, went horribly wrong for this very reason. So I do not think we are being overly conservative; we are willing to be convinced, we are using superplasticizers for special applications in many different ways and attempting to build up some experience of this, but we are certainly not going to go overboard tomorrow and start specifying it as the magic ingredient.

Martin E. Hodgson. The question was raised as to who is responsible for new materials. One of the most obvious features of the construction industry is its ability to pass the buck; the second most obvious feature is that it cannot even make plain concrete right, consistently. What can it do with new materials? Nobody seems to want to get together and talk together. Somebody says the suppliers of the material must be held responsible for the material. The supplier of material employs chemists as a rule, not civil engineers, not architects, not even contractors. He employs people to find out all they can about the material from a chemical viewpoint and extends that as far as he is able, to preserve confidence down the line. The industry, which includes the manufacturer, should surely be responsible for evaluating new materials and finding out what they can do and what they cannot do and getting decent specifications and decent control when these new materials are used on site.

Pritpal S. Mangat. With reference to concretes with fifth and sixth ingredients, I would like to place them in two broad categories. The first being materials like concrete with superplasticizers or lightweight aggregate concrete which are new materials but have been used for a number of years, if not in this country, then certainly abroad. The claim that enough data on durability is not available is not really valid in such situations and I think the real reason why these materials are not used here is because of the conservatism of the people who are in a position to use them. The second category of materials would be the newer sixth ingredient materials like fibre reinforced concrete or polymer impregnated concrete and so on. All these materials I suppose one could say are in a state of development and therefore I think the industry overall has to bear with the researcher and the people who are trying to promote these materials, and not to be too sceptical about any applications or any ideas which people come up with. They should generally encourage research into these fields and keep in mind that any future application may not be the standard sort of concrete applications which are made today. We might have to modify things, we might have to look afresh at things. So summarizing again, two categories of new materials, ones which are not new in the real sense and they can be used, enough information is available if industry is prepared to look for it, and a second category of materials which are in the development stage and from which we should not expect miracles, but we should be prepared to look around and investigate.

John M. Rolfe. Mr. Chairman, at the risk of saying too much I would like to add a comment about what my friend here said about requiring guarantees and putting forward what I think is possibly an American solution, which I have known to work, where you do not depend on the company which is going to go bankrupt with an unsatisfactory material to guarantee you. You require an independent guarantee from an insurance company. The supplier's problem is then to convince the insurance company that they can provide a guarantee at a reasonable premium, the premium is built into the cost of the supply of the material which permits economic assessment of its merits. You tell the client that you are building in this premium to guarantee him against any comebacks. If all the pounds and pence add up right at the end you can go ahead and use it with an independent guarantee, even if the supplier does go bankrupt and the job goes wrong.

Chairman. Does that not put the onus of making a technical assessment for this product onto the insurance company and are they capable of making it?

John M. Rolfe. Certainly it leaves it in the hands of the insurance company and the manufacturer. He has got to have his very best salesman available for that purpose.

Martin E. Hodgson. I think that that is the normal solution, in fact. Most manufacturers have product insurance, some have blanket insurance across the whole range of products. Even if the company goes bankrupt the insurance company certainly does not and thus one can claim back on the insurance company, if the product is at fault.

John M. Rolfe. On the other hand if the insurance company will not play then you do not buy the product.

Chairman. Well you see there is a rather parallel situation to this one between building societies and the National Housebuilders Registration Council in this country, where, to a degree, the building societies have begun to be a kind of technical arbiter. Many people in the industry now object that the result of this is that the building societies are putting constraints onto the way houses are built, they only want the things that they have seen done for the last fifty years and they are sure and certain about. They should not have this degree of power. Now would your suggestion perhaps not create that situation again in other fields, so that the insurance company would begin to be very conservative and very restraining on anything new because they would want to be very certain of their insurance commitment?

John M. Rolfe. Mr. Chairman I would say that it would depend a lot on the insurance company and, of course, the premium the manufacturer is prepared to pay to get his product established.

Chairman. I confess that I am personally very suspicious of financial solutions like this. They all sound very good, but my experience is that technology never seems to get advanced when you seek advice as to which direction you should go from somebody who is financing you. The accountant often has too much influence on modern society and not too little.

John M. Rolfe. I only put this forward as a way in which a new product can become established, without the consulting engineer sticking his neck out too far.

Paul Poitevin. In France, for example, we have a system for admixtures, for which there is no national standard, but in the tender documents it is generally specified that the admixture must be certified and there is a certification corps which is half professional and half state. In practice there are perhaps 50 admixtures which are certified and it costs the manufacturer, in one year, 12,000 francs to have a product certified. This system eliminates the hazards. It does not give reliability but a reasonable assurance that the product is not bad.

Chairman. Yes, your Agrément system is much more developed in that sense than our British one. There would be Agrément certificates which give this kind of back-up from an independent organisation.

Christopher Coward. Again on the question of consultants being conservative, personally from my experience that is not one of the major problems. For many years on large sites with batching facilities, we have employed cement replacement using products like PFA, either direct from the CEGB or as a graded product, and also ground granulated blastfurnace slag such as Cemsave, with a lot of success. Concrete made with these constituents has much better fresh properties than ordinary Portland cement concrete and can in fact be cheaper, or at least no more expensive. However, we had a problem recently where one of our large sites changed from site batching to ready-mixed concrete as the site was running down and we were obliged to stop using these cement replacements and go over to sulphate-resisting cements, at increased cost, because the ready-mixed concrete suppliers were not prepared to put in another silo to contain these materials. I think the ready-mixed concrete industry in this country is very conservative. I think they should be educated to use these materials because they are selling most of the concrete.

Hugh Stewart. I think we may finally have found the one person we can all hammer.

Chairman. I think it is fair to say that the consulting engineer has to be a bit conservative because in many cases he is not gambling with his own money but with his client's. He has somebody else to think about as well as himself; it is not just his own safety.

Hugh Stewart. I know Dr. Dhir here has said that we are conservative; I do not think he has a great opinion of consulting engineers at this present stage. Mr. Rolfe touched on this when he was talking about low technology and I think really low technology affects us all whether we are contractors or consultants at the end of the day, because we each have a degree of low technology on all our sites. This is the point which I think worries consultants: that there is this degree of low technology. If we use very modern methods, such as superplasticizers etc., then there is a risk of things going wrong, as happened at C&CA when they were doing the demonstration on superplasticizers. Now what insurance company is going to give you a back-up to something where there is such an element of operator error. If it goes wrong, it is not going to be the fault of the material, it is going to be the fault of the man who made it, so we cannot accept this. I was very interested to hear Mr. Rolfe, I know he was not talking in the context of materials but he was speaking about using ordinary concrete and reducing his cement and his water content, having it almost, as I think he said, at negative slump concrete. I think you may be amused to hear this, because I am based in Northern Ireland at present, and we once said the same thing to a contractor, he ended up putting sand and stone into the trench and hoping it would harden.

WORKSHOP SESSION 3

Construction Techniques

John Maxwell. I would like to clarify what the reason was for the recommendation on the use of dowel bars, was it mainly economy?

