CONCRETE MIX DESIGN, QUALITY CONTROL AND SPECIFICATION

SECOND EDITION

KEN W. DAY

E & FN SPON

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Concrete Advice Pty Ltd, Croydon, Victoria, Australia

London and New York

First published 1995 by E & FN Spon, an imprint of Chapman and Hall

Second edition published 1999 by E & FN Spon
11 New Fetter Lane, London EC4P 4EE

Simultaneously published in the USA and Canada
by Routledge
29 West 35th Street, New York, NY 10001

E & FN Spon is an imprint of the Taylor & Francis Group

This edition published in the Taylor & Francis e-Library, 2005.

British Library Cataloguing in Publication Data
A catalogue record for this book is available from the British Library

Library of Congress Cataloging in Publication Data
Day, Ken W.
Concrete mix design, quality control, and specification/Ken W. Day. —2nd ed.
p. cm.
Includes bibliographical references and index.
ISBN 0-419-24330-5 (hardbound: alk. paper)
1. Concrete-Mixing-Quality control. 2. Concrete-Specifications.
I. Title
TA439.D39 1999
620.1′36–dc21 98–51602
CIP

ISBN 0-203-22073-0 Master e-book ISBN

ISBN 0-203-27554-3 (Adobe eReader Format)
ISBN 0-419-24330-5 (Print Edition)

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Preface to second edition

A great deal has happened in both the computer industry and concrete research laboratories since the first edition of this book. As a conse-quence, significant changes are also occurring in the premix concrete industry. Since the objective of this book is to provide guidance to those wishing to employ the best available techniques to design, control and specify concrete, it must necessarily be revised.

Since many other books are currently being published on ostensibly similar subjects, it is important to define the objectives of this book more precisely. The book is not the result of library or laboratory research and is not academic in tone. It rather seeks to present in a readable manner the author's personal knowledge and experience of designing and con-trolling production concrete. In several respects, the views expressed are contrary to those traditionally held and this was one motivating factor for the first edition. The book should be of interest even to those who have no intention of ever operating a computerized control system but who need to be aware of common fallacies in the quality control area. It is essential reading for those actually involved in the operation of qual-ity control.

A secondary objective is to present the computerized methods devel-oped by the author. These methods have enabled some of the major pre-mix suppliers of the world to attain better control and lower variability than is otherwise attainable. An effort has been made to achieve this in the simplest possible manner, using the absolute minimum amount of test data and no complicated mathematics. The Windows version of the system now presented gives greater speed and simplicity of use and can handle large quantities of data, but the basic techniques can still be applied in a spreadsheet format as was originally used by the author (Day 1988–9).

It is not denied that recent developments in packing theory mix design can cope more accurately with extremes of grading and particle shape than the author's specific surface technique. However, the author's technique is simpler, requires less data, and usually gives similar answers over a substantial range of cement contents and aggregate types. It is also noteworthy that tables of mixes generated by this system are usually in close agreement with well established tables in use by major concrete suppliers.

An innovation is the inclusion of a compact disc (CD) with this volume. Hopefully most readers will have access to an IBM compatible computer with a CD-ROM drive and operating on Windows. If so the CD will permit them to view (and listen to if they have sound) screencams (essentially similar to videos) demonstrating use of the system. There was concern in the first edition that multigrade, multivariable cusum graphs would lose much of their impact and appear difficult to follow when only seen in monochrome. The CD enables full colour presentation. Also programs are provided enabling readers to enter their own data to examine whether it supports the author's contentions about such matters as cusum quality control, prediction of 28-day results, and specific surface mix design. Further information about the CD may be found at the end of the book.

The book may be unpalatable to specification writers not prepared to consider change. It is the author's contention that the practice of specifying minimum cement contents, in particular, has held back the development of concrete production technology by many years, perhaps by as much as 40 years. It is not the only culprit but it is the main one. Other factors include refusing permission for rapid mix adjustment, insisting on inefficient testing regimes, not permitting the use of pozzolans, and failing to require or reward low variability. Surprisingly, it appears that the USA is prominent amongst those hampering development in these ways.

The main use of the original spreadsheet system and its manual fore-runners in the 1970s and 1980s was to monitor concrete quality on behalf of major projects (Day, 1969, 1981). The system has lost none of its capacity to do this in gaining features which make it very attractive to concrete producers, indeed it is now possible to economically control projects anywhere in the world from anywhere else in the world. However, it appears that those project controllers who are advanced enough to know of the system are also smart enough to realize that it is better to have the system operated by the concrete supplier (Petronas Towers, Malaysia, being a major case in point).

There is some danger that the new in-words 'quality assurance' will attain undue prominence and submerge admirable intentions in excessive paperwork, raising those in charge of the bureaucratic aspects of quality assurance above those who actually understand the mix design and quality control of concrete. It should not be so, and it need not be so. Quality assurance is good for concrete if properly applied. Conad quality control now provides a 'Quality Assurance Diary' feature which may obviate much of the usual paperwork and time consumption in maintaining complete and verified records of the control exercised.

Finally it should be said that, as with computer use in general, the technology behind the system may grow evermore complex, but hopefully the user will find it easier and easier to use.

Preface to first edition

This book has the limited objective of teaching the reader how to design, control, and specify concrete. Although few people currently carry out these operations well, they are relatively easy to learn. However they are analogous to driving a car as opposed to becoming an expert mechanic.

A further objective is to emphasize that the application of more advanced technology to these matters should reduce rather than increase cost. The selection of an appropriate quality or durability is important, but it has little to do with quality control/assurance. The objective of the latter is to enable attainment of the selected quality at minimum cost.

The realization is dawning that it is essential for concrete to become a fully reliable or 'zero defects' material rather than a material of questionable quality which the purchaser must thoroughly test and accept or reject. This is because the incorporation of a single truck of defective concrete in a structure incorporating 20 000 such truckloads can give rise to costs of investigation, of replacement, and more importantly of delay, well in excess of $1 000 000. It gives some idea of the resistance to change that this situation, which few would now deny, was pointed out by the author in the 1950s (Day, 1958–9).

If the above contention is accepted, it must give rise to a new set of rules and concepts. We have learned that there is no such thing as an absolute minimum strength. We now have to learn how to ensure that no structurally unacceptable concrete is supplied, i.e. to detect and rectify adverse quality shifts before any actually defective concrete is produced. Concrete cannot reasonably be rejected on the basis of the 28-day strength tests when there may, by then, be another five or six storeys of the structure built on top of it. Control action must be seen as an urgent and highly organized activity in which time is the essence and an hour is a long time.

The author has spent more than 30 years designing, controlling and specifying concrete. In doing so he has found remarkably little assistance from standards and codes of practice. In effect it has been necessary to operate on two planes simultaneously. One of these is the official plane on which one must check for compliance with specifications, codes of practice, etc., and the other is the practical plane on which the satisfactory outcome of the work actually depends. It is the author's hope that this book will assist in reconciling standard practice with realism.

The assumption is made that the reader has access to at least one com-prehensive work on concrete technology and little of such standard material is reproduced here. The implication may be noted that little of this material is actually used in the day to day design and control of con-crete. Whilst this is true to a considerable extent, it should be realized that proceeding in the absence of a more comprehensive knowledge of concrete technology can be like walking through a minefield with a map showing only natural features.

It used to be sufficient to know the simple things about concrete and to assume that inexplicable random variations were inevitable and should merely be allowed for. It is not so, there is an explanation for everything which happens to concrete and it is necessary to discover the explanation if we wish to eliminate recurrence of the event.

It is not possible for any individual to acquire more than a small part of even the currently available knowledge of concrete technology and further knowledge is becoming available at a bewildering rate. Indeed a person's knowledge tends to be the inverse of what he conceives it to be, since in doubling his knowledge, his perception of the totality of avail-able knowledge quadruples and he appears to be only half as far along the path to total knowledge.

The best one can hope for is to have the particular 10% of knowledge which is actually necessary for the task in hand. The objective of this book is to provide such knowledge of a small, but intensively used, corner of concrete technology. In so doing, the reader will be made aware of other problems to which more detailed answers should be sought else-where.

It is important to understand the place of computers in the fields of mix design and quality control. Very little knowledge about computers is necessary in order to use them to great effect in these fields and they are so powerful and cost-efficient that they are virtually indispensable. A computer must not be thought of as an infallible font of all knowledge, rather it should be seen as a microscope which will reveal what is otherwise unseen and inexplicable. It is a data processing unit which can accept vast quantities of information and carry out a very complicated analysis of it in a few seconds. One message of this book is that we are rapidly approaching the point at which complete automation is techni-cally possible, but it must be realized that computers work on the GIG0 principle (garbage in, garbage out) whether that garbage is inaccurate test data or an unrealistic program.

The pace of technical progress heaps additional pressure and respon-sibility on concrete technologists, but it also provides the means to react swiftly and precisely to change. No longer need filing cabinets be searched for data, yield calculations and statistical analysis done and combined grading curves painfully constructed. The already expert technologist is not disenfranchised but is assisted to work a hundred times more quickly with all the hack work done automatically. The raw amateur is helped and advised not only to a solution but also to an understanding of the situation which may otherwise take years (and many costly mistakes) to acquire.

Some will object that this book is impractical because its recommendations cannot be implemented within the existing structure of codes of practice and national specifications in their country. There are two replies to this. One is that the book describes how things can be done better and how they are likely to be done in the next decade. The other is that there is little in the book which has not already been done by the author. Of course it requires co-operation, but if the controller and the controlled both want effective and equitable control then it is possible to implement it (particularly on very large projects) whilst still paying the necessary lip service to official requirements.

Layout and scope

It is again emphasized that this book is not intended to be a first or only source of knowledge on concrete technology, nor does it attempt a coverage of all aspects. The author therefore makes no apology for the omission of material which is both non-contentious and readily available in any book which does attempt a comprehensive coverage. Thus, the book does not describe the production of concrete materials, well-established test methods, or the chemistry of cement and admixtures.

The principal matters addressed are in the first six chapters with supplementary material in the remaining chapters. The material in the later chapters is aimed more at providing context and explanation for the views and techniques in the earlier chapters than at providing comprehensive information on the subject of the chapter.

The book does attempt a limited historical perspective and a brief survey of the alternatives to the author's methods of mix design and quality control. This is done partly to avoid any unjustified appearance of originality and partly to show the limitations of other approaches.

Many would consider it logical to place specification ahead of mix design but the author's view is that specification cannot usefully be considered until the processes of design and control have been studied. Concrete has to be fully understood before it can be effectively controlled. Too many specifications have been written without an understanding of the material and its production.

Acknowledgements

There are three individuals without whom this book could not have happened and four more without whom it may have been very different. The first group comprises: O.Jan Masterman, Technical Director, Unit Construction Co., London in the 1950s, who somehow inspired and guided me to originate in my first two years of employment the greater part of the philosophy and concepts herein recorded; John J.Peyton, John Connell & Associates (now Connell Wagner), Melbourne, without whose encouragement I would never have started my company Concrete Advice Pty Ltd in 1973 and so the nascent control techniques would never have developed to fruition; John Wallis, formerly Singapore Director of Raymond International (of Houston, Texas), without whom my Singapore venture would have foundered in 1980, leaving me without computerization and without the broad international proving grounds for the mix design system.

The second group comprises: John Fowler, who wrote the first computer program using my mix design methods, at a time when I had a firm opinion that mix design was partly an art and could never be computerized; D.A.Stewart, whose book *The Design and Placing of High Quality Concrete* (Stewart, 1951) was a first major influence; David C. Teychenné, who led where I have followed in specific surface mix design; and my son Peter, who transformed Conad from an amateur spreadsheet into a professional computer program.

A third kind of indebtedness is to those who assisted in the actual production of the book. They have become too numerous to list all of them by name but Hasan Ay and Andrew Travers are especially thanked for their work on figures and tables and, for the second edition, Matt Norman.

Harold Vivian, Bryant Mather, Dr Alex Leshchinsky and Dr François de Larrard are especially thanked for invaluable advice and contributions, Sandor Popovics for his published works and thought provoking discussions, Joe Dewar, Bryant Mather and John Peyton for their kind Forewords, also Vincent Wallis on whom I have relied for an (often brutally) honest opinion over more than 30 years, and of course my wife, who has endured a great deal in the cause of concrete technology.

A new kind of indebtedness is to those individuals in my major client companies who have not only enabled my company (Concrete Advice Pty Ltd) to survive and prosper but have also contributed in no small measure to improvements in the system. They include Peter Denham and Dan Leacy of CSR Readymix, Paul Moses of Boral and Mark Mackenzie of Alpha, South Africa.

The Conad computer program has come a long way since the first edition and thanks are due to my staff at Concrete Advice Pty Ltd. Michael Shallard and Lloyd Smiley wrote the latest program and Andrew Travers, now Manager of the company, knows how to use it better than I.

Finally, I must thank my younger son, John Day, now Technical Manager of Pioneer Malaysia, for using these techniques so effectively as to make the world’s tallest building, Petronas Towers, the best example yet of low variability, high strength concrete.

Foreword
by John J.Peyton,
Director, Connell Wagner Rankin Hill, Consulting Engineers

The writer is a consulting engineer based in Melbourne, Australia, and in charge for the last 20 years of the structural work of the office of Connell Wagner, the largest structural office in Australia. In the 1970s and 1980s, and continuing to the present day, we have been involved in some very large and prestigious building structures, most of which have been in concrete and many of which have pushed back the previous barriers in terms of concrete strength, durability, appearance, and general quality requirement. Outstanding early examples were the Arts Centre and Concert Hall. These were prestige structures required to have a substantial degree of permanency and situated in some of Melbourne's most aggressive soil conditions. Collins Place was another landmark project of the early 1970s comprising twin 50-storey towers. It was the first use of 8000 p.s.i. (55 MPa) concrete in Australia and an early use of structural lightweight concrete in the floors. Subsequently we have used 80 MPa concrete and permitted it to be pumped 'bottom up' as a fully flowing material in four-storey sections of tubular steel columns in high rise construction. In the 1980s and 1990s, 50 to 60-storey buildings have become a commonplace. The increased tempo of construction has caused us to accept such techniques as insitu temperature monitoring to enable earlier prestressing. Gone are the days when conservative nominal stripping and stressing ages could be specified.