George Barnbrook. I think I could give our comments on this question. We at the Cement and Concrete Association certainly feel that dowel bars and tie bars work very efficiently. They have been used successfully in our standard method of design for ground slabs for at least six or seven years now and we see no reason why any other system should replace them. Looking at keyed joints, there are two problems which occur. First of all during construction, the problem for the contractor is in forming a good key in the first slab, then when the second slab is placed, very good construction is needed, otherwise if concrete is not fully compacted into the first key, you have a very inefficient joint for load transfer. The second point of course is on the design side. With concrete shrinkage, the chances are that the slab will contract and pull out of the key; in other words the joint could open to such an extent that the sloping faces of the key could start to lose contact and fairly quickly lose some efficiency of load transfer. On the basis of efficiency the dowel bar, or tie bar, is certainly much simpler to install and more efficient and reliable. That is the basis of our recommendation in that respect.

Alan A. Lilley. For some thirty years we have been using dowel bars in road and airfield pavements. They are considered to be the most efficient load transfer device and if the dowels are set in properly the joints move freely. The tongue and groove type of joint, as Mr. Barnbrook has said, has a very low level of efficiency in load transfer. From memory, measurements have put the efficiency of a tongue and groove joint something like 40 per cent, whereas the dowelled joint has an efficiency of 90 to 95 per cent, so the quality of the two joints is not comparable. I would also have doubts about the forming of good tongued and grooved joints.

Chairman. I certainly take the point to the extent that I think we have a lot to learn in ground floor construction from highways experience. As you say the highway area has developed first and I think we have a lot to learn from what is being developed in that field.

George Barnbrook. There is a question of cost of course. I would expect that the keyed joint is cheaper to form than the dowelled bar joint and perhaps many consultants or detailers would prefer it just on that basis.

Alan A. Lilley. Yes I agree. May I come back on another point which may partly explain the difference between road and floor practice. Taking the example of the French, in their road works they tend to have very close joint spacings and therefore there will be little joint movement which would help the tongued and grooved joint. This argument could be extended one stage further; a floor, in an indoor atmosphere suffers less movement than a road and therefore the drop off in efficiency which I was quoting may be an exaggeration as far as floor slabs are concerned.

John Maxwell. Is there any evidence about added difficulties with compaction in the joint interface areas due to the dowel bars, thus creating weaknesses?

Chairman. I do not know if anybody has any evidence to present on that but I would certainly feel that the quantities of reinforcement present as dowel bars is so minimal that if we can get over the problems of compaction with normal quantities of reinforcement we can certainly do the same at dowelled joints.

Alan A. Lilley. The problem with dowelled joints is to keep the bars parallel to each other, the centre line of the slab and the slab surface. A lot of time and effort has been expended on establishing how to keep dowel bars in position during construction, and several practical methods have been developed.

Frank R. Benson. I would like to ask Mr. Barnbrook whether we need have either. I should think it depends upon the amount of load transference, but could one not, in fact, rely on normal shear capacity if one takes the normal precautions of roughly scabbling the first surface or using expanded metal formwork.

George Barnbrook. The problem really is that with any of these ground slabs, even if you are considering slabs with bays only five metres wide, and certainly inside a building, there is going to be drying shrinkage, causing opening of joints, almost fully taking place within twelve months. For this reason any attempt to scabble, unless you are fully tying across the joint with adequate reinforcement, is almost a waste of time in my opinion because the shrinkage will open up that joint and much of the aggregate interlock you have attempted to obtain will be lost.

Frank R. Benson. It depends on the amount of shrinkage and the depth of scabbling surely?

George Barnbrook. Yes, and the width of the bay, of course.

Frank R. Benson. Mind you we do normally carry the reinforcement through.

George Barnbrook. That is a different concept altogether. Ground slab detailing and design is an empirical system and the system we recommend is certainly simpler to help the contractor to get things right on site. We have seen the method work now for six or seven years without any problems.

Ross W. Hayes. Mr. Deacon mentioned the possibility of continuous reinforcement to ground slabs and the possibility of further research being done on it. I wonder if either he or his colleagues are able to give some indication of the increased amount of reinforcement that might be required to enable us not to worry too much about the length of the pour in this case.

George Barnbrook. We were discussing that very point yesterday and the percentage I seem to remember was a change from 0.15 or 0.2 per cent for present design up to something like 0.25 or 0.3 per cent to give continuous reinforcement. Not exactly a massive increase in reinforcement in ground slabs to reduce the necessity for a considerable number of joints.

R. Colin Deacon. I was talking to Professor Hughes about this yesterday, questioning whether his theories can be applied to industrial floors. I did a check calculation and the figure which came out on that was of the order of 0.25 per cent which is about half, or perhaps a little less, than the sort of figures that come out for traditional continuously reinforced concrete pavement. I am not quite sure why there is an anomaly here or whether it is because there is a different concept from roads to floors and whether this concept is valid. We have not tested this yet and this is what I would like to do, because I think if we can cut out some joints with certainty, this would help all round. We certainly do not see our being able to use expansive cements yet and the use of reinforcement seems to be the only way in which to do it. However, a little bit more work and effort is needed yet.

Alan A. Lilley. If I can again refer to concrete roads, with continuously reinforced concrete slabs we get a close spacing of very small cracks, small enough not to be harmful. I do not know whether in a particular floor situation if these cracks would be accepted or rejected, I think that it would depend very much on what the floor was used for. In roads they are so fine that salt solutions are not thought to penetrate down to the level of the steel and therefore are not harmful. Presumably something similar could be argued in the case of floors. Unlike roads, floors will not suffer frequent thermal movements, they only suffer basically one movement, which is drying shrinkage, and that may make life easier. The other thought going through my mind is to abandon steel altogether and have a very rough subgrade so that we get a multiplicity of cracking because of restraint, cracks which might be too small to be of significance. There is also the possibility of using fibres. I think we still have to hold these options open.

George Barnbrook. I think Professor Hughes in the same conversation yesterday made an important point, and one which would tend to modify something Mr. Lilley has just said; that for floors inside buildings we tend to ignore early thermal stresses. Professor Hughes' point was that these stresses occur but that they do not quite come up to the level which would cause cracking. However, the early stress is there all the same and then drying shrinkage adds to this to cause movement and further strain in the slab.

R. Colin Deacon. If I can just come in on that one because I do not think there have been a lot of checks made on the thermal changes in slabs. I remember when Wimpeys were building the very big slab that features in our film and in our publications, there is about 28 acres of it, we did instrument one particular section of it with thermocouples as well as strain guages. I cannot remember exactly now, but I am pretty sure that the temperature changes in that slab were very, very small. This section was about 140 feet long, sorry to be Imperial, and it had sawn joints every 15 feet or thereabouts, and I do recall over a period of something like six months from the winter through the summer back to the autumn there was hardly any noticeable movement at all on any of the sawn joints in terms of opening. At the end of the bay there was a free contraction joint and at the end of that six months period that joint had opened up by approximately one tenth of an inch. I do not at the moment have any great evidence to suggest that thermal movements in floor slabs under cover are terribly significant. I was recently brought in to look at a floor which was 72 metres square, designed with an expansion joint in both directions to cut the floor into four square panels of 36 metres. The central expansion joint had been designed to be 12 mm wide and the client was very perturbed that over a period of about two years this joint was now 50 mm wide. Now even adopting a fairly high value for drying shrinkage this was ridiculous, so there had to be something else at work. I discovered that this slab had been constructed in the height of the summer of 1976, when we had very high temperatures. Now although the ambient temperature in this factory was 70°F, it is pretty clear to me that the actual temperature of the slab itself in contact with the ground is a good deal less than that and I convinced myself after discussions with my colleagues, that in fact it was very likely that there had been something like a 20°C drop in temperature from the time of casting to the current time. That sort of temperature drop added on to a reasonable figure of drying shrinkage will account for something like 30 mm of contraction. Apart from the movement of the expansion joint there was no other cracking in the slab anywhere else, so there was colossal contraction which had taken place against base friction over a period of two years; I remain unconvinced that early temperature is a great problem in a slab if it is cast under a roof.

George Barnbrook. I was talking to Dr. Dhir only a quarter of an hour ago when he was thinking of suggestions for future work for himself to do and perhaps we could suggest this particular area as a possibility.

R.V. Iyer. Could I ask some of the gentlemen present here what their views are on casting a ground slab which is designed as a raft foundation 600 mm thick. The temperature gradients in this naturally will be quite excessive, much more than in a thin slab. Is there any danger of cracks within the depth of the concrete, and what sort of effect would that have on the performance of this foundation? The foundation is heavily reinforced top and bottom.

Chairman. I think the generally held view is that provided those cracks do not reach the surface and open up a possible path for corrosion, the problem is not so intense.

Anon. I am sorry, I am not very clear what the question is. Is the steel reinforcement cracking steel or thermal steel or structural steel?

George Barnbrook. I think the steel in the concrete would not know whether it is thermal steel or any other sort of steel.

Anon. I accept that point, but my reaction there is that we are thinking about plastic cracking, etc., which always seems to come to a halt with the very light fabrics which we use unless there is a very aggressive environment underneath. I would think that with that kind of depth you would be fairly safe.

George Barnbrook. I would have thought that with a 600 mm slab there would have been little problem providing there was a reasonable percentage of reinforcement top and bottom. In the general case if you have 0.15 per cent or so this would control cracking in that thickness of slab unless there are extreme temperature problems. However, I cannot see that happening with that thickness.

R. Colin Deacon. I think in this case I would tend to be a little cautious. I think this is where the application of Professor Hughes' approach probably is valid because this is a substantial thickness and you have then got to consider perhaps whether you have got a jointed structure or whether it is going to be a fully restrained structure. If you use Professor Hughes' approach, as in BS 5337, if it is a fully restrained condition this would be option 'one' of that code or standard, and you will find that you will need, depending upon crack widths that you permit yourself, a fairly substantial amount of reinforcement for a continuous structure. So I think with a 600 mm thickness, heat of hydration effects, quite apart from any ambient effects, may become significant. What I am really saying is that the average industrial floor that we deal with is usually less than about 250 mm thick and I do not think heat of hydration effects are very significant.

Chairman. What always intrigues me is that Professor Hughes' work, as you say, is included in the water retaining code. Is the result of that work in fact included in any of your recommendations for design of ground slabs as such? It always seems to me that there is a place for it other than in water retaining structures.

R. Colin Deacon. We have not included it in our recommendations for the reason which I have suggested, that the sort of slabs that we are dealing with are pretty thin and non-structural anyway. We do not have any evidence as yet of heat of hydration effects being terribly significant. In fact one of our colleagues did a theoretical run through based upon the heat characteristics of a number of cements to predict likely heat of hydration temperatures in various slabs. Even without protection, within the thinner dimensions that we are talking about, they are significantly smaller than the blanket figures given in that code.

George Barnbrook. I think there is an important consideration in this case and that is the one of joints. If we are talking about industrial floors there is a very great difference to water-retaining structures and reservoir floors and the like. In an industrial floor it is important to try and cut out joints. I think our philosophy so far has been that if you can make the structure of the ground slab simple, and have joints so that the slab will move, then you are going to cut out the random cracking which clients may object to. However, joints in a floor are always points of difficulty with trafficking by forklift trucks and the like. If we can move towards some method to reduce the number of joints this is all to the good and I would like to ask Mr. Benson perhaps to comment on the possibility of using post-tensioning as in flat slabs. I think this has been used on one or two occasions and this is one way to eliminate joints altogether in slabs.

Frank R. Benson. Well, it is not possible to make post-tensioning economic in suspended slabs, where the advantages are so much more obvious, so we have not really thought of post-tensioning ground slabs at all.

George Barnrbook. I think Mr. Bobrowski has designed two or three slabs on the ground now with post-tensioning, in warehouse situations, and the level of prestress is very low. Compared with normal reinforcement it may not be a lot more costly and elimination of shrinkage problems and joints is easily possible.

Frank R. Benson. Yes obviously it has that advantage.

George Barnbrook. Is it simply a question of economics?

Frank R. Benson. It is a question of economics, but it is also a question of ensuring that the prestress is there, when you have got subgrade friction. I should have thought what was a major problem was how much do you prestress it in order to ensure the residual prestress which is required to do the job, a highly complex design problem.

Anon. Again thinking out loud, I suspect the only way of putting the prestress in is the way our American friends were suggesting, and that was using the expansive cements.

Chairman. I certainly take Mr. Benson's point regarding the economics of the situation. We do not seem to be able to get the economics right for prestressed suspended slabs but certainly with low levels of prestress in ground floor slabs I am sure there must be opportunities there. With prestressed slabs I suspect that sooner or later somebody would want to put normal reinforcement in as well and I suspect the economics would go by the board. There always is a tendency to want to put normal unstressed reinforcement in slabs; I suspect that somebody would invent a reason why it should still be there.

Frank R. Benson. Well the whole of the design of ground floor slabs is really empirical. It is introducing a technical factor to ask how much prestress you actually put in the slab, and I should think it is going to be arbitrary again if they are ever developed. Mr. Barnbrook suggests 400 lb/in^2, but on what basis?

George Barnbrook. It is a figure I have heard that has been used.

Frank R. Benson. Yes, but there are the losses you see. If one could compute the losses, it would still have to be largely empirical. You apply so much force per tendon to achieve a certain amount finally and this has to be proved to be satisfactory. The C&CA have done their work on reinforced slabs, the same sort of work has to be done for prestressed slabs.

Chairman. Of course there would be a limit on bay length; because of prestress losses along the length of the bay, you would need to have joints at intervals.

Frank R. Benson. I can see situations arising where you have a very high level of prestress at the edges and perhaps lose it when you are half way along, without knowing you are losing it.

R.V. Iyer. The practice of prestressing slabs also introduces a new element of danger when you put in machine foundations or cut trenches or put in cables or conduits in the floor, which has often happened in my experience of ground slabs. With industrial or factory floor slabs, it has always been the case that once you have finished the job and handed it over to the client, he starts messing with the floor.

George Barnbrook. I think you are quite right. I think factory floors or power station floors are the problem areas. Perhaps with a warehouse situation this is not so much of a problem, but certainly in the industrial situation it could be quite a problem.

R. Colin Deacon. This is why I personally do not like the practice, which is quite common, of using the slab itself to tie the legs of a portal frame together. I think in this case it could be disastrous.