Consulting engineers in general have considerable difficulty dealing with the vagaries of concrete test results. Excessive severity in dealing with marginal results is a waste of resources, yet any leniency results in continued infringements. What should be done with the odd set of low results? What if the concrete is already inaccessible, or has a storey or more of subsequent construction on top of it? How can a recurrence be prevented? How can it be explained to the client or the Building Authority? Dare we really use high strength concrete?

In many cases recalculation of the force on the particular offending element permits its acceptance at a lower concrete strength. However, this leaves a suspicion in the client's mind that the whole structure has been over-designed. In other cases the element is jack hammered at con-siderable cost and inconvenience, or perhaps load tested. This may be considered necessary to maintain control, even when it is obvious that the shortfall is insufficient to cause a structural problem. And how do we know that the trucks of concrete which were not tested are any better than those we did? It is possible to cause endless disruption and more problems by non-destructive testing. The fact is that consulting engineers in general are structural designers rather than concrete technologists. The more one knows about concrete, the easier it is to admit the truth of this. Only those who have stayed within conservative limits and con-fined themselves to routine work can sustain a belief that they actually know all about concrete. It is a false economy, and a disservice to the client, to soldier on with the same old specifications and limits, ignoring new technology.

This brings me to Ken Day and the almost magical disappearance of these problems when he appeared on the scene in the late 1960s. Ken always seemed to be able to put his finger on the cause and extent of any problem with concrete and to cut through the typical hocus pocus rea-sons given by suppliers and contractors. I referred to him at one stage as our favourite form of cash penalty whenever problems were encoun-tered. Better still, we found that the problems simply did not occur when we required contractors to engage his services in the first place. At last the formality of prior mix approval acquired real meaning.

The writer realized how valuable a resource this was, and by the early 1970s arranged to have Ken's services used more extensively on our pro-jects through specifying his engagement by the contractor. This enabled him to develop his mix evaluation, result analysis and reporting services to a high order. I was particularly impressed by the early stage at which any slight shortfall was detected and resolved before any actual low results were experienced. His services freed us from concerns as we pushed strengths higher and achieved greater economy when able to rely on full attainment of our design intentions.

This book sets out in a challenging way the basis of Ken Day's unique capability in the control of concrete quality. I am pleased and proud to see that Ken believes I had a hand in the origination and recognition of these skills. I recommend the book as required reading to any person charged with the specification and responsibility for concrete quality, and as an in-depth study to concrete technologists who aspire to assist them effectively. However let us all hope that Ken succeeds in his mis-sion to educate the concrete producers of the world so effectively that there are no longer any problems to overcome.

Melbourne, July 1993

Foreword
by Bryant Mather
Director, Structures Laborafoy, US Corps of Engineers
Waterways Experiment Station

I was once required to attend night courses at the laboratory in which the Director, Mr Charles E. Wuerpel, taught a course in concrete mix design. I failed to learn to do it—too much arithmetic. This was in 1942. This book, I believe, tells the reader that there is a great deal more arithmetic now than then, but don't worry; the computer will take care of getting it done, provided you instruct the computer properly

Somewhat later I became convinced that most of the time one could make concrete of appropriate quality for a given use with locally avail-able materials. I once upset some nuclear physicists by suggesting that 'sidewalk' concrete from the local neighbourhood ready mix plant, if uni-form, would make just as good a biological shield around a nuclear reac-tor as concrete with high density aggregate imported from a thousand miles away I suggested that the extra volume needed would cost less than the shipping cost of a smaller volume of expensive aggregates. Ken Day understands this.

It has been one of my theses that properly proportioned concrete can't be over-vibrated—but one needs to proportion concrete for 'excessive' vibration if one wants to vibrate it excessively Ken knows this, too. I have also argued that one can design structures (including pavements) for whatever loading is relevant and then select a design concrete strength for that loading (compressive, flexural, or whatever) and pro-portion a mixture to achieve it; but having done so, it is dumb not to use compressive strength for routine control testing; Ken says this. He knows that hot weather does not directly increase water demand, he just implies it like everybody else. However, unlike everybody else he has a proposal ('equivalent slump') to get to the truth of the matter, as I have been urg-ing for many years.

The concept of distinguishing acceptable non-conforming concrete from unacceptable non-conforming concrete that Ken calls 'structurally defective' and 'contractually defective' needs more emphasis, and this book provides it.

It is good to be reminded of the fact that, when worrying about the 'actual' strength of concrete in a structure, it is quite proper to worry about its present strength under some conditions and its eventual strength under others. It is also good to read: 'Fortunately the pressure to specify minimum cement contents was resisted.' We had the battle of New Orleans about this in the ACI and a rule was made that if there were to be a minimum cement content there must also be an alternative performance requirement that if met would obviate the need for the prescriptive one.

I was very happy to see 'waterproofers' put into quotation marks, since the term implies doing something that is not possible.

For these and other reasons I am pleased to commend this book to its readers, in spite of my belief that, in concrete technology, 'mix' should only be used as a verb and 'design' should only refer to selection of properties of concrete structural elements and hence the title should have been 'Concrete Mixture Proportioning.'

Clinton, Mississippi, July 1993

Foreword
by Joe D.Dewar
J D Dewar Consultancy (Former Director, British Ready-Mixed Concrete Association)

Concrete is moving fast from the stage of an art to that of a science; rule of thumb is being replaced by a blend of theory and experience and the development of expert systems aided by the computer.

While it has been said that almost anyone can design and control concrete because it is such an accommodating material, it still takes an expert to do it economically and consistently. To teach others to do so requires an even rarer brand of expertise. Ken Day has demonstrated his ability to be a leader in this field.

Ken is one of the few world citizens working in concrete with his experience drawn from five continents. He can write easily about cubes or about cylinders, about working in the tropics or in temperate climes, operating in advanced cities or in a wilderness.

If you are an expert concrete technologist, there is a wealth of information on well-tried and new systems which you can adopt or adapt as seems right to you. If, on the other hand, you are a novice, this is also the book for you because Ken Day has many things to teach us and he does so with a relaxed style that makes reading this book a pleasure. More than this, he is not a mere theoretician suggesting a new approach which might work. He is a practitioner who is sharing his life's knowledge bought by hard work and learning from his experiences.

You do not need to agree with everything Ken writes or to do all that he proposes. His idea is to make you think before you draw conclusions or make decisions on the situations you face.

There are many statements in the book with which, from my experience in European standardization and ready mixed concrete, I would strongly agree, including: the unlimited future prospects for the application of computers; the potential for further developments based on the cusum system of control; control testing by the producer is more efficient and cost effective than acceptance testing by the client; mix design should allow for different levels of cohesion; laboratory trial mixes should not be a specification requirement and are an inefficient use of resources; good concrete can be made with almost any aggregates if the proportions are correct; specifying by strength is the best way to ensure conformity with the durability requirements of minimum cement content and maximum water/cement ratio.

There are a very few ideas with which I would disagree, specifically: use of specific surface or surface area index as a basic design parameter; and cash penalty specifications. These are a heaven sent opportunity for the less scrupulous to capitalize from poor testing. The provision of positive incentives is a far more attractive proposition than negative penalties.

But these are small criticisms because Ken's ability has enabled both of these ideas to work for him. He identifies that most concepts have their limitations.

I hold the view that each day should present a new challenge, a different viewpoint, a change of mind, or an addition to knowledge. Ken obviously shares these views. When Ken stops producing new ideas it will be because either he or the use of concrete has come to an end. I look forward to the next edition, and the next….

Strawberry Hill, England, August 1993

1
Design of concrete mixes

1.1
PHILOSOPHY

There are hundreds of systems of concrete mix design, just as there are hundreds of cures for the common cold. In both cases the question is whether any of them really works. In the case of concrete mix design there is certainly substantial evidence to the contrary. Nearly all systems end by suggesting eye adjustment of a trial mix. Most commercial concrete results from the continued *ad hoc* modification of existing mixes without any application of formal mix design.

If the purpose of a mix design system is to enable ideal materials to be proportioned so as to produce good general purpose concrete of the desired strength then it will have very limited value. To be of real value a system must be able to guide the selection of available materials (of whatever quality) and proportion them so as to produce the most economical concrete which is suitable for the desired purpose. It is not particularly essential that the first mix produced has exactly the desired strength (although it may be essential that it exceeds this strength) since it is easy to subsequently adjust cement content. The first essential is that the most advantageous selection of aggregates be made and the second is that the concrete shall have the desired properties in the fresh state.

We are accustomed to categorizing concrete by strength and slump but a further description is necessary. This is currently covered by a verbal description such as 'pumpable', 'structural', or 'paving'. What is really needed is a numerical value covering this property, which is essentially the relative sandiness or cohesion of the mix. The author has devised such a parameter, which he calls the Mix Suitability Factor (MSF) (section 3.3).

It may be that mix design itself is not the most important problem. A typical large premix concrete producer will have hundreds, possibly thousands, of mixes available. What is needed is a system of mix maintenance to enable all these mixes to be kept tuned to satisfy specifications at the lowest cost as material properties, weather conditions and client requirements vary, and even as it becomes necessary or advantageous to substitute different materials.

Under 'philosophy of mix design' it is also appropriate to consider the American description of the process as 'mixture proportioning'. *Webster's Collegiate Dictionary* (to take an American source) gives to proportion as 'to adjust in proper proportion or relation, as to size, quantity or number; to adjust the proportions of; to divide into or distribute in proportionate parts'. In no way does this, in the author's opinion, come close to covering the selection of the appropriate detailed (as opposed to specified) properties of a concrete to be used for a particular purpose and the selection of the most advantageous combination of materials to provide those properties.

It is arguable that what the computer does is to proportion a mixture, but the instructions issued to the computer are much more appropriately described as design. To quote Webster again: to design is 'to form or conceive in the mind; contrive; plan; contemplate; intend; to have intentions or purposes'. It is the author's earnest intention that the current volume should assist the reader to design concrete mixes rather than merely proportioning mixtures.

1.2 DESIGN CRITERIA

The design of concrete mixes has been made much more complex by the availability of many different cementitious materials (normal, high early strength, sulphate resisting and low heat Portland cements plus fly ash, blast furnace slag and silica fume) as well as myriad admixtures to reduce water requirement, entrain air, accelerate or retard setting, and reduce permeability or shrinkage.

It may help the reader to start with the old-fashioned idea that concrete consists of cement, coarse aggregate, fine aggregate and water. Historically the problem of mix design has been seen (if as other than using nominal proportions) as to select suitable aggregates and determine their optimum relative proportions and the cement requirement to produce a given strength at a given slump.

Early investigators tended to be concerned with how to define and produce ideal concrete. Frequently this meant trying to determine the ideal combined grading of the coarse and fine aggregate and therefore how these materials should be specified and in what proportions they should be combined.

Today, our consideration should be: firstly, what aggregates are economically available; secondly, what properties should the concrete have; and thirdly, what is the most economical way of providing these required properties.

Spelling this out more clearly:

1. Use available aggregates rather than searching for ideal aggregates.
2. Recognize that there is no concrete ideal for all purposes, but rather define what is required for a particular purpose.
3. Understand that there will be competition based on price.

In this new edition, it must be pointed out that grading is again receiving attention, especially for high strength concrete. However, now it is the grading of the fines in the mix, including the cement, which is seen to be important.

The use of slump as a criterion requires comment. Slump certainly does not accurately assess the relative workability of two different mixes. However, it seems likely to survive because it is a good way of checking the relative water content of two nominally identical mixes, as in the case of successive deliveries of the same mix. It may be that the combination of slump and the author's MSF does have some of the absolute validity that slump alone lacks. The concept of an 'equivalent slump' may extend that validity (section 12.1.3).

1.3 USE OF STRENGTH AS A BASIS

As already noted, mix design has generally meant designing a mix to provide a given strength. While strength is often not the most important requirement, the reason for its use as a criterion is clearly shown by the step following its selection in most mix design procedures. This is to convert the strength requirement into a water/cement (w/c) ratio. The relationship between strength and w/c ratio is generally attributed to Abrams in the USA (Neville, 1995). Actually, Feret in France in 1896 (Neville, 1995) preceded him and proposed a more accurate proportionality, that between strength and the ratio of cement to water **plus voids**. It may be that accuracy was not the important thing, partly because the w/c ratio itself was arguably more important than the strength it was assumed to represent, and partly because the simplicity of the concept was as important as its accuracy.

While the concept of w/c ratio is simple, and its **approximate** implementation is also simple, it would be a difficult criterion to enforce by testing. A case could be made that the most accurate way of establishing the w/c ratio of a given sample of production concrete (of which the w/c ratio *versus* strength relationship has already been established) is to test its strength.

So, much of the importance of strength is as a test method and a means of specification for w/c ratio.

A primitive way of designing a mix, assuming that only one fine and one coarse aggregate are involved, would be to make a mix of any reasonable proportions (say 1:2:4) and fairly high slump (say 100 mm). If a sample of this concrete were heavily vibrated for several (say, 15) minutes in a sturdy container (such as a bucket, not as small as a cylinder mould) then any excess of either coarse aggregate or mortar would be left on top. If the top half were discarded, then the proportions of the bottom half would be a reasonable guide to the desirable sand percentage to use. This is a useful exercise for students since it illustrates the concept of filling the voids in the coarse aggregate with mortar and demonstrates that an ideal mix cannot be over-vibrated once it is fully compacted in place (in that the remaining concrete will not further segregate however long it is vibrated).