R.V. Iyer. I remember the case of a garage floor where the slab was used as a tie between the legs of the portal. The department which owned the garage cut a trench through the centre for a new pit which they wanted for cleaning vehicles.

George Barnbrook. I think the technique of post-tensioning slabs is a fairly novel one for most people in this country. I have been very lucky to see one slab during part of its construction - the placing of the unbonded tendons in the slab and concreting - in a job in Toronto and I can say that the specialist sub-contractor putting in the tendons made it look so easy I am amazed that it is not used more often.

Chairman. I think it must be a natural unwillingness on the part of contractors to change well established techniques. We have been building reinforced concrete flat slabs, reinforced concrete solid slabs for some years now. I suppose the contractors feel that they have the expertise in this matter and to try something new must always be an economic risk to them. I am sure that if only they could be persuaded to get the ball rolling then things could perhaps change.

Peter G.K. Knight. Mr. Chairman, the point mentioned about service trenches and the like will be even more serious in a prestressed slab. The case of power station basement slabs, where there is an absolute maze of service ducts and trenches has already been pointed out.

Chairman. Yes, this would be okay provided that one knew about it at the design stage. The current recommendations are that the majority of prestressing tendons pass over the columns lines and therefore the actual distribution of the tendons within the main part of the suspended slab itself is in fact fairly widely spaced. Provision could be made for openings and ducts, etc. at the design stage, but I accept that subsequent changes could obviously present problems.

Frank R. Benson. I could talk about the point which you raised about unwillingness to change, Mr. Chairman, for a considerable length of time. The fault lies not with the contractor; the fault certainly lies in the design aspects and accepted methods of design in my opinion. I have been trying to employ my company in post-tensioned lift slabs since 1963. It is going at a tremendous rate in the U.S.A., I have had several visits out there. Contractors generally speaking are convinced about it and we as a contractor certainly are convinced that we would gain a lot of advantages by using post-tensioning in slabs. There is nothing difficult about the construction techniques whatsoever. I think it is a question purely of economics and especially economics in design; I do not think that the contractor is necessarily conservative. Certainly we want to do it, there are so many advantages to the contractor and to the client, but still it is not used. However, it is beginning to show now that there may well be some economy, but I do feel that until C.P.110 is revised, it will never become the popular type of construction that it is in the U.S.A., and Australia to a lesser extent. I think that most flat slab buildings are post-tensioned in the U.S.A. and virtually all the lift slab buildings are. We have constructed hundreds of reinforced concrete lift slab buildings but we have done only one post-tensioned slab building. We have also built our own prototype post-tensioned slab structure to test, but we cannot sell post-tensioning when we are in competition.

R.V. Iyer. Could I ask anybody here if there has been any research done on the absolute minimum water content that can be used with superplasticizers in floor slab concrete? How far down can you take the water-cement ratio?

R. Colin Deacon. I would just like to chip in a thought here because this came up at a symposium on superplasticized concrete which was held in Durham last year I think. You can go down to something like a water-cement ratio of 0.3 in certain circumstances, which seemed to one or two participants, with justification, to be getting dangerously near the sort of theoretical water requirements for full hydration of the cement. The thought occurred to a number of us that if you are working down at those limits and the typical site carelessness with curing takes place, on perhaps an exposed slab, you can whistle out even more water and run the severe risk of taking out so much water that you would not get complete hydration. My personal view would be that perhaps working down to about 0.4 is far enough.

Turning to the theme of precast construction, can I come in with a suggestion, a worry really, which has not been explored this week? I got myself into bad blood with the precast concrete industry a year or so ago by having the temerity to suggest that there was no such thing as a structural topping, by that I mean a topping in conjunction with a precast unit, acting compositely, for the reason which I expressed the other day, of the great difficulty of ensuring bond between a thin layer of concrete and any other hardened part of concrete. I feel this is an area which we perhaps do not fully understand, and realistic tests have not been done. I always come back to the site problem, that in our experience when we are dealing with toppings, the reality of the situation is that toppings very frequently crack and debond. They can debond quite easily for a distance of half a metre from a crack or from a joint. If you have got that situation I find it very difficult to understand how composite action can take place and I think the tests that have been done do not fully reproduce this sort of situation which does happen on many slabs and so I think it would be a very useful thing if a realistic programme of work could be undertaken on the composite action. You see we have done a lot of work on composite action on things like bridge decks where you have got substantial amounts of concrete as the topping, tied in to the precast unit with a lot of shear reinforcement. We understand that it works, but I do not think there has been nearly as much work done in the flooring situation, where you are dealing with thin toppings of perhaps 50 or 75 mm, often without any shear connection at all, relying entirely upon bond. The tests that have been done, even those that I have seen that have tried to simulate partial debonding, in my view have not correctly reproduced what really does happen. I would like to see some tests done under simulated real conditions just to see what sort of factors of safety we do have. It would give a lot of consulting engineers much more confidence because although the precast industry is satisfied with this form of construction, and is pressing for it to be more widely used, I know that a lot of consulting engineers do have reservations about this and if we could get tests done to substantiate this situation, I am sure this would do everybody a lot of good.

Chairman. It always seems to me that the problems with a test of that nature seem to be adequate end conditions. To simulate the end conditions you have to put longitudinal restraint to the topping. I do not know whether anybody has any experience in this field. I have certainly been considering the problem in looking at composite action between prestressed lintels and blockwork and brickwork, for example, and it seems to me that this is a similar sort of problem where you are relying on the actual physical bond of the mortar between the top of the lintel and the underside of the blockwork. The problem is simulating in the laboratory the actual end restraint conditions that one achieves in practice and I think that what you are saying is probably a similar sort of problem, i.e. correct simulation.

Frank R. Benson. Perhaps I could say a word. I am not an expert in this field at all, but perhaps Colin Deacon could answer this. Does he in fact find it an impractical problem? I have always been worried about the very same thing, bonding to precast units, but one would have thought in the majority of cases the stress that has got to be transmitted across the interfaces is very, very low and presumably that is the answer that is given. Obviously it is a worry; on the other hand there are millions and millions of square feet of these type of floors in buildings, is there any sign of distress to suggest that this is a problem?

R. Colin Deacon. No, I think this is something that people do say, that we have not had any gross failures. I think that there could be two reasons for this: one that there is not sufficient bond and shear transfer, and I think Barry Hughes at Birmingham has done a little bit of work on this which seems to indicate that the problem does exist, but that it is not completely defined yet. However, I think the other real answer to this is that our knowledge of loadings is such that I suspect that our floors are so grossly over designed that the precast unit is taking all the load without calling for the topping anyway. The thing that worries me is that under C.P.110, and the way we are going with codes, our factors of safety are being whittled away while our workmanship is deteriorating. These two things are going in opposite directions and probably we are going to refine our knowledge of loads on a statistical basis over the years, so that we may be designing right down to require the full section acting compositely and will therefore be whittling away our factors of safety.

Chairman. On the same subject you mentioned workmanship there. Obviously the actual surface of the interface between the precast units and the in-situ topping must be a consideration.

Alan A. Lilley. I would like to refer to bonded cement mortar toppings to roads. It is not uncommon for 10 to 50 mm of mortar or concrete to be bonded to old concrete pavements. I have done some work in this field and I believe that the basic rule of the game is that the base concrete must be sound and clean, a condition achieved by bush hammering, and the topping vibrated very heavily in place. Under those conditions, pull off tests and so forth indicate that failure does not occur on the bond plane. The biggest single area that I was concerned with was the resurfacing of bays about 60 ft x 20 ft, with a little over an inch thick topping. This was in 1965. I had a look at it last year and nothing has happened. In my view the toppings must have cracked because they are bonded to a bush hammered slab, all I am claiming is that the topping, which was fully bonded, does not appear to have cracked, presumably because the cracks are very fine and very closely spaced.

Chairman. That is without the presence of any joints within the length you mentioned, but keeping the joints of the bays coincident with the initial slab joints?

Alan A. Lilley. Yes, we resurfaced three bays each about 60 ft x 20 ft. The joints in the topping were made coincident with those in the main slab. These main slabs had been very badly damaged by a downpour of rain and the options were to take out 10 in (250 mm) of concrete or resurface the area.

Chairman. That is an interesting economic argument, if you get damage on the surface of the slab, which is the economic solution: to properly prepare the existing slab and put a topping on or to take the whole lot away and replace it?

Alan A. Lilley. Again thinking of this particular job, an airfield, the contractor was in trouble because the resident engineer had said the concrete had to be taken out. It happened, for some entirely different reason, that I was on site and asked whether they would consider the option of a thin bonded topping. The resident engineer was very suspicious and I had to work very hard to persuade him that thin bonded toppings were viable. The contractor was then faced with the question of whether it was economically possible; furtunately he found that it was because a year later at that particular airfield they had to carry out further overlay work as part of a runway extension. The longitudinal falls of the two runway lengths were in opposition and where the extension overlay the old runway it was tapered in using a thin bonded topping, down to about half an inch. Again I looked at it a year ago and there were no defects at all. I am not sure whether thin bonded toppings could be applied to suspended floors. There may be a lot more vertical bounce or displacement with a suspended floor which could induce a much higher shear stress at the interface than I was faced with on the runway slab.

R. Colin Deacon. The key is workmanship.

Alan A. Lilley. Cleanliness at the interface is essential.

Chairman. And what about the use of bonding agents on the surface?

Alan A. Lilley. I have a personal golden rule: never use bonding agents.

George Barnbrook. I think what happens, Mr. Chairman, in practice has a lot to do with the success or efficiency of the bond and success with the topping. Most precast concrete units can be ordered to have a reasonable texture on the top surface. That is the first point that the client's adviser must be aware of. These units are then delivered to site with this required degree of texture for good bonding. The next problem in the workmanship area is keeping them clean, which could be difficult although they could be kept fairly clean with a reasonable degree of control. The other factor is grouting. I put a question yesterday to Professor Hughes about filling between the joints of his precast concrete units during the tests he did, because he was marvelling at the fact that because the grout went between one joint, it changed the mode of failure of the units he was testing in fatigue. Now the normal situation is that precast concrete units are usually required to be grouted up, as they call it, which means that there is a lot of cement and sand and water around at that time. Some people use a fine aggregate concrete and some people use a sand and cement grout. In some cases this material is spread over the whole floor so that the texture produced on the precast floor unit is completely reduced. If the topping follows that grout fairly quickly, or almost at the same time, you may have good results. If that grout is allowed to harden to some degree you have a different situation and this will affect the type of bond that can occur on the site. Now if the contractor is aware of these problems, if they are problems, he can produce well bonded toppings.

Chairman. Which brings us back to specifications and adequate site control in fact.

Alan A. Lilley. I think I would add one other danger area to the three listed by Mr. Barnbrook. The precast units may be quite old by the time the topping goes on but there will be carbonation on the surface and intuitively I would want to have them acid washed to clean them. Again we are back to cleanliness as being the crux of the problem. I normally do not like using acids but in that situation I would recommend them.

George Barnbrook. I think that provided the concrete units are given a good water washing this would give fairly good results providing there has been no massive deterioration from diesel oil dropping from compressors or other detritus. That is the area to be watched. I think going back to the basic question of the topping, there are two aspects. One is serviceability; the architect or the client would be very interested in what happens with the joints if differential curling occurs affecting his carpets and tiling. If it is in an office building, that is an important aspect. The other one is structural capacity. I agree with Colin Deacon, this is an area which is not thoroughly researched, but all the information that I have seen, mainly related to the high alumina cement problems, in tests that were done at that time, indicates that there was very little difficulty in achieving the structural capacity that was required even with, amazingly, sand and cement screeds which were said to be not particularly well compacted or bonded.

Peter G.K. Knight. The widespread use of floor ducts for services is a worrying feature of all this. Penetration of the topping seems quite likely and this could lead to the formation of planes of weakness. The provision of a screed to contain the ducts would solve the problem, but it must be of adequate depth.

Chairman. You are equating it effectively to a joint destroying the composite action between the precast unit and topping.

Peter G.K. Knight. Well if there is going to be an induced joint or plane of weakness, that is where it will be.

WORKSHOP SESSION 4

Surface Finishes and Failures

Peter C. Hewlett. Concrete structures crack and that is a practical fact. The causes are manyfold and I would - for the moment - deliberately wish to avoid discussing the causes of failure and concentrate on what can be done to aid a cracked and distressed structure, in particular using synthetic resins as the repair medium, be it as adhesive and/or sealant. The repair procedure may quite correctly and conveniently be called the structural bonding process, since we are interested in making fractured concrete monolithic again.

By means of this process, resins are injected in a controllable manner so as to fill or treat crack or void, thereby reinstating the structure to its original design capability and/or preventing further down-grading of the structure. Resin injection will not impart a greater structural capability than was originally intended for the structure. In other words, if cracking occurred due to sustained over-load, then such over-load has to be removed before resin repair can be effective. There are also situations where reinforcement may have been over-stressed, causing permanent deformation. Such situations have to be assessed in detail before the structural bonding process can be considered.

Let us assume that the cause of failure has been determined and a strong but ductile adhesive can be inserted within the fractures. We are concerned here with the ways and means of carrying out the repair, the selection of the materials, and an understanding of those factors that affect, control, and perhaps limit, this repair technique.

We are talking about placing a liquid resin - such as an epoxide or other - in cracked concrete in order to impart structural strengthening. Of course, if the resin completely fills the gap, then it will also act as a sealant. This is not its ultimate function, however, and because of the structural requirement, we think that the criteria governing effective performance must be more stringent. In very general terms these criteria can be listed as:

1. Resins should be of low viscosity and cure in a controllable manner.

2. When set the resins should be dimensionally stable and adhere to wet or damp concrete, i.e. be compatible with a highly alkaline interface.

3. Resins should be capable of accommodating movement - particularly thermal - without suffering fracture within themselves or detachment, or inducing secondary cracking near the repaired zone.

4. The equipment used for injection should be mobile, deliver a uniformly mixed product continuously, and be subject to control of injection pressure very quickly.

It is desirable that freshly mixed resin is always being delivered into the crack, giving a greater chance of maintaining steady viscosity, so assisting penetration in depth even into cracks as fine as 10 µm. The absence of batch mixing means long delivery lines can be used and inaccessible cracks can be reached without fear of the resin curing prematurely.