Before continuing to examine the development of methods of mix design, the reader's attention is drawn to Chapters 7, 8 and 9 on concreting materials. These chapters are not intended to provide sufficient background for the inexperienced reader (who should also consult the standard texts) but do make clear the extent to which the author's views differ from or extend those likely to be encountered elsewhere.

1.4 CONSIDERATIONS OTHER THAN STRENGTH

1.4.1 Durability/permeability

These two properties are considered together because to a large extent permeability reduction is the main factor in achieving durability.

Chemical attack

Readers should look elsewhere (Neville, 1995; Biczok, 1964; ACI SP47, 1975) for detailed information on attack by a range of aggressive substances and for details of the mechanisms of chemical attack. Here we shall consider attack by sulphates, chlorides and seawater (which combines the two) and deterioration by alkali-silica reaction.

The most readily attacked components of hydrated cement paste are those resulting from tricalcium aluminate (C_3A) and the calcium hydroxide which is liberated in substantial quantities during hydration. Sulphate resisting Portland cement is cement in which the C_3A content is subject to an upper limit (usually

5% but differing in different countries). The C_3A content in low heat Portland cement is also limited (for different reasons, usually 3 to 6%) and so it will have similar sulphate resistance.

Sulphate resisting or low heat cements may be expensive and inconvenient to use as they require an additional silo and it should be noted that concrete made with them is actually less resistant to chloride penetration. It has been shown (Kalousek *et al.,* 1972) that the use of a proportion of fly ash and/or blast furnace slag can provide a similar degree of sulphate resistance while still providing chloride resistance, lower permeability and resistance to leaching as well as often reducing cost below that of ordinary Portland cement (OPC). Slag cements in particular are extremely suitable for marine applications, providing at least a 60% slag content is used.

Fly ash produces its beneficial effects by combining with the calcium hydroxide, converting it to more durable calcium silicates, and by reducing permeability through denser packing and reduced water requirement.

Alkali-silica reaction is a disruptive expansion of the cement matrix arising from the combination of alkalis (usually, but not necessarily solely, from the cement) and reactive silica (usually in the coarse aggregate). While relatively rare, the phenomenon can be totally disastrous when it does occur. There are three possible strategies to limit its occurrence. One is to avoid total alkalis (sodium and potassium) in the cement exceeding 0.6% calculated as Na_2O. Another is to test the aggregate for reactivity. A third possibility is to provide an excess of reactive silica in the form of fly ash, silica fume, or natural pozzolan so as to consume any alkali present in a non-expansive surface reaction product.

1.4.2 Permeability

There are three avenues by which water can penetrate concrete:

1. Gross voids arising from incomplete compaction, often resulting from segregation.
2. Micro (or macro) cracks resulting from drying shrinkage, thermal stresses or bleeding settlement.
3. Pores or capillaries resulting from mixing water in excess of that which can combine with the cement, i.e. water in excess of 0.38 by mass of cement.

Gross voids may be regarded as too obvious a cause to be included. However, they are worth mentioning because they may be made more likely by action which would otherwise reduce porosity, i.e. a harsh, low slump mix will have a low water content and a richer mortar (higher cement/sand ratio) than a sandier mix of equal strength. Obviously a low permeability concrete must be such that it will be fully compacted by the means available. It must not depend on unrealistic expectations of workmanship.

Thermal stresses are the result of heat generated during hydration, which is an exothermic reaction (section 1.4.5).

Water occupies 15 to 20% of the total volume of fresh concrete and, when the w/c ratio exceeds 0.38 by mass, not all of this water can be consumed in the hydration of the cement. To the extent to which the voids left by the excess water are discontinuous, they will not provide easy passage for water. This explains the tendency for graphs of permeability against water content, w/c ratio, etc., to rise slowly for a while and then suddenly sweep upwards almost asymptotically at the point at which the voids became interconnected (Fig. 1.1)

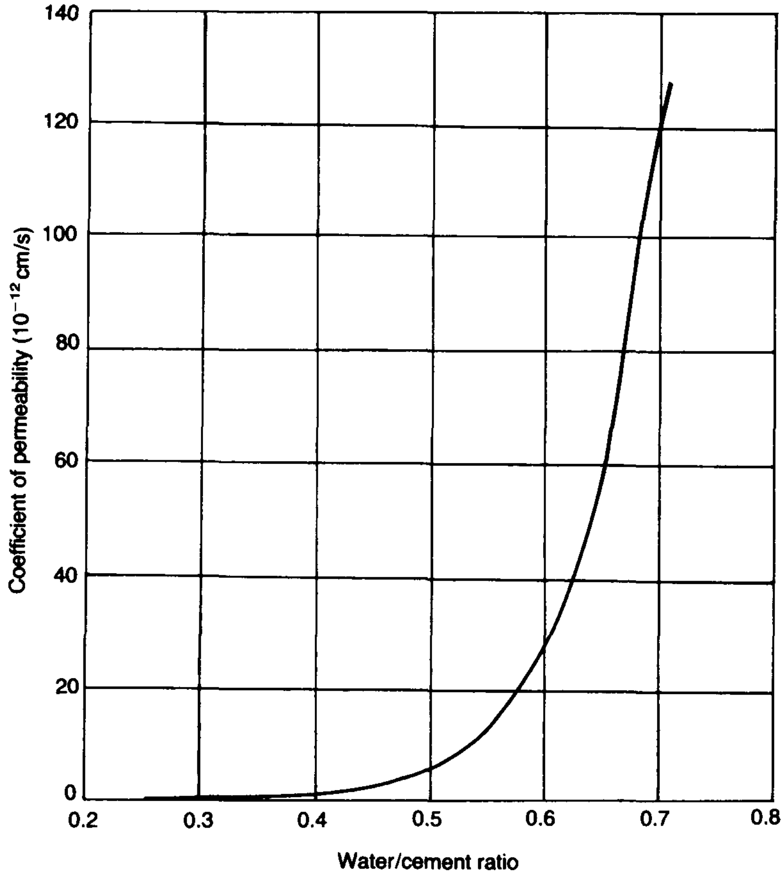


Fig. 1.1 Relation between w/c ratio and permeability.

The latest packing theories of mix design have demonstrated that close attention to the packing of fine material of cement size and smaller can reduce total void space in the paste fraction, especially when accompanied by superplasticizers.

The total amount of pore space is not the only factor determining permeability. Another important factor is the distribution of the pores and their discontinuity. Bleeding is a source of continuous or semi-continuous pores. Bleeding is initiated by the settlement of cement particles in the surrounding mixing water, after compaction in place. This tends to leave minute pockets of water under fine aggregate grains. There may be enough water to allow the fine aggregate grains to settle slightly and the water to escape around them and rise up through the concrete. The process occurs on a larger scale under the coarse aggregate particles and eventually the whole mass of the concrete settles slightly, leaving a film of water on the surface. The process can happen very gently without having a great effect on the concrete properties. If bleeding is severe, the rising water tends to leave well defined capillary passages and it is then known as channel bleeding. Water penetration of the hardened concrete is obviously greatly facilitated by both the vertical channels and the voids formed under the coarse aggregate and even fine aggregate particles.

Reduction of permeability can be effected either by avoiding bleeding in the first place or by blocking the channels after formation. Pore blocking after they have formed takes place as cement continues to hydrate and extends gel formation into the pores. This requires the concrete to be well cured and is greatly affected by w/c ratio (Table 1.1 and Fig. 1.2). Another means is to line the pores in the concrete with hydrophobic material. Such materials are marketed as 'waterproofing admixtures' and may be soapy materials such as stearates or materials such as silicones. Hydrophobic material may provide a temporary benefit but lose its effectiveness in the longer term.

Factors affecting bleeding are:

1. Amount of fine material (including cement, slag, fly ash, silica fume and natural pozzolans).
2. Air entrainment.
3. Water reduction through admixtures or lower slump.
4. Continuity of grading (especially including fine aggregate grading).
5. The use of methyl cellulose or other gel-forming admixtures (mainly in grouts).
6. Retardation, whether due to low temperature or chemical retarders, delays gel formation and so extends the period of bleeding.

Table 1.1 Time taken to achieve discontinuity of voids

Water/cement ration	*Age of concrete at which capillary pores become blocked*
0.40	3 days
0.45	7 days
0.50	14 days
0.60	6 months
0.70	1 year
over 0.70	infinity

Essentially, the mortar in concrete consists of a mass of particles saturated with water that is trying to escape: the more water there is, the more will escape by bleeding. The better the particles pack together, the more difficult it will be for water to pass through the mass. Cement, slag, fly ash, entrained air, rice hull ash and silica fume (in increasing order of effectiveness) are good inhibitors of bleeding. Silica fume is the most effective inhibitor of bleeding. It is many times finer than cement and particles of it fill the interstices between the cement particles. Small amounts (as little as 10 to 30 kg/m^3) are sufficient to prevent bleeding almost completely. It should be noted that the effectiveness of the fume is greatly reduced if it is incompletely dispersed. Essentially, this means that silica fume should always be used in conjunction with a superplasticizing admixture and given adequate mixing time.

It should be noted that eliminating or greatly reducing bleeding can create problems with evaporation cracking. Such concrete may require careful attention to preventative measures such as the use of aliphatic alcohol evaporation retardant or polythene sheeting, mist sprays etc.

1.4.3 Pumpability

To some extent pumpability can be equated with resistance to bleeding. A blockage (pumping failure) occurs when the mix segregates under pressure. Any chance resistance to movement along the pipeline (badly

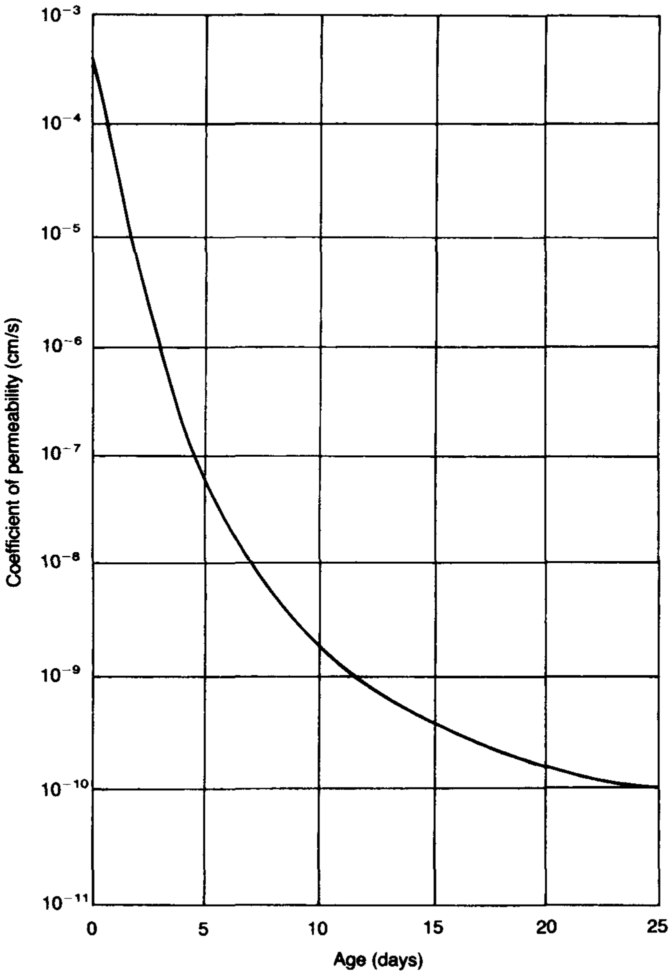


Fig. 1.2 Reduction of permeability with curing.

fitting joint, failure to completely clean the pipes, or even just a bend) causes an increase in pressure and can result in mortar being forced through jammed coarse aggregate leaving a stone jam. It can also result in water being squeezed out of the concrete, either to move further along the pipe and out through a leaking joint or into incompletely saturated coarse aggregate. A particular risk arises when pumping is not continuous. Concrete may pump quite readily when fresh, but if it bleeds during any small delay, restarting may prove impossible. This may cause particular problems when attempting to 'suck back' excess concrete in a pipeline (which could be 50 or more storeys high) at the end of a pour.

Segregation of mortar occurs at a gap in the coarse aggregate grading or, more frequently, between the coarse and fine aggregate gradings. Bleeding of water or segregation of cement paste can occur at a gap in the fine aggregate grading. Pumpability is best assured if all sieve sizes (with the exception of the largest, of which there will be more) are present in approximately equal quantity. In particular any two consecutive sieves should together retain at least 8% of the total volume of the concrete for good pumpability.

Concrete has been pumped to the top of the world's tallest building, Petronas Towers in Kuala Lumpur, Malaysia. The inclusion of silica fume and a high-solids superplasticizer were considered to be essential to achieve this.

1.4.4 Shrinkage

Shrinkage is an important (potentially deleterious) property of concrete. Cracking results when shrinkage is restrained and differential shrinkage can give rise to such problems as curling of slabs and sloping floors in multistorey buildings (differential shrinkage of central core and peripheral columns). Shrinkage also causes stress losses in prestressed members and failure of joints which have not been designed to cope with the amount of shrinkage experienced.

Shrinkage generally means **drying shrinkage** (although further shrinkage takes place during carbonation) and is caused by the contraction of hardened cement paste when it loses water. The contraction is resisted by the aggregates in the concrete, especially the coarse aggregate. It is therefore to be expected that shrinkage will be largely dependent on the total amount of water in the original mix and on the elastic modulus of the coarse aggregate and its proportion, although the latter two are relatively minor effects except in the case of lightweight aggregates. Many coarse aggregates are subject to moisture movement which is obviously directly passed on to the concrete. With some aggregates this effect can be as large as that due to the use of an oversanded pump mix. Another factor which influences shrinkage is the gypsum content of the cement. Gypsum is calcium sulphate and the SO_3 content of cement is limited by most national codes, to avoid the risk of excessive expansion. Not all the SO_3 is in an active form, so these limits may be below the optimum level for shrinkage reduction and admixtures are available to supplement it. The material added produces calcium sulpho-aluminate (ettringite) and it can produce not merely reduced shrinkage but even expansion or 'chemical prestress'. At least in the USA, shrinkage compensating or even prestress producing cements are available, having such a material interground with normal cement.

It should be noted however that the action of 'shrinkage compensators' is not in fact to directly inhibit the occurrence of drying shrinkage. Rather they produce an expansion which continues so long as the concrete is kept wet and the normal drying shrinkage then occurs when it is permitted to dry. The mechanism is for the expansion to be resisted by reinforcing steel so as to produce a precompression in the concrete. Subsequent shrinkage then merely dissipates this compression without producing tensile stresses. There is some tendency for a threshold effect in which inadequate doses of shrinkage compensator, or inadequate curing, produce an expansion tendency that is entirely dissipated in creep and has little or no effect on the final situation.

A particular form of shrinkage known as 'autogenous shrinkage' occurs in high strength concrete. This is caused by the consumption of water by hydration ('self dessication') and relates to mixes having a w/c ratio below about 0.38. Such shrinkage occurs much more quickly than normal drying shrinkage and cannot be prevented by measures such as curing compounds or polythene sheeting. The only effective recourse is actual water curing since the concrete needs to take up additional water.

1.4.5 Heat generation

Large masses of concrete generate substantial quantities of heat during hydration and can cause temperature rises in excess of 50°C in some circumstances. Since differential temperatures of 20 to 25°C between different parts of a mass of concrete may cause stresses sufficient to initiate cracking, action is necessary to avoid this. Sometimes very high costs have been incurred by using specially produced flake ice in (or instead of) mixing water, by casting cooling coils (in which brine is circulated) into the concrete, or by injecting liquid nitrogen into the mixing truck as a means of lowering the concrete temperature. It should be noted that normal crushed ice is not suitable for direct addition to concrete. This is because it can take some time to melt and so make slump control extremely difficult. In the extreme, all the ice may not have melted prior to final compaction into place of the concrete. This has been known to produce low strengths in test cylinders.

It is certainly true that the higher the supply temperature of concrete, the more rapidly it will generate heat. The use of cooled concrete is therefore beneficial in spreading the heat generation period, allowing more time for heat to escape and therefore reducing peak temperatures. It also reduces the necessary water content and therefore the necessary cement content. In tropical climates it is generally worthwhile to take all available **inexpensive** measures to produce concrete at a reduced temperature. These may include shading aggregate stockpiles and perhaps sprinkling them with water. For a permanent installation it probably also includes evaporative cooling of the mixing water. However, whether the expense of refrigerated cooling, or the addition of ice or liquid nitrogen, is really justified is more doubtful and depends on the circumstances and alternatives. It is generally more economical firstly to reduce heat generation to a minimum and secondly to insulate the outside of the mass so that differential temperatures are reduced by allowing the whole mass to heat up together.

Essentially heat generation can only be reduced by reducing the amounts of C_3A (tricalcium aluminate) and C_3S (tricalcium silicate) present. Some amelioration is possible at a given strength by reducing water content by either mix design or admixtures and so reducing the amount of cement necessary. Silica fume has been used in this way since, although it generates as much heat per kilogram as cement, and generates it just as quickly, it replaces approximately three times its own mass of cement. Low heat cement produces less heat than OPC and also spreads its heat generation over a longer period, so allowing it to dissipate better. It should be noted that, while low heat cement may be sulphate resisting, sulphate resisting cement is not necessarily low heat. Both cements limit the amount of C_3A, which is the worst generator of heat per unit mass, but there is no limitation of C_3S in sulphate resisting cement and this in fact is the largest source of heat, being present in much larger proportion than the C_3A.

If fly ash is available, it may be both more economical and more effective to use fly ash to reduce cement content than to use low heat cement. This is especially the case since the ash will also reduce bleeding settlement and will directly reduce permeability. High early strength is rarely a requirement for mass concrete, and in any case the generated heat will accelerate strength gain. It may therefore be possible to use a very high percentage of fly ash (even as high as 50%) in a mass concrete foundation. However, fly ash (unless type C, high calcium ash) requires calcium hydroxide released by the cement to form cementitious compounds and cannot be used in as high a proportion as slag.

Blast furnace slag is also an excellent way of slowing heat generation in normal conditions, but caution should be exercised when very large sections are involved. Although the **rate** of heat generation is reduced, the **amount** of heat generated may even increase. If the heat cannot escape, a higher final temperature may be reached. Some references suggest that in excess of 70% of slag must be used to give a reduction in peak temperature under adiabatic conditions but other investigators find benefit from smaller amounts. It should be emphasized that adiabatic conditions are only approximated in very large masses of concrete, such as a raft foundation more than three metres thick. Whilst smaller masses of concrete may generate substantial heat, the heat may be able to escape quickly enough for the slower heat generation of slag blend cements to result in a substantially lower peak temperature than with OPC.

1.5 SELECTION OF DESIGN STRENGTH

Having agreed that a principal criterion for the design will be compressive strength, it is necessary to decide how that strength should be selected.

The mix design process will deal with a **target average strength**. The average strength selected must take into account:

1. The degree of variability anticipated.
2. The degree of certainty of avoiding rejection required.
3 Any early age strength requirement.
4. The required durability.

1.5.1 Variability

The reader will find further detail on variability in Chapter 10 but essentially variability is to be assessed in terms of **standard deviation**, which is a measure of the spread of results assuming concrete strength to be a **normally distributed variable**. The author has found this assumption to be well justified in practice except that only about half the results theoretically expected to be below the mean minus 1.64 σ usually occur.

The formula used is

$$X = F + k\sigma$$

where X=required average strength

F=specified strength

σ=standard deviation

k=a constant depending on the proportion of results permitted to be below *F*.

In the USA the 'permissible percentage defective' is usually 10% giving a *k* value of 1.28.

In most of the rest of the world the percentage is 5% giving a *k* value of 1.645 (which in the UK is rounded to 1.64 and in Australia to 1.65).

Values of σ, the standard deviation, can range from less than 2.0 MPa (290 psi) to more than 6.0 MPa (870 psi) so that the required target average strength can vary by 6 MPa (870 psi) or more according to the degree of control achieved (Fig. 1.3).

Some quite experienced persons, including a number of ACI committees, believe that **coefficient of variation**, which is standard deviation divided by average strength, is a more appropriate measure of variability than standard deviation itself. There is certainly an increase in testing error at higher strengths, which adds to apparent variability. However, having personally produced very high strength concrete at very low variability, the author is not in favour of coefficient of variation and believes that those who favour it are deluding themselves as to the degree of control achieved on their high strength concrete. The truth lies somewhere between constant standard deviation and constant coefficient of variation for high strength concrete and everyone is therefore entitled to their own choice. However, the author has routinely analysed, month by month, many thousands of test results from many different suppliers, on many different projects, and in several countries. These results, from any one plant, almost invariably show very little difference in standard deviation in grades from, and including, 20 to 40 MPa.

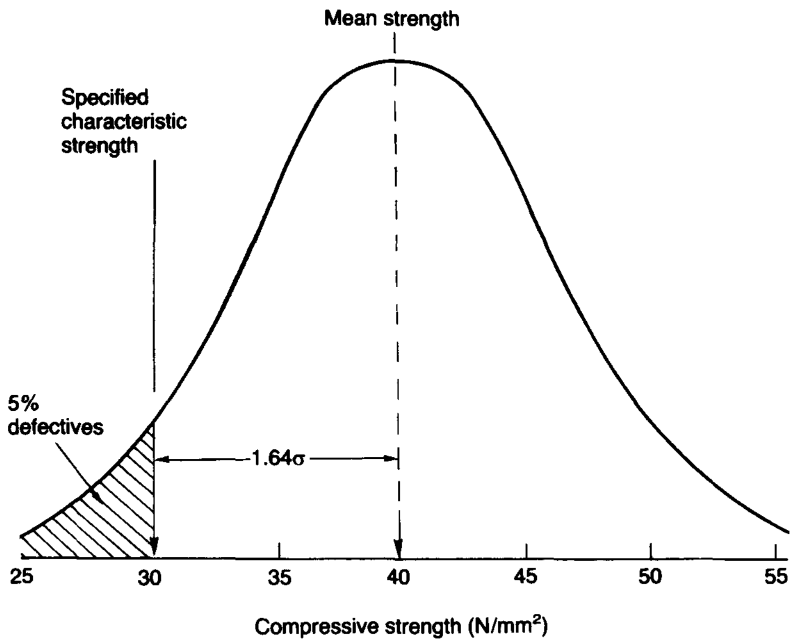


Fig. 1.3 The normal distribution.

1.5.2 Safety margin

The concrete producer would face a 50% likelihood of his concrete being adjudged defective if it was exactly of the intended mean strength and was perfectly assessed (Chapter 10). Therefore he may decide to add a safety margin of say 1 or 2 MPa (150 or 300 psi) to avoid such problems. However, the cost of such an additional margin would reduce his competitiveness and some of the expenditure may be more usefully directed to reducing variability. In the UK it is normal to use a target strength two standard deviations above the specified strength. This is all the more onerous since standard deviations of 4 to 6 MPa are apparently normal there, compared to 2 to 3 MPa for normal strength concrete in Australia. Thus mean strengths are typically 10 MPa above specified strength in the UK and only 5 MPa higher in Australia.

The cost of a safety margin may be unattractive to the producer, as being a large proportion of his profit margin. However, the cost of such a margin may be close to negligible compared to the total cost of the structure and the owner of the structure may be well advised to allow a margin by specifying a higher grade of concrete than strictly required (section 12.4). In the UK all premix suppliers have joined together in QSRMC (Quality Scheme for Ready Mixed Concrete). Amongst other advantages, this avoids any competitive disadvantage in the use of a high strength margin.

1.5.3 Early age strength

A concrete of higher 28-day strength will produce a larger percentage of that strength at an early age and also generate more heat giving addi tional maturity at the early age. (This is not necessarily so if the higher strength concrete contains more pozzolan or uses low heat cement). Therefore using a higher grade than really needed for 28-day strength can show a double benefit in its effect on earlier prestressing, stripping or de-propping.

If inadequate moist curing is anticipated, this may effectively mean that the concrete should be designed to develop its strength early since it will not continue to increase in strength once dry. Similarly in cold countries, where concrete needs to be protected until it reaches a critical strength, it may be necessary to design for this criterion rather than 28-day strength.

1.5.4 Durability

As discussed in section 1.4.1, durability is largely ensured by the use of suitable cementitious materials and a sufficiently low w/c ratio. It was pointed out in section 1.3 that specifying a strength is very similar in effect to specifying a w/c ratio. If the strength required to ensure the desired w/c ratio is higher than that required from the structural design viewpoint, then the higher of the two must always rule. The use of entrained air or of special cementitious materials is not ensured by specification of a strength level and must be specified in addition to the strength.

A strength of 30 or 32MPa (say 4500 psi) may be the minimum required to provide good durability of reinforced concrete in external structures. In aggressive circumstances, this may need to be increased to 40 or 50 MPa (6000 to 7000 psi).

1.6 THE CUBE/CYLINDER RELATIONSHIP

The world is divided as to whether it is better to assess concrete strength by cube or cylinder specimens. The UK, much of Europe, the former USSR and many ex-British colonies use cubes, the USA, France and Australia use cylinders.

The use of large aggregate concrete, except for special uses such as dams, is becoming rare. For high strength concrete, aggregate with a maximum size of more than 20 mm (¾ in) is a disadvantage and for very high strengths a smaller size still, 10 to 14mm (⅜ to ½ in) gives better results. Therefore previously used specimen sizes of 150 mm (6 in) cubes and 150 diameter×300 mm long cylinders can be replaced by 100 mm cubes and 100×200 mm cylinders. Some researchers consider that the smaller specimens will give higher strength (up to about 5% higher) and greater variability, others have found **lower** variability with the smaller specimens (Ting *et al.,* 1992), but the differences are not sufficient to concern us unless they affect a comparison between different laboratories.

What concerns us here is the cube/cylinder ratio. The British Standard BS 1881 nominates this ratio as 1. 25 for all circumstances but this is not the author's experience, which is that the ratio varies from over 1.35 to less than 1.05 as strength increases. A formula giving results in accordance with the author's experience, but not claimed to be thoroughly established, is:

$$\text{Cube strength} = \text{cylinder strength} + 19/\sqrt{(\text{cylinder strength})} \qquad \text{or}$$

$$\text{Cylinder strength} = \text{cube strength} - 20/\sqrt{(\text{cube strength})}$$

where cube and cylinder strengths are both in MPa or N/mm^2.

ISO Standard 3893–1977(E) gives the conversion table shown in Table 1.2.