With this type of equipment the operator has almost instantaneous control, by means of a push button and pressure gauge, of the pressure under which the resin is injected. This is a most important point since many repairs are only to a depth a few tenths of a cm or less, and the fractured concrete may not resist much pressure without further fracture and at worst a failure occurring.

It is worth digressing for a moment to consider how pressure can be brought up in a crack and the need to control it. Resin or liquid may be represented as flowing into a simple uniform or horizontal gap or fissure, as shown in Figure 1. Penetration in this manner may be represented mathematically as:

$$Q = \frac{2b^3 \ \pi \ \Delta P}{3\eta \ \mathrm{Ln}(\frac{r1}{r2})}$$

where Q = injection rate, cm^3/sec
$2b$ = crack width, cm
ΔP = pressure drop, $dynes/cm^2$
η = resin viscosity, poise
$r1$ = radial penetration, cm
$r2$ = injection centre radius, cm

The pressure falls off very quickly close to the injection point whilst resin is flowing freely:

$$Q = \frac{k \ \Delta P}{\mathrm{Ln}(\frac{r1}{r2})}$$

As r_1 increases, so does ΔP in order to maintain Q. Unless pressure is monitored carefully and is capable of being quickly controlled, ΔP can be destructively high. Another point to note is that for a given injection pressure, flow rate falls off markedly with radial penetration into the crack (see Figure 2, curve A), resulting in very long injection time (see Figure 2, curve B). This effect makes batch mixing an inadequate procedure.

What of the resin materials themselves? Firstly, our mixed resin system is of low viscosity (400-1,500 c.p. depending upon ambient temperature, which, by nonsolvented epoxide standards, is low). This low viscosity allows adequate penetration even into the finest of cracks. There is little merit in attempting to treat cracks finer than 10 to 20 µm since most concrete will have cracks of this order due to normal hydration shrinkage.

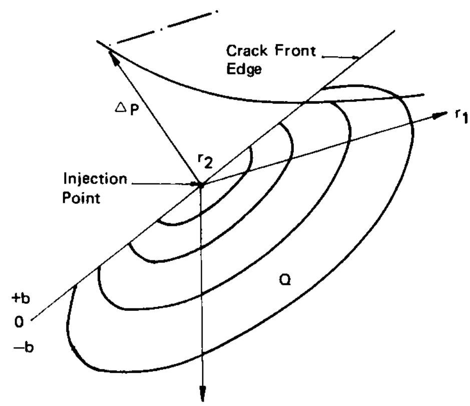

Figure 1 Fluid Flow in Planar Fissure
(The Injection Stage)

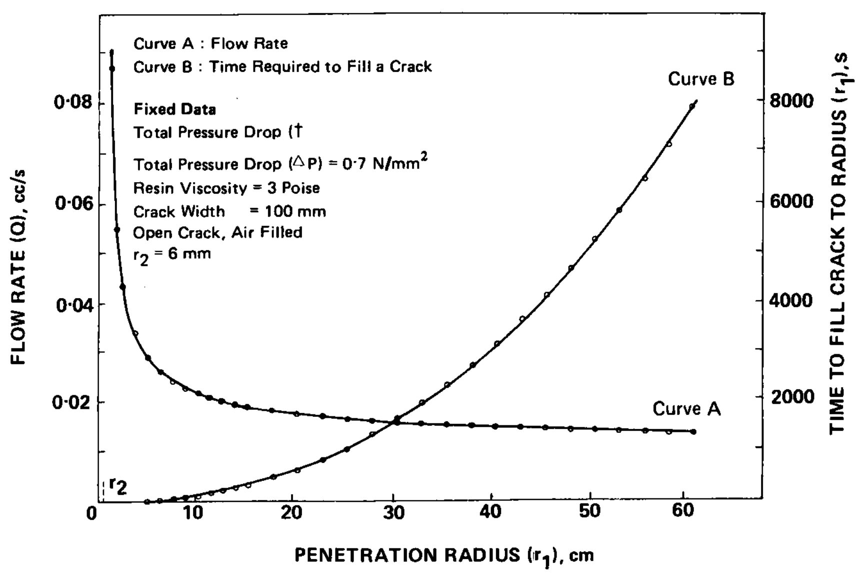

Figure 2 Relationship between Penetration Radius and Flow Rate and Filling Time

From the previous equation, the flow rate Q is linearly dependent upon the reciprocal of viscosity:

$$Q = k\,\frac{1}{\eta}$$

Therefore halving the viscosity doubles the flow rate or perhaps, more important, keeping all things constant, halves the injection pressure to maintain the same flow rate. The dependence of Q on 2b (crack thickness) is shown in Figure 3; increasing or decreasing the resin viscosity would displace the curve upwards or downwards parallel to the one shown.

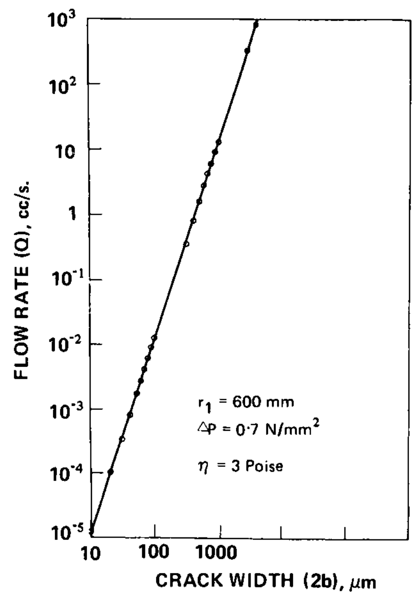

Figure 3 Relationship between Flow Rate and Crack Width

The resin described is a quick curing system (16 minute pot life per 100 g mass) giving a reasonable cure time in a thin film which probably represents the situation in a crack. For instance a 5 thou. film has a tack free gel time of about 3-3½ hours at 25°C reaching full cure in 3 days (6 days at 4°C). Once cured the resin has excellent adhesive properties to both wet and dry concrete. Direct tensile adhesion tests using dry, cored specimens gave consistent concrete (2.55 N/mm^2) and mortar (2.80 N/mm^2) failure. When cured, whilst extremely tough, the resin remains non-brittle and can accommodate some movement. The mechanical properties of typical resins used are shown in Table 1.

The resin has a lower modulus than concrete and we contend is therefore better able to accommodate movement strains*. There are few situations where a high modulus resin would be of benefit. Ideally, of course, the resin should match exactly the bulk properties of concrete so that injection results in a resin weld. As yet such resins do not exist.

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*Typical values for concrete:
Ultimate Compressive Strength = 27.6 to 69.0 N/mm^2
Compressive Modulus = 4 to 6.5 kN/mm^2
Tensile Strength = 2 to 3 N/mm^2

Table 1 Mechanical Properties of Typical Repair Resins

BULK PROPERTY	RESIN SYSTEM	
	1050	1380
Compressive Strength (ASTM D-695), N/mm^2	67.5	113
Compressive modulus (ASTM D-695), N/mm^2	$2x10^3$	$1.84x10^3$
Tensile Strength (ASTM D-638), N/mm^2	47.6	62.4
Elongation at Break (ASTM D-638), %	-	2.5
Flexural Strength (ASTM D-790), N/mm^2	68-79	83.4
Flexural Modulus (ASTM D-790), N/mm^2	$2-2.84x10^3$	$4.8x10^3$
Heat Distortion Temperature (ASTM D-648), °C	~49	57-59

We have mentioned the equipment and materials, but, in addition, this method has made an improvement on the ways and means of preparing cracks and locating injection centres. It has been common practice, when preparing the crack for injection, to chase out loose material and locate injection nipples by drilling out first. In anything but very wide cracks, this procedure is no longer recommended since it is expensive and drilling can cause dust and detritis to be lodged in the crack, reducing adhesion, as well as reducing accessibility into the crack.