1.7
FLEXURAL AND TENSILE STRENGTH

The strength of concrete is generally taken to be its compressive strength but, for very large quantities of concrete used in concrete roads, airfield paving, and the like, what really matters is the flexural strength. It is important to be clear about the relationship between this strength and tensile strength. The flexural test uses both a special test specimen and a special testing machine (or a special fitting to a compression testing

Table 1.2 Cube/cylinder strength conversion

Concrete grade	*Compressive strength at 28 days MPa (N/mm)*	
Cylinders 150 mm dia.×300 mm	*Cubes 150 mm×150 mm*	
C 2/2.5	2	2.5
C 4/5	4	5
C 6/7.5	6	7.5
C 8/10	8	10
C 10/12.5	10	12.5
C 12/15	12	15
C 16/20	16	20
C 20/25	20	25
C 25/30	25	30
C 30/35	30	35
C 35/40	35	40
C 40/45	40	45
C 45/50	45	50
C 50/55	50	55

machine). There are differences between different national codes, but the test specimen is normally a beam of either 100 mm or 150 mm square cross-section and long enough to test on a span three times its depth. The 100 mm beam is suitable for maximum aggregate sizes up to 20 mm and the 150 mm beam should be used for larger sizes of aggregate.

The beams are usually required to be tested under either a central point load or third point loading (Fig. 1.4). In either case, failure is initiated in direct tension at the extreme bottom face in the test, and this is usually required to be a side face in casting so that both the top and bottom faces in testing will be smooth moulded faces to provide an even bearing for the loading rollers. In centre point loading, the stress will be a maximum immediately below the central loading point. The occurrence of a defect at other locations along the beam may therefore not cause a reduced failure load. In third point loading, the bending moment, and therefore the bottom face stress, is essentially constant over the middle third of the span. Failure is therefore likely to occur at the weakest point of the middle third. It is not obvious which is the better test. The third point loading puts a much larger amount of concrete under effective test. This may be considered fairer, and more likely to give a true result. However, if the failure is seen as at all likely to be at a defect in the beam which is of a type not anticipated to occur in the actual slab or structure, then it may be better to get a point value strength which is less likely to be affected by such a defect. It will be pointed out later that any consideration of flexural strength must take into account the likely scatter of test results, i.e. the test has a higher within-sample variability than either compressive or indirect tensile tests.

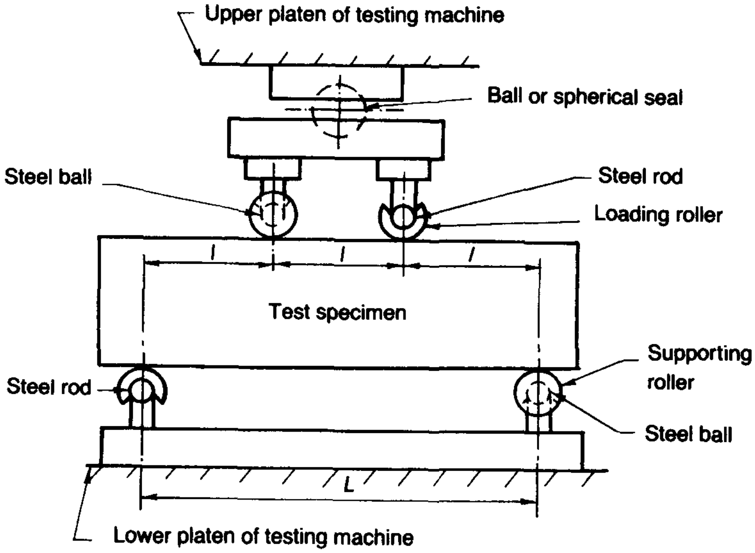


Fig. 1.4 Third point loading test.

The flexural stress at failure is calculated on the basis of an assumed triangular stress distribution. This is not what happens in practice (Fig. 1.5). The extreme fibres exhibit plastic flow as they near failure, and so distribute load to higher layers of concrete. This increases the total load at failure and so gives an inflated value of tensile strength (i.e. the flexural strength) which is calculated ignoring this effect. It can be seen that the apparent strength could be influenced by the rate of loading. Failure in tension takes time to occur, if the load is not increased above the minimum needed to cause eventual failure. This would cause the beam to fail at a lower load if tested very slowly. The strength level, the age, and the condition (wet or dry) at test also have a bearing on the ability of the concrete to exhibit plastic flow. Thus concrete of a higher strength grade, older concrete, and dry concrete, is more brittle than their converses.

It has already been noted that flexural or tensile strength tests tend to be more affected by defects or imperfections than do compressive strengths. The imperfections may arise in manufacture or in handling. Small areas of honeycombing or segregation are one kind of possibility. A different possibility is that a failure to maintain the specimens in a saturated condition may permit the development of microcracking. A microcrack in a bottom face location will have a much larger effect on flexural strength and tensile strength than it would on compressive strength.

A direct measurement of tensile strength has been obtained on a research basis (by casting specially shaped test specimens or using various types of friction grip) but is not generally practicable as a control test. Research work has also been done on applying a tensile stress by applying an internal gas pressure (Clayton and Grimer, 1979). However, tensile strength is routinely measured indirectly, using the Poisson's ratio effect to generate a tensile strength by applying a compressive stress at right angles. The usual procedure is to use a cylindrical specimen placed sideways in a compression testing machine so that the compressive force is applied across a diameter. The effect is to generate, across the vertical cross-section, a substantial compressive stress immediately under the loading, but a uniform tension over almost the entire area (Fig 1.6). The test is known as the Brazil test for the double reason of its similarity to cracking nuts in a nutcracker and the part played in its initial development by Carniero, working in Brazil (although the test was also independently developed in Japan).

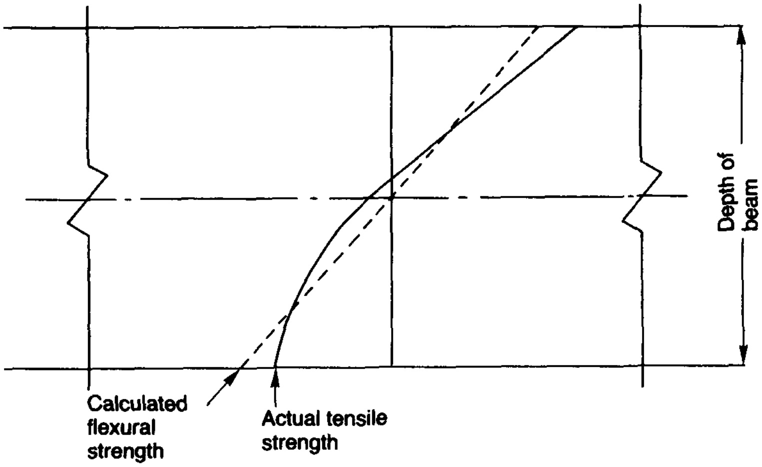


Fig. 1.5 Stress distribution in flexural test.

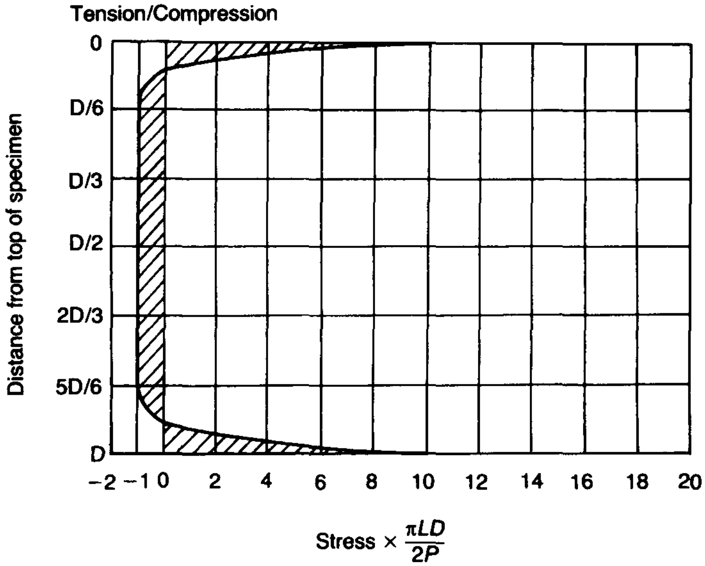


Fig. 1.6 Stresses across a diameter in Brazil test.

The tensile stress is evaluated by the formula:

$$\text{Tensile stress} = 2P/LD\pi$$

where L and D are the length and diameter of the cylinder, and P the applied force.

There is no universally agreed relationship between flexural, tensile and compressive strengths. Indeed there would be little interest in any other test than a compression test if such a fixed relationship existed. The reason for taking an interest in directly measured flexural strength is the possibility that some factor may cause a significant change in the relationship. For example the possibility exists that a coating of fines on the coarse aggregate could reduce the bond of mortar to it. This may cause a large reduction in flexural strength without making as much difference to the compressive strength. More certainly, there is a reduction in the ratio of flexural to compressive strength when the coarse aggregate particle shape is changed from crushed to rounded. So concrete specified and controlled on the basis of compressive strength may not ensure a particular load-carrying capability for concrete paving.

Although not the subject of general agreement, readers with a practical rather than a research interest will find it to be a workable assumption that both flexural and tensile strength are related to compressive strength by an equation of the form:

$$\text{Flexural or tensile strength} = K \times \sqrt{(\text{compressive strength})}$$

The value of K will vary as noted above but will be of the order of 0.6 for indirect tensile strength and 0.7 to 0.8 for flexural strength.

If compressive strength does not necessarily define flexural strength, it would seem reasonable to specify and control concrete for paving (and anywhere else where flexural strength is the controlling factor) on the basis of flexural strength. This is done by many, but not all, bodies (such as roadbuilding authorities) concerned with such concrete. The case for doing this becomes a little less obvious when the author relates his experience of being able to predict 28-day flexural test results more accurately from 7-day compressive tests than from 7-day flexural tests. It is important to understand the restricted conditions to which this experience related. Firstly, the results were being analysed by computer, with the exact current average value of the coefficient K in the above equation being automatically fed back and used at all times. Secondly, the aggregates in use were being rigorously controlled. There was no change of source, shape or contamination. The experience therefore does not rule out the possibility that, if such a change had occurred, it may have been missed by the compressive test and not by the flexural test. What it does show is that the flexural test is too inaccurate to detect the small batch to batch variations still occurring in excellently controlled concrete. The test data involved (which are unpublished and not available for publication) also included indirect tensile tests. The author found that this data was intermediate in reliability between the flexural and compressive data.

As is hopefully made clear in this volume, the purpose of routine testing of concrete is to detect as quickly as possible any change in the quality of the concrete being produced/supplied, and to do so at minimum cost. The flexural test is a more expensive test to carry out than the compression or indirect tensile tests. It seems therefore that, for major paving projects such as roads and airports, the ideal is to specify the concrete on the basis of flexural strength but to control it largely on the basis of compressive strength. The initial concrete supplied would be tested intensively to ensure that the required flexural strength was being provided and also to establish the relationship between compressive and flexural strength for the particular mix. It would also be required that the relationship be reconfirmed from time to time, and particularly in the event of any changes in mix or ingredients. It seems unlikely that an event (such as contaminated aggregate) could affect flexural strength without showing any effect whatever on compressive strength. Any significant change detected in compressive strength could lead to a reconfirmation of the flexural/compressive strength relationship. It also seems extremely unlikely that anything could affect flexural strength without having an equally large effect on indirect tensile strength, so this offers an intermediate alternative.

2
Some historical and alternative systems of mix design

The reader may find a little initial difficulty in comparing historical data with current practice because they tend to be in relative proportions rather than batch weights. There is a good reason for this in that concrete used to be produced using bagged cement and volume batched aggregates. Care should be taken in interpreting old data as they do not necessarily make clear whether the relative proportions (e.g. 1:2:4) were by weight or by volume. A particular difficulty in volume batching was that sand 'bulks' or increases in bulk volume when it is damp (as it normally is). The phenomenon is due to the surface tension of water increasing the friction between particles in contact. It follows that dry sand and inundated sand will occupy the same bulk volume, that a peak value will be reached well short of the moisture content to which a sand drains, that fine sands will be more affected than coarser sands and that it will take more water to cause peak bulking the finer the sand. Bulking can be as much as a 30 to 40% increase in original bulk volume at a moisture content of the order of 6 to 8% by weight (Fig. 2.1).

2.1
1:2:4 MIXES

At one time it was common to nominate concrete as one part cement, two parts sand, four parts coarse aggregate (or 1:1:2 when stronger concrete was needed). As will become apparent later, this mix would be satisfactory only with a particular sand grading and therefore led to the specification (in the UK) of 'Class A' sand, a restricted grading envelope of sand which made good 1:2:4 concrete. On both sides of the Class A envelope was a further envelope called 'Class B' sand—sand which made reasonable but not good concrete if the 1:2:4 proportions were retained (Fig. 2.2).

In 1954 Newman and Teychenné (1954) showed that equally good concrete could be produced from the Class B sand providing the relative proportion of sand to coarse aggregate was adjusted appropriately. They proposed the division of sands into four grading zones instead of two classes. Sand as a percentage of total aggregates was to range from 40% with the coarsest (Zone 1) sand to 22% with the finest (Zone 4) sand. Zone 2 at 33% is the old 2:1 ratio and Zone 3 would require 25% of sand (Fig. 2.3).

Although Newman and Teychenné allocated sands to the four zones on the basis of percentage passing the No. 25 BSS sieve (ASTM 30, Metric 600 μm) they did indicate that specific surface would have been a preferable basis except for the difficulty of measurement (sections 2.5 and 3.4).

The author's system owes a great deal to this paper. The grading zone concept has now been dropped in favour of the BRE system (see below).