Depending on crack width, we use a thermoplastic seal (0.2 to 1 mm) through a filled polyester (1 to 6 mm) to a fast setting cement or coarse grained filled polyester. Each sealing method has its own injection point location technique, and post injection drainage stop. The thermoplastic is simply applied to the crack hot, the crack having previously been taped at the injection centres. The thermoplastic cools almost immediately effecting a seal. The tapes are removed exposing a short length of crack which becomes the injection centre. The mixing head of the injection gun has a self-sealing rubber gasket on the outlet and this forms its own injection cell. As the injection proceeds along the crack, the holes are sealed using solid wax which is rubbed over the surface, or by rods or suturing of injection tubes. After an entire crack length has been treated, the thermoplastic is simply removed with a spatula or, on complicated shapes, by sand blasting, leaving the surface almost unblemished.

It is not always possible to be confident that all the cracks in a structure have been treated, but then total treatment may not always be necessary. With the exception of cracks resulting from continuous tensile stress, or where cracking may have caused reinforcement to be over-stressed, partial resin injection may be all that is necessary.

Looking ahead a little, there may be developments in the area of injected sealants. Being applied in thin gap situations it will be necessary to make very flexible materials since excessive strains may occur. Developments in gaining access to cracks, such as between an applied render and a tile facing, require thought. Resin materials having bulk and thermal properties which are closely matched to those of concrete may also evolve.

I think we can say that this method of resin injection has removed a lot of the doubt associated with previous techniques, and is an attractive, and very often cheap, means of structural repair.

CLOSING DISCUSSION

Chairman: Ravindra K. Dhir

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Contributors

R. K. Dhir
C. Hobbs
D. N. Trikha
J. M. Rolfe
G. C. Mays
R. K. Dhir

CLOSING DISCUSSION

Ravindra K. Dhir. Gentlemen, to keep up with the superb organisation of the Conference Proceedings which we all have witnessed since Monday, and which many of the participants have commented on to me, I suggest we should begin this Closing Session on time, now. First of all, I must say that right through these Proceedings my task as the Convener of the Conference has been a very easy one. The Conference Organising Committee has looked after me very well. The Session Chairman and Keynote Speakers, the Authors of the papers presented at the Conference and the other participants have all performed a splendid job.

It is also most gratifying when one organises a Conference of this nature to get the kind of response we had, particularly the response from abroad. We have had the privilege of having twenty-four countries represented at this Conference and I sincerely hope that this international dialogue will continue in the future.

The cross-section of organisations represented at the Conference was generally wide ranging, although I think I also speak for the Conference Organising Committee here when I say that we would have liked to have seen greater representation from consulting engineers, even more contractors and at least a few architects of whom regrettably we had none.

Coming then to the Closing Session of the Conference, the initial thought of the Conference Organising Committee was that I should attempt to summarise all the papers presented and discussions which took place, including the discussion which took place during the Workshop Sessions this morning. Reading the papers, listening to the authors' presentations and to the tremendous amount of disucssion which has ensued, which has certainly highlighted the need for the Conference, I feel that an overall summary would be an impossible task to perform. What I intend, therefore, is to ask the four Chairmen of the Workshop Sessions, and they have agreed to it, to briefly summarise their thoughts and the thoughts of the others who attended their Sessions to see if there is any message which has come across during the Conference. As Chairman of Session 1, could you please begin Mr. Hobbs.

Cyril Hobbs. The few things I do say about Workshop Session 1 will perhaps confirm what happened in the last Workshop Session and show that I am reflecting something of what came out of the Conference.

As a keynote speaker on the materials side, I put forward the idea that we had made a lot of progress in the field of concrete technology over the last thirty years and a great deal of it has been put to use. I thought the next thirty years was going to see a considerable elaboration and expansion of the whole field, bringing in a larger number of new materials. Throughout the Conference I seem to have got from discussions in Sessions, between Sessions and from this morning, a general agreement that yes, we have made a lot of progress over the last thirty years and a lot of technology has been sorted out and we are a lot wiser than we were then. Out of it all, however, has come quite clearly a sense of disappointment that whilst we have made quite a lot of progress in understanding what concrete is about and understanding what we are trying to do, we have been less successful in getting this knowledge down the line, in getting it passed on to people, in getting it applied and actually getting it used. I think this feeling of disappointment in the level of use came out several times in the disucssion. So that when we started looking at the future and began to think about where we were going, there was some feeling that since we were having such a lot of trouble sorting out the technology of a concrete made up of about four basic ingredients, we are going to have even more trouble over the next thirty years if we find ourselves trying to sort out a concrete made up of six or seven or eight different ingredients. I think perhaps the Conference was less optimistic than I was about where concrete technology is going to go over the next few years.

What certainly came out, and I think it came out very strongly, was this problem of taking the message down the line, getting it used, getting it applied, getting the communication right. I have written down three words which to me seem to be the theme that came out of this Conference. The first word is 'communicate', for God's sake learn how to pass to each other information about what it is we are trying to do. I think we have to recognise in our industry particularly that we very rarely go down learning curves because in our industry it is very rarely that we put together exactly the same team to repeat exactly the same operation several times over. We hardly ever put up two identical buildings. When we do we almost certainly build them with either a different client, or a different consultant, or a different contractor. Even if the building is the same, somewhere along the line the team is different so the learning curve, which can go on in many other industries, does not happen in ours. This makes it very essential, therefore, that each team should learn some lessons and then pass the lessons generally throughout the industry to whoever might be in the next team who get together to do something similar. I think this is one of the problems we are always meeting. So the first lesson is communication.

The second is 'revelation' and by this I mean passing on our experiences, bad as well as good. We have talked about this problem of failures and passing on reasons for failure and monitoring buildings and so on. The problem here is the continuity of interest. The person who puts a building up and would like to monitor it, to get some information about it is probably never again in quite that same group, or doing the same sort of thing again, so the continuation of the monitoring and the feeding of the information back is not something that any one person ever quite identifies with.

Then the last word that I put down, this came out again this morning, was 'simplify'. Somehow or another we have got to get the complexity sorted out before construction begins. If there has got to be sophistication, it has to be at an early stage so that when it comes to the actual operation on site then that is simple and easy. It is no good putting all the complexity on to the site. So I have these three words, 'communication', 'revelation' and 'simplification' and it seems to me that those are the themes that have come out of this morning's Session and the Conference as a whole.