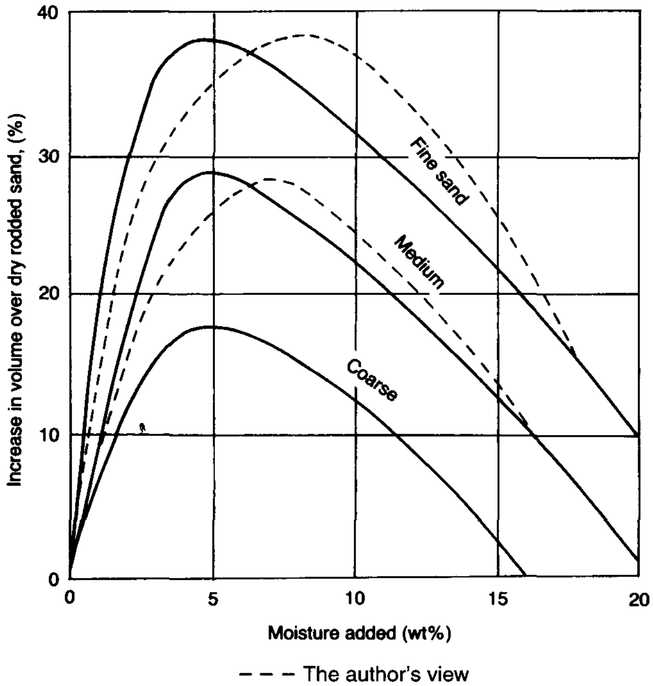


Fig. 2.1 Bulking of sand.

2.2 IDEAL GRADING CURVES

Many investigators have put forward 'ideal' grading curves, either as actual curves or as mathematical formulas. Prominent amongst them were Fuller and Thompson in the USA (1907) and Bolomey (1926) in France. Bolomey modified the Fuller and Thompson formula to include cement and to vary the grading according to the desired workability and the aggregate particle shape (section 7.1).

The weakness of the ideal grading approach is that it is rarely possible (or economical) to replicate exactly the ideal grading in the field. Also the grading may be ideal for one use but could not simultaneously be ideal for all uses.

2.3 GAP GRADINGS

There have also been many proponents of the use of gap gradings, e.g. D.A.Stewart (1951). The technique is to use a large, often single sized, coarse aggregate (often 40 mm) and a relatively fine sand. With such a combination it becomes valid to measure the voids in the coarse aggregate and provide just sufficient mortar to fill them, with a small surplus.

There is no doubt that gap-graded concrete compacts more rapidly under vibration (Plowman, 1956) and a given strength can usually be obtained more economically (at least if cement content is the only cost criterion) with a low slump, gap-graded mix. However, several factors often militate against such mixes. The first, as with ideal continuous gradings, is that suitable aggregates may not be economically available. The second is that gap-graded mixes have a strong tendency to segregate at anything more than low (say, 50 mm) slump.

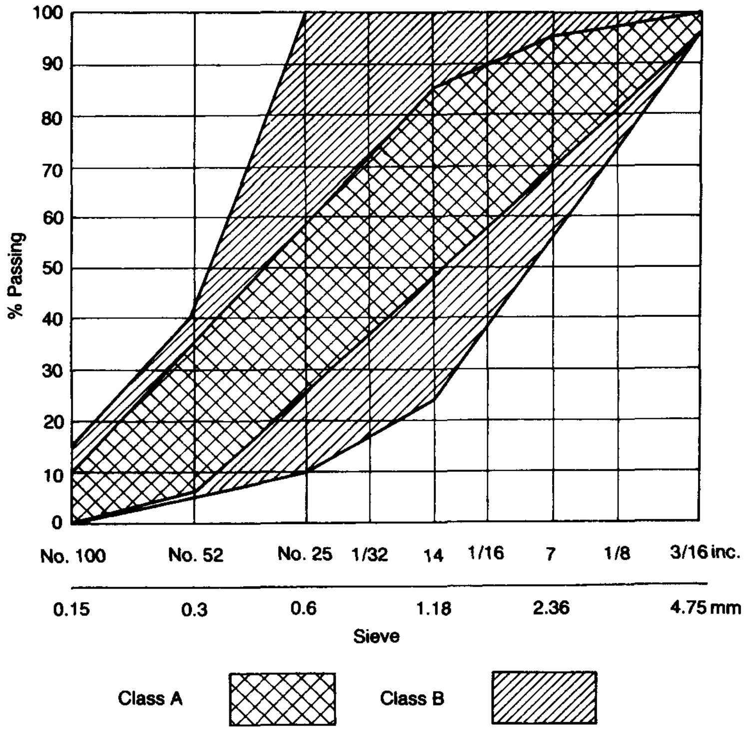


Fig. 2.2 Class A and B grading zones (BS 882:1944 *Concreting Sands*).

Although such concrete is easier to consolidate than a continuously graded mix of similar slump, it is sometimes difficult to convince workmen of this and water is frequently added with disastrous effects.

In short, gap-graded mixes can be unbeatable when used by those familiar with such mixes, and in suitable conditions, but are not to be recommended for general use.

Another property of gap-graded mixes is that, with a very stable coarse aggregate, very low drying shrinkage is attainable. This is taken to the ultimate in 'prepacked' concrete. This technique involves filling the formwork to be concreted with a large single-sized aggregate and then pumping in an appropriate mortar from the bottom up. Since the coarse aggregate is everywhere in contact, shrinkage is not possible except as aggregate moisture movement. Such concrete is very suitable for use as a foundation block for large pieces of machinery, the concrete often being placed after the machine has been set in position (vibration being unnecessary).

Exposed aggregate finishes are a matter of taste but in the author's opinion there is no more attractive finish than that obtained with heavily gap-graded concrete, i.e. a concrete with a high proportion of a large, single-sized coarse aggregate and a small proportion of a relatively fine mortar.

2.4
ROAD NOTE 4

For many years, in the 1940s, 1950s and beyond, this was the accepted UK system. It offered tabulated data based on an extensive trial mix series at the Harmondsworth Road Research Laboratory (Road Research Laboratory, 1950).

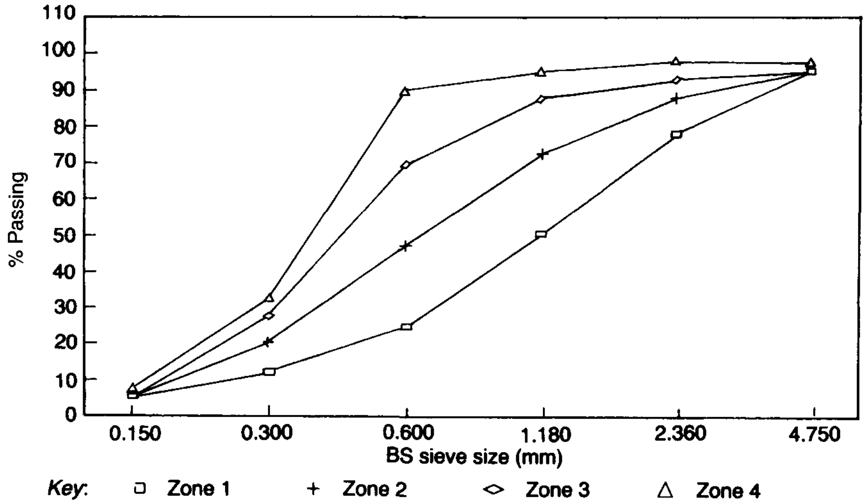


Fig. 2.3 British sand grading zones (mean values).

Four alternative gradings were included so that the user could choose to use a harsher or sandier mix. These 'type grading curves' are still used as noted below.

The tabulated data not only covered four gradings but also three different maximum sizes of aggregate (40 mm, 20 mm and 10 mm) and two different particle shapes. The system was purely empirical and so could not be readily adapted when admixtures came into use and cement properties changed. As coarse sand became less readily available, it became harder to match the grading curves. The fact that the system dealt with aggregate/cement ratio rather than batch quantities per cubic metre (or per cubic yard) became inconvenient with the rise of ready mixed concrete.

However the tabulated or graphed gradings have long survived the demise of the actual system, being generally used (including by the author) as a frame of reference as to what constitutes harsh and soft gradings (Fig. 2.4).

See Chapter 7 for further detail of sand grading zones.

2.5
BRE/DOE SYSTEM

The British replacement for Road Note 4 was *Design of Normal Concrete Mixes,* published in 1975 by the UK Department of the Environment (DOE) (i.e. the Building Research Establishment and the Transport and Road Research Laboratory). The system is attributed to D.C.Teychenné, R.E.Franklin and H.C.Erntroy, and clearly owes much to Teychenné's work on specific surface. It relates the percentage of a fine aggregate to its grading and the w/c ratio and accurately copes with a very wide range of fine aggregates. It is also up to date in terms of the relationship between w/c ratio and strength and copes well with adjustments to this relationship and to water requirement on the basis of trial mixes. The latest (1988) version (DOE, 1988) does allow for air entrainment and the use of fly ash and ground granulated blast furnace slag (ggbfs) but does not provide a choice of harsher or softer mixes or readily give an accurate yield or density. This version bears the BRE logo on the cover so the system may be found described as either the DOE or the BRE system.

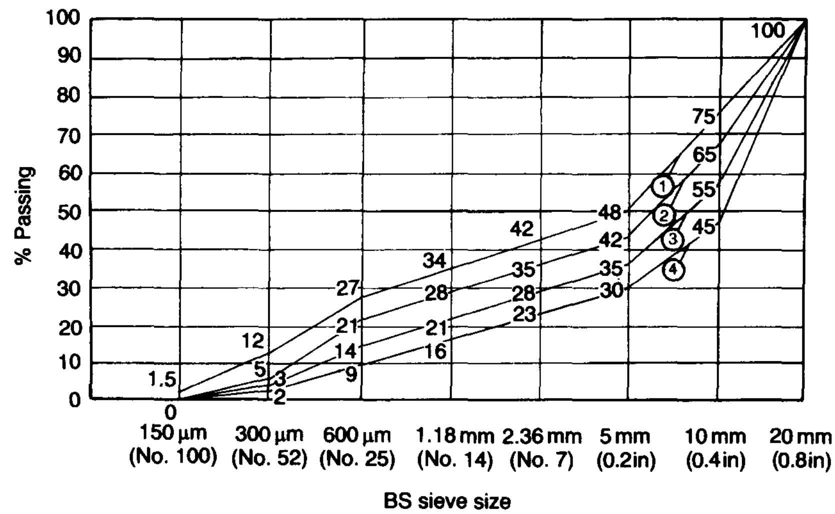


Fig. 2.4 Road Note 4 reference gradings for 0.75 in (20 mm) maximum size aggregate.

The basis of this system in concrete technology is almost identical to that of the author's Conad Mixtune system, even though the design process is completely different. It is, therefore, interesting to examine the techniques used in some detail and assess the relative advantages and disadvantages of the two approaches.

The most obvious and major difference is that the DOE system is presented for manual operation using tabulated and graphical data whilst the author's system is computerized. However, there is no reason why the DOE system should not be computerized and the author's system could be presented manually. If these changes were made, the DOE system would work a little more accurately than it now does, in interpolating values from graphs and tables. The author's system, as seen in Chapter 3, would require a substantial amount of calculation or the provision of design aids in the form of graphs or tables. This clearly illustrates the point that computerization allows an elaboration of the technological basis without detriment to the ease of use.

It is possible that, given a brief to produce a computerized system, the DOE team would have produced something very similar to the author's system. However, if the author were required to produce a new manual system, he would graft the specific surface technique onto the ACI bulk density system (section 2.7) and would still have a more elaborate water prediction system.

Teychenné (together with A.J.Newman) (Newman and Teychenné, 1954) was essentially the person from whom the author learned the specific surface theory. However, although the theory is still the fundamental basis of both systems, the author and the DOE team have gone in different directions from using exact specific surface. The 1975 DOE system used sand grading zones and the 1988 version substitutes percentage passing the 600 micron sieve as their simplified approximation. (Obviously this cannot be as accurate as true specific surface but was selected as a balance between simplicity and accuracy.)

The author found that even true specific surface did not give a sufficiently accurate prediction of water requirement and therefore originated his 'modified specific surface' (MSF, section 3.3). Even in a manual system, the additional effort involved is minuscule and certainly does not justify the DOE simplification. It may be concluded that the DOE simplification was considered worthwhile because true specific surface still did not provide great accuracy so that little was lost by the simplification. It may also be that the

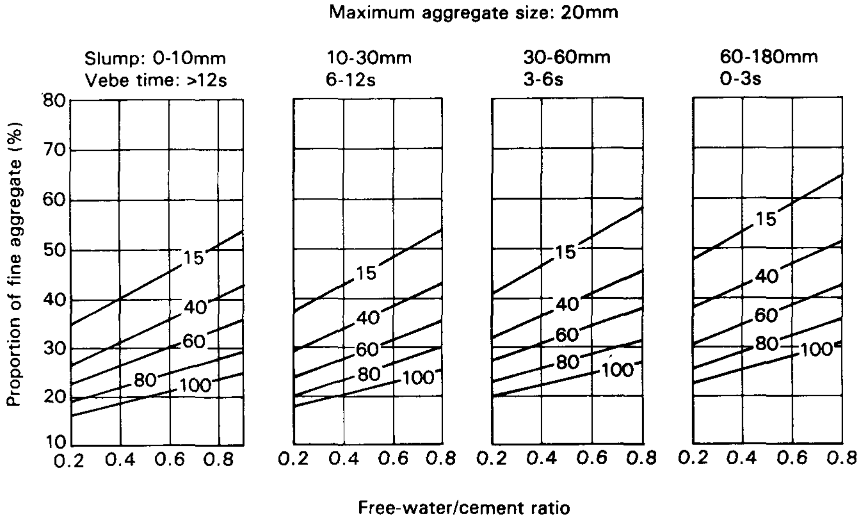


Fig. 2.5 Selection of fine aggregate %.

simplification was attractive in terms of avoiding the need to promote the concept of specific surface, which has a long history of rejection and disbelief over the last century (Chapter 3).

The mechanism of selection of fine aggregate percentage is illustrated in Fig. 2.5. This figure is for 20 mm maximum aggregate size.

The BRE booklet also provides similar charts for 10 mm and 40 mm maximum sizes. The difference between the recommended percentages of a given fine aggregate differs more between the different maximum sizes than this author would consider desirable.