D.N. Trikha. In the Workshop on Structural Design we tried to tackle some of the issues which emerged from the papers presented in Sessions 2 and 3. In order that we could have a lively discussion, the various aspects of analysis and design of slabs covered in these Sessions were grouped into seven themes. These were elastic analysis; non-linear methods of analysis, like the finite element method, yield line theory, the strip method and design at ultimate load; flat slab design and punching shear problems; slabs in frames and shear wall buildings; optimum design; pavement slabs and slabs on the ground; and experimental work. In the very short time at our disposal we did have a very meaningful discussion and many points were raised. I find myself totally inadequate to summarise all the opinions expressed. However some of the points which were made, I shall try to elaborate.

On elastic methods or strip methods of design it was felt that the generalised form proposed at the conference involves solution of a set of linear simultaneous equations. Some people expressed the fear that this made the original method more complex without necessarily increasing the accuracy, especially at working loads. Others of course objected to this observation. It was, however, agreed almost unanimously that when suggesting a method of analysis the codes must be careful to give the limitations of the method and indicate particular situations in which the method gives proven accuracy.

In non-linear analysis it was pointed out that in most cases membrane forces and tensioning effects were neglected, and non-linear formulations by finite element methods were questioned. It was ultimately agreed that although finite element methods could be used to consider various aspects, the finite element formulation must essentially remain simple enough to be tractable.

In flat slabs and punching shear, many people are generally sceptical about the provisions in this regard in the codes, especially in C.P.110. It was pointed out that the rules are based on tests which do not really simulate the stress conditions of flat slabs. A case was made for greater experimental and theoretical effort.

In the case of slabs in frames or slabs in shear wall buildings, a very important point was raised as to the applicability of the methods of analysis to predict long-term behaviour. It was generally felt that although theoretical models could be proposed for such studies, there was greater need for tests on a long-term basis. The hard-pressed academicians threw up their hands and thereafter agreed that such a task should be performed by field engineers. Since the data may not always be acceptable to reputed journals for publication, a way should be found for wider dissemination of this information.

For the topics of optimisation, slabs on ground and experimental work, there was insufficient time for a full discussion. However, a general point which did emerge was that simulation of soil underlays by Winkler type reactions could lead to wrong results. Geotechnical engineers were specially requested to give realistic but more usable data to structural engineers for soil-structure interaction studies.

John M. Rolfe. The Session that I was concerned with was Surface Finishes and Repairs and Maintenance. I think the feeling that came out of the Session was that surface finishes are very much to do with repairs and maintenance because new surface finishing techniques have been developed to cope with systems which did not work in the past. We have had floor failures in the past and you come back again to try and solve the problems with the new methods and new techniques which have been developed. We spent a little while looking at certain techniques which were not fully ventilated in the discussion after the

Conference Session and the discussion then rather diverged from the topic of the papers before us at the Conference on to what it was all about: why do we have buildings going wrong and if they do go wrong who is to blame and how do you put them right? I think the conclusion reached at the end was that we need more communication, that we can learn from buildings which fall down. We all know this, but people are not very keen to have their failures discussed in public.

It was suggested that we should also learn from buildings which stand up, by systematic instrumentation of buildings and long-term monitoring of them. Evidently there are difficulties in this, this has been made very clear, but surely this is what we, the engineering profession, are there for, to overcome difficulties. This is what we are doing all the time, we are in the communications field and we have got failures because we fail to communicate properly. We fail to communicate at every level. The owner fails to communicate with the consulting engineer, the consulting engineer fails to communicate with the owner. They both fail to communicate with the contractor who fails to communicate with the sub-contractors and they fail to communicate with the workman on the site who is the man who has to do the job in the end.

I feel very strongly myself that the engineer is the interpreter. He is the man in the middle of the whole complex of communication because he has to take instructions from an architect who says: 'I don't care how many columns you have, as long as you have none on the ground floor' - I have had that actual instruction given to me. He has to take instructions from an owner who says: 'I want floors to last forever at no extra cost'. He has to take instructions from technical journals and the academic world which talks about 'semi-infinite elastic half spaces' when it is merely talking about how a floor is performing under load. He then has to take this to, in your case, I believe, Irish navvies, in my case unsophisticated, largely unskilled workmen and try and get them to construct one part of the 'semi-infinite elastic half space'. Unless the engineer communicates at all levels he cannot succeed. I think the feeling of the Workshop Session was that we only learn by communicating with each other at all levels. We are all cogs of the industry whether we are academics, consultants, contractors, clients or specialist commercial firms and unless we do get together and work as a team we are not going to achieve the results we would like to achieve.

Geoffrey C. Mays. Rather than taking any general consensus of opinion, the discussion during the Workshop on Construction Techniques ranged round a number of specific topics. We started off by talking about ground floor slabs in general terms and the use of dowel bars as opposed to shear keys for load transfer at joints within ground floor slabs. This raised the question of the increases in reinforcement which are necessary in continuously reinforced as opposed to jointed slabs. Discussion then moved to the significance and the effects of thermal changes causing shrinkage cracking in ground floor slabs which are under cover, as opposed to the view that the main problem in ground floor slabs is one of drying shrinkage. The consensus of opinion was that for the normal thicknesses of slabs which are constructed under cover in buildings, there was no evidence to suggest that thermal changes would cause a problem.

The idea of using prestressing in ground floor slabs was mentioned, the purpose being perhaps to use the compression exerted by the prestress to reduce shrinkage effects. It was felt, however, that significant problems would be encountered, mainly due to losses of prestress, from friction from the ground, for example, so that at the end of the day there would be some doubt as to the actual level of prestress one actually had in the ground floor slab. Again in connection with prestressing, opinions were expressed on the difficulties to be encountered with ducts and accesses through slabs, whether they be ground floor slabs or suspended slabs.

We then moved on to the rather questionable area of why in fact the use of prestressed slabs is not as common as it could be. Here there are obviously two conflicting arguments: one which says that it is conservatism on the part of contractors to change their methods and the other which goes back to the question of the problems of uneconomic methods presently being used in design.

A point was raised on the use of superplasticizers as to the minimum water content which one would desirably use and I think it was generally felt that there should be a minimum limit, and it was suggested that a minimum of 0.4 should be recommended, to prevent a lack of hydration of the cement.

We then moved on to a topic which was rather out of the actual field for this particular Workshop Session and that was the problems of cracking and debonding between in-situ concrete toppings and precast units and the problems encountered because of the loss of composite action. The discussion closed by expressing the need for adequate workmanship and surface preparation both when preparing precast units for the reception of the in-situ toppings and when preparing a surface to receive repair work.

Ravindra K. Dhir. Thank you gentlemen, you have left me just sufficient time to say a few final words before formally closing the Conference. The main purpose behind organising the Conference, as the Committee saw it, was to explore a wide range of themes pertaining to concrete slab technology and to seek the widest possible interaction between people and organisations concerned with slabs. The Committee felt that in this way it is possible to identify any problems as a first step towards looking for the solution. My Committee and I would like to think that the lead this Conference has attempted to provide could be taken up elsewhere and discussion and dialogues started here would continue in the future.

With these few words, I would like to close the Conference. Although for me it is a moment of anticlimax, my wife is pleased as she and the family would be able to see more of me which they have had to do without during the last three months or so. May I finally wish you a happy journey home. Thank you very much for coming along. It was a pleasure to meet you and to work with you for a week and I hope we will meet again somewhere, sometime. Thank you.

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