It can be seen that a higher fine aggregate percentage (and therefore a higher surface area, giving greater cohesion) is used for higher slumps. At one time, the author's system automatically related specific surface to slump in the same way, but this was found to be too rigid, even though normally desirable. Fine aggregate percentage is also related to cement content, i.e. to w/c ratio at a given water content. This is the same result as obtained by inclusion of cement as the author's EWF and MSF (Chapter 3).

The tabulated water contents are shown in Table 2.1. This is partly of interest for comparison purposes and partly to show the treatment of pulverized fuel ash (pfa), also called fly ash.

The remaining interesting technique used is that of combining the tabulated strength data (Table 2.2) with a dimensionless series of w/c-strength curves (Fig. 2.6). The technique is to enter the graph on

Table 2.1 Required water content (BRE)

Slump Vebe time (s)		*0–10 >12*	*10–30 6–12*	*30–60 3–6*	*60–180 0–3*
Maximum size of aggregate (mm)	*Type of aggregate*	*Water content (kg/m³)*			
Part A					
Portland cement concrete					
10	Uncrushed	150	180	205	225
Crushed	180	205	230	250	
20	Uncrushed	135	160	180	195

Slump Vebe time (s)		*0–10 >12*	*10–30 6–12*	*30–60 3–6*	*60–180 0–3*
Maximum size of aggregate (mm)	*Type of aggregate*	*Water content (kg/m^3)*			
Crushed	170	190	210	225	
40	Uncrushed	115	140	160	175
Crushed	155	175	190	205	
Part B					
Portland cement/pfa concrete					
Proportion 'p' of pfa to cement plus pfa (%)	*Reduction in water content (kg/m^3)*				
10	5	5	5	10	
20	10	10	10	15	
30	15	15	20	20	
40	20	20	25	25	
50	25	25	30	30	

Table 2.2 Strength of normal cement mixes at 0.5 w/c ratio

Type of cement	*Type of coarse aggregate*	*Comprehensive strength (N/mm^2) Age (days)*			
3	*7*	*28*	*91*		
Ordinary Portland (OPC) or sulphate-resisting Portland (SRPC)	Uncrushed	22	30	42	49
Crushed	27	36	49	56	
Rapid-hrardening Portland (RHPC)	Uncrushed	29	37	48	54
Crushed	34	43	55	61	

the 0.5 w/c line with the appropriate tabulated strength value. An adjustment to any other strength or w/c value can be made by moving parallel to the printed curves. The same graph can also be used for adjusting values in accordance with actual test results.

The table provided for cement/pfa mixes gives identical 28-day strengths but substitutes $w/(c+0.3f)$ for w/c ratio, i.e. the fly ash is discounted to 30% of the cement strength value. This is excessive in this author's experience with Australian fly ashes. The table offers no opinion on strengths at earlier or later ages than 28 days but presumably these would be lesser and greater respectively, than those for normal Portland cement.

2.6 MANUAL USE OF CONAD SYSTEM

2.6.1 Basis of manual use

It has already been noted (section 2.5) that the BRE/DOE system is effectively based on specific surface and is non-computerized. However, the author's system was in use for many years (by himself only) prior to computerization. It is not necessary to forego the more precise assessment provided by modified specific surface just because a computer is not available.

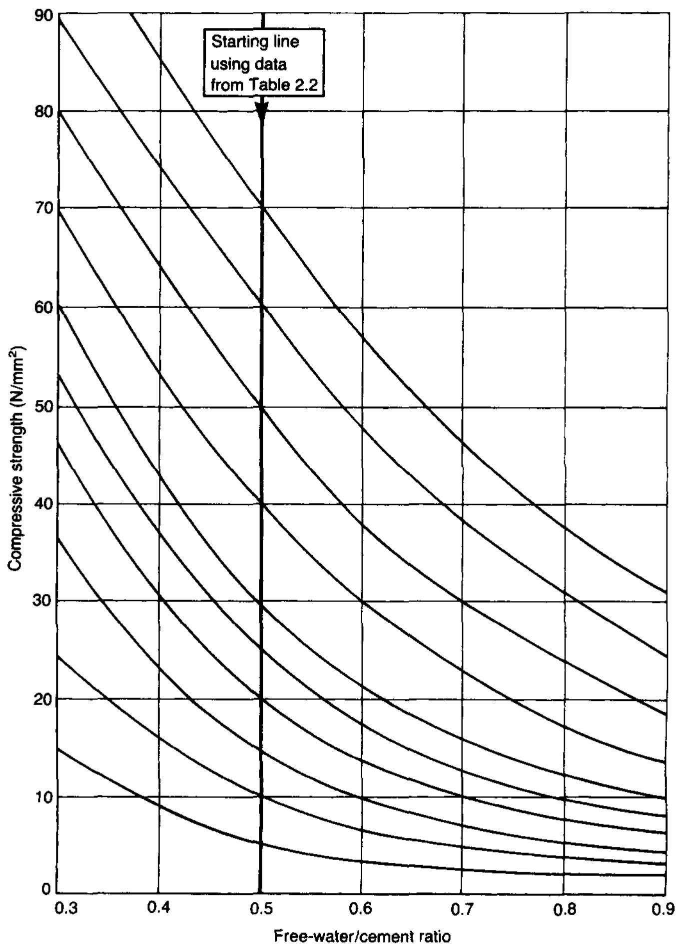


Fig. 2.6 Strength-w/c curves.

Calculation of the modified specific surface of each aggregate using a calculator is little more arduous than fineness modulus calculation (section 3.7). If the effect of varying cement and entrained air contents are to be neglected, as in most mix design systems, the determination of the desirable fine aggregate percentage is extremely simple. The designer may have a particular combined grading curve in mind (e.g. one of the four Road Note 4 type gradings). The specific surface of the desired grading can be determined in exactly the same way as for an individual aggregate. With experience, what will be in mind will be a direct value of combined specific surface taking into account all circumstances (including desired slump, cement content, air content, etc.).

The fine aggregate percentage is then calculated as:

$$\%\ \text{Fine aggregate} = \left(\frac{\text{desired combined SS} - \text{coarse aggregate SS}}{\text{fine aggregate SS} - \text{coarse aggregate SS}}\right) \times 100 \qquad (2.1)$$

Where more than two aggregates are to be used, the combined specific surface (SS) is given by:

$$\text{Combined SS} = (\text{SS agg } 1 \times \% \text{ agg } 1 + \text{SS agg } 2 \times \% \text{ agg } 2 + \ldots)/100 \tag{2.2}$$

All aggregates may be directly combined by trial and error in this way or all coarse aggregates may be combined in arbitrary proportions and all fine aggregates treated similarly. Equation (2.1) may then be used to determine the relative percentage of combined fine aggregates to that of combined coarse aggregates.

Before the advent of pocket calculators, the author had designed many mixes in the field, literally on the back of an envelope, from no more information than a sand grading. The process took about five minutes. Coarse aggregate SS was usually guessed at 4 or 5 (it has only a small effect) and it was necessary to have in mind either a cement content or a w/c ratio.

Of course, accuracy is improved by all the features now incorporated in Mixtune, the author's computerized mix design system described in Chapter 3. A computer enables all these to be brought to bear in less than the five minutes for manual use of the original basic concept. As many as desired of these features may be added manually but to add them all would extend the process to an hour or more. The point is that the basic concept already provides as much or more accuracy and much more flexibility than most other mix design systems and only a direct assumption, such as a 1:2:4 mix, is quicker to use without a computer.

2.6.2 Example of manual approximate design

Desired characteristic strength	40MPa
Allow for standard deviation (range 3 to 6)	say, $+1.65\times4$
Required mean strength $40+(1.65\times4)$	$=46.6$ MPa
Water requirement (160 to 200)	say, 180 litres/m^3
Required w/c ratio (using strength=*25/(w/c)* –8)	=0.458
Cement requirement=180/0.458	=393 kg/m^3
Required specific surface (22 to 30)	say, 25

Sand specific surface

Note that a very fine (Zone 4) sand would have an SS of about 64 and a very coarse (Zone 1) sand one of about 40.

Grading:	*Sieve*	*% Pass*	*%Rtd*	*Factor*	*Total*
	4.75	100	0	8	0
	2.36	90	10	15	150
	1.18	80	10	27	270
	600	60	20	39	780
	300	30	30	58	1740
	150	10	20	81	1620
	0	0	10	105	1050
					5610

$$\text{Sand specific surface} \qquad = 5610/100 = 56.1$$

Say, coarse aggregate specific surface approx. 5

Required sand % = [(25 − 5)/(56.1 − 5)] × 100 = 2000/51.1 = 39%

Cement paste volume	= water + cement + air
	= 180 + 393/3.15 + (2.0 × 10)
	= 324.8 litres/m^3
So, aggregate volume	= 1000 − 324.8 = 675.2
Say, SG of fine aggregate	= 2.6
and SG of coarse aggregate	= 2.8

Then

Wt of fine aggregate	= 675.2 x 0.39 x 2.6 = 684 kg/m^3
Wt of coarse aggregate	= 675.2 x 0.61 x 2.8 = 1153 kg/m^3

The approximations in this design are in selecting the water content and the strength formula. A more accurate way of estimating water content and a more accurate strength formula are given in Chapter 3 or tabulated values can be selected from other systems.

The required specific surface is not an estimate but a selection by the designer to suit the particular job conditions. If desired, selection can be via the tabulated values of mix suitability factor in section 3.3 (with no entrained air and a cement content of 250 kg/m^3 specific surface and mix suitability factor are identical).

The process described above is simpler than most published systems whilst still providing accurately for the effect of varying fine aggregate grading and permitting the designer to select the type of concrete desired.

2.7 THE ACI SYSTEM

The American Concrete Institute (ACI) (ACI 211, 1991) system is no doubt the most widely used system in the world and has a number of good features. The principal such feature is the use of the bulk density or unit weight of the coarse aggregate as a starting point. This very neatly allows, in one number, for the combined effect of grading, specific gravity (particle density) and particle shape of the coarse aggregate on the desirable sand content. The sand content is further varied on the basis of the fineness modulus of the sand (Chapter 7) and the absolute volume of cement, water and entrained air. In effect the volume of all other ingredients is established and the balance is taken as sand (Table 2.3).

The system does not provide for selection, at the user's choice, of other than the tabulated proportion of coarse aggregate but it is not invalidated by this being done. Water content prediction takes into account only slump, maximum aggregate size and whether or not air is entrained (Table 2.4). The tabulated strength *versus* w/c ratio figures are very

Table 2.3 ACI table for proportioning of coarse aggregate

Nominal maximum size of aggregate (mm)	*Volume of dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli of fine aggregate*			
2.40	*2.60*	*2.80*	*3.00*	
9.5	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.60

Nominal maximum size of aggregate (mm)	*Volume of dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli of fine aggregate*			
2.40	*2.60*	*2.80*	*3.00*	
25	0.71	0.69	0.67	0.65
37.5	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.80	0.78	0.76
150	0.87	0.85	0.83	0.81

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C29.

conservative indeed (Table 2.5). Given accurate specific gravity figures, yield is automatically exact by this system.

The system can be quite readily computerized and the author (as a member of ACI Committee 211, the revising committee for the document) has been advocating for several years that the committee do this officially. What is missing from the system is a recognition that different degrees of sandiness (cohesion) are appropriate for different uses. This could readily be provided in the form of a multiplying factor for the tabulated values of proportion of coarse aggregate, which could be called a ‘cohesion factor’.

The other weak aspects of the system are the tabulated water requirements and the assumption that strength is solely dependent on w/c ratio. If these defects were remedied and the system computerized, it would be a strong competitor to the author’s system (Chapter 3). There would be no difficulty in replacing the fineness modulus of the fine aggregate by specific surface in deciding upon (i.e. calculating) the proportion of the bulk density (or unit weight) of the coarse aggregate to be used. It should also be noted that the latest version of ACI 363 (ACI, 1992), which deals with high strength mixture proportioning, contains an adjustment for predicted water requirement based on percentage voids in the fine aggregate. This has yet to flow through to ACI 211 (ACI, 1991), which deals with normal mixture proportioning, but could be an important improvement. This adjustment is further discussed in section 3.14.

2.8
TRIAL MIX METHODS

The most widely used formal trial mix system is that used in the UK by the British Ready Mixed Concrete Association (BRMCA).

Table 2.4 ACI 211 water requirement tabulation. Appropriate mixing water and air content requirements for different slumps and nominal maximum sizes of aggregates (SI)

Slump, mm	*Water, kg/m^3 of concrete for indicated nominal maximum sizes of aggregate*							
9.5	*12.5*	*19*	*25*	*37.5*	*50*	*75*	*150*	
	Non-air-entrained concrete							
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	–

Slump, mm	*Water, kg/m³ of concrete for indicated nominal maximum sizes of aggregate*							
9.5	*12.5*	*19*	*25*	*37.5*	*50*	*75*	*150*	
Approximate amount of entrapped air in non-air-entrained concrete (%)	3	2.5	2	1.5	1	0.5	0.3	0.2
	Air-entrained concrete							
25 to 50	181	175	168	160	150	142	122	107
75 to 100	202	193	184	175	165	157	133	119
150 to 175	216	205	197	184	174	166	154	–
Recommended average total air content, % for level of exposure:								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
Extreme exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5	4.0

Table 2.5 ACI strength *versus* w/c ratio

*Comprehensive strength at 28 days (MPa)**	*Water/cement ratio by mass*	
Non-air entrained concrete	*Air-entrained concrete*	
40	0.42	–
35	0.47	0.39
30	0.54	0.45
25	0.61	0.52
20	0.69	0.60
15	0.79	0.70

*Values are estimated average strengths for concrete containing not more than 2% air for non-air entrained concrete and 6% total air content for air-entrained concrete. For a constant water/cement ratio, the strength of concrete is reduced as the air content is increased.

The initial trial mix uses an aggregate/cement (a/c) ratio typical of the range likely to be supplied in practice. The fine to coarse aggregate ratio is adjusted by eye until optimum plastic properties are obtained. A range of mixes with varying cement contents is then prepared, and water requirements and strength obtained at a given slump are determined. The data is then plotted to enable interpolation of properties at 5 or 10 kg increments of cement content.

While the above sounds crude, the actual detailed process is very carefully specified and has been found to give repeatable results. Drawbacks of the process are:

1. The need for laboratory facilities and, more importantly, expert personnel.
2. The time and cost involved.
3. While the system is very flexible in coping with strength variations (the scale already exists up or down which the cement content can be varied) it cannot cope with changes in aggregate properties (if the sand grading changes, the whole process must be repeated).

4. The only way of considering the relative merits of alternative *aggregates* is to carry out the whole process with both sets of aggregates. It would be very tedious and expensive to consider all possible permutations of several coarse and fine aggregates in this way.

J.D.Dewar has devised a computerized simulation of this process.

2.9 DEWAR: PARTICLE INTERFERENCE AND VOID FILLING

Dewar (Dewar, 1986; Dewar and Anderson, 1992) has developed a comprehensive theory and an associated mathematical model of particle mixtures that has been validated for powders, aggregates, mortars and concretes. The theory is a development of ideas generated by Powers (1968), in particular the use of the parameter voids volume per unit solid volume of particles.

The essence of the theory is that when particles of two different sizes are mixed together the benefit of reduction in voids caused by the smaller particles filling the voids between the larger particles is partially offset by interference in the packing of both sizes. Dewar has been able to model both effects mathematically into a comprehensive system that has been computerized by Questjay Ltd in the UK.

The operation of the system for concrete requires knowledge of only three parameters for each solid component. These are:

1. Relative density

   (a) for aggregates—SSD basis.
   (b) for powders—modified kerosene value.

2. Mean size (on log basis)

   (a) for aggregates—from grading tests.
   (b) for powders—from particle size distribution or from fineness test.

3. Voids ratio

   (a) for aggregates—from loose bulk density tests (in SSD condition) and from relative density.
   (b) for powders—from Vicat tests for standard consistency and from relative density.

Knowledge of the mean size of each material enables effects of size ratio to be computed. Influences of the range of sizes about the mean size together with effects of shape and texture are accounted for by measuring the voids ratio of each material.

For a simple mixture of only three components, e.g. cement, sand and gravel, the computer program first blends the two finest materials, cement and sand, into the full range of mortars and then blends the mortars with gravel, selecting only those blends that will have adequate cohesion at the selected slump. The resulting concretes cover the complete range of all possible mixtures enabling selection of the most appropriate mixture for any purpose, e.g. strength, durability. The results obtained by Dewar from theory correlate well with practice in the UK ready mixed concrete industry, which needs to have a wide range of economic mixtures always available for instant use.

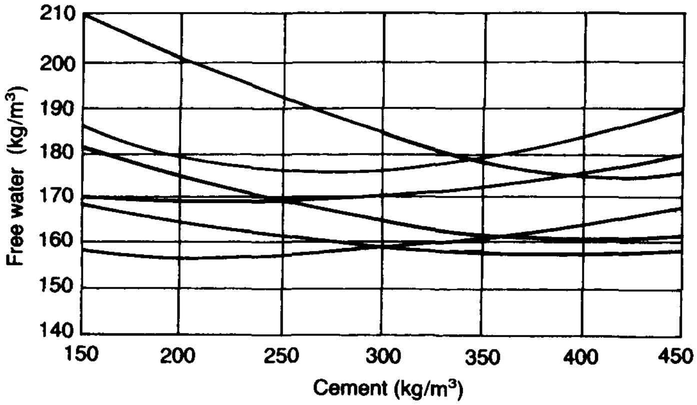


Fig. 2.7 Examples of relationships between free water demand and cement content.

Figure 2.7 shows, for several different sets of materials, how the variation of water content of concrete with cement content can be modified considerably by the properties of the materials used. With such variation it is important to know the relationship applicable to each set of materials to be combined together. It will be noted that some relationships are essentially constant over a wide central band but others are far from constant.

Dewar suggests that one of the uses for his program can be to examine other methods to determine their range of applicability. This could be particularly useful when extending beyond the original range of a method.

By way of example, Dewar examined cursorily a number of methods including an early version of the Conad system (Table 2.6).

Dewar was able to show generally good agreement between theory and the Conad system over most of the range. However, Dewar's Fig. 2.7 is a warning to all developers of systems to identify the range of applicability to reduce the risk of significant error.

On the assumption of the validity of the theory developed by Dewar, the question can be raised as to whether different methods can be equally valid at least within a particular range. Private discussions have concluded that although different terminology may be used there may be a hidden common basis in many systems.

For example, the concept of mean size ratio used by Dewar with

Table 2.6 Comparison of Conad and Dewar predictions

Parameter	*Method*	*Cement content (kg/m^3)*			
120	*230*	*310*	*420*		
Free water (1/m^3)	Theory	186	162	162	176
Conad	158	160	161	163	
% fines	Theory	51	46	41	32
Conad	53	48	44	38	

regard to particle interference, that of specific surface index, and that of fineness modulus, are not identical but they do have common links. Specific surface has the dimensions of m^2/m^3, i.e. 1/m, and is thus the

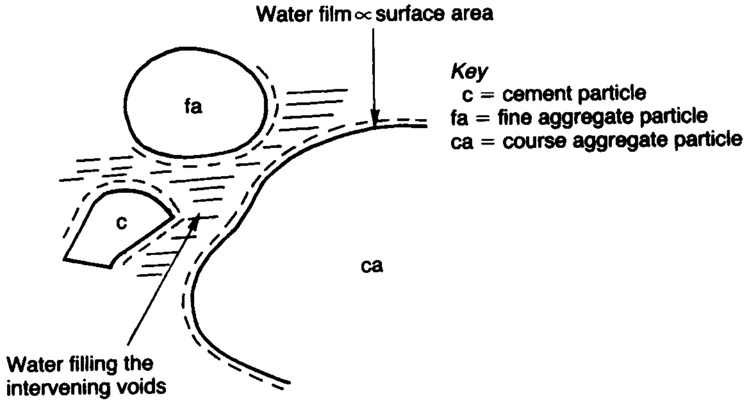


Fig. 2.8 Functions of water in filling voids in concrete.

reciprocal of linear size. Fineness modulus is determined from grading and is thus also size related. Thus when other factors are constant or are not dominant, apparently different concepts may lead to similar results.

Dewar's main contention against surface area concepts when used on their own is that they do not account for variation in grading about the mean size or for shape, both of which have an influence on water demand because of their influence on voids between particles. Expanding on this, his contention is that water demand has three components (Fig. 2.8):

1. Water to fill voids close to a particular surface.
2. Water to fill voids between particles at normalized workability (50 mm slump).
3. Additional or reduced water for selected workability.

The reason for the differentiation between components (1) and (2) is that particle interference reduces the ability of smaller particles to fill voids close to the surface of larger particles compared with their ability to fill voids in 'open' space. There is a close but not identical analogy between (1) and the specific surface concept of water to coat the surface of particles.

Size ratio and thus other size related factors, affect both (1) and (2).

Particle interference and voids have been traditionally minimized by employing a large differential between the sizes of cement, fine aggregate and coarse aggregate, the respective mean sizes being in the order of, say, 0.015 mm, 0.4 mm and 12 mm, i.e. relative size ratios of about 30.

However, even this differential is not sufficient to reduce particle interference to zero and the coarser particles are required to maintain a dilated structure to accommodate the finer particles with consequent increased voidage and water demand.

The above section was kindly contributed by Dr Dewar. It is difficult for the author to compare the results of the two systems because they use different data, e.g. the author has extensive data on mix designs and their performance and constituent materials but his data do not include the bulk density data used by the Dewar system.

The above has not been revised since the first edition. The Conad system now recognizes that water content will increase outside an ideal range of cement content of the order of 300 to 350 kg/m^3. However, the Conad system is less concerned with an initial estimate of water requirement and more concerned with its variation as slump, temperature, sandiness, and air content vary. Also the Conad system is designed to accept feedback of production data including water contents and to amend predictions accordingly.

Readers are strongly advised to read Dewar's latest book (Dewar, 1999) for an in-depth account of his PhD thesis on mix design. Also Questjay Ltd have produced a user-friendly Windows version of Dewar's system called Mixsim98.

2.10 DE LARRARD: VOID FILLING AND MAXIMUM PASTE THICKNESS

François de Larrard, working at the Ponts et Chaussees (Bridges and Roads) laboratory in Paris, has originated a theory basically very similar to that of Dewar, but favouring aggregate void measurement under vibration rather than loose poured measurement as with Dewar. Dr de Larrard has also introduced a concept he calls MPT, or maximum paste thickness, which appears to account well for the strength reduction (at any given w/c ratio) for mixes with a higher proportion of cement paste. His work includes very extensive mathematical coverage of many types of concrete and many pozzolanic and chemical admixtures.

Dr de Larrard has written a very comprehensive book (de Larrard, 1999) which is also highly recommended to readers interested in precise mathematical mix design. He was offered a few pages in this volume (as accorded to Dewar) to briefly summarize his theories but preferred to have the author express his own views.

De Larrard has had the advantage not only of excellent facilities and assistance at the Ponts et Chaussees laboratory but also of collaboration with extensive actual project work, equipment fabrication, material supplies, etc. His work has included the origination of the BTRHEOM (section 12.1), a parallel plate viscometer which has been of considerable assistance in his mix design work.

2.11 POPOVICS

Professor Sandor Popovics is without doubt the world's leading 'numerical concrete technologist'. His prodigious output of textbooks and research papers provide a larger number of quantified relationships between the properties of concrete materials and the properties of the resulting concrete than all other researchers combined. The difficulty in his works is never to find a relationship between the factors in which one is interested, but rather to select one from the many alternatives available.

His work provides any originator of a new mix design system with a wealth of assistance but does not lead to a unique 'Popovics system'. Rather it sets out every conceivable basis on which a mix could be designed and how the existing major systems have dealt with the problem.

He does express a strong preference for the use of a fineness modulus (FM), both to proportion fine and coarse aggregates and to estimate water requirement. However, he admits that fineness modulus does not necessarily provide adequate security against an inappropriate amount of fine fines. He prefers the latter to be regulated by 'common sense' but alternatively proposes to regulate it by his *D-m-s* criterion. In this, D is the maximum aggregate size, *m* the fineness modulus and *s* the specific surface. In this author's opinion, a requirement to have both the same fineness modulus and the same specific surface is almost the same as requiring the identical grading. The *D-m-s* approach therefore suffers from the disadvantages of the ideal grading approach.

Popovics quotes a Hungarian reference (Palotas, 1933) as an experimental justification of the use of fineness modulus. In this, 13 very different gradings with the same FM were made up and gave very similar workabilities and strengths. The data presented are very convincing.

Various investigators are quoted to establish the existence of an optimum FM for any combination of maximum particle size, particle shape and cement content. A table is provided from Walker and Bartel (1947) which, together with a list of adjustments, permits selection of the ideal FM.

Popovics is quite strongly opposed to the use of specific surface as the sole criterion of suitability and has re-analysed the data of Newman and Teychenné to show that they can be better explained by FM than by SS. This is possible because all of the mixes have the same maximum size and none have any appreciable amount of finer than 150 µm. There is a substantial difference between the findings of the two alternative criteria (FM and SS) about very coarse and very fine particles. SS attributes only a small influence to maximum size and very large influence to fines. FM is exactly opposite. Popovics considers FM to be correct at the fine end. He quotes many attempts (Popovics, 1992; Table 7.1) to quantify the true relative effects of the various sieve fractions, but he has not provided his own set of numbers.

Popovics' relationships between water content and slump and the various influences on this relationship are included in Chapter 3 for comparison with those of the author. They were not to hand at the time the author's system was evolved and incorporation of some of them could possibly improve the author's system.

Similar remarks apply to Popovic's formulas modifying the relationship between strength and w/c ratio. He is understood to be currently developing a further improvement to these formulas.

2.12
BRUSIN METHOD

Michel Brusin, for 20 years in the French precast industry but now Secretary General of RILEM, has developed an advanced system of computerized mix design and has kindly provided details of it for this book. The system offers the options to calculate an ideal grading curve using any of three established techniques. These are the methods of Bolomey, Faury and Dreux-Gorisse. As with the Conad system, the system can receive, store, and incorporate details of the available materials and the required concrete properties. It can then determine the relative proportions of materials to most closely match the desired curve.

However, the system does substantially more than merely match an ideal grading curve. The program incorporates a database from which it is able to translate input verbal descriptions such as 'rounded gravel' or 'sharp sand' into desirable allowances for the relative percentages of the materials. Provision is made for the use of different strength classes of cement (such classification is widespread in Europe but not in the UK, USA or Australia) and the use of this, together with the mix proportions, to generate a predicted strength. It is able to incorporate the addition of fly ash, slag and chemical admixtures.

A particularly interesting feature of the system is its calculation of the percentage voids which will remain after hardening and evaporation of uncombined water.

The system is able to draw on the results from actual concrete mixes to derive corrections to its initial designs. It is recommended that trial mixes are always carried out and the system readily permits revision. The original input details, such as cement content, are remembered by the system. When an adjustment, such as additional water, necessitates an adjustment of batch quantities to restore correct yield, the system enquires of the user whether the adjustment is to maintain the original cement content, w/c ratio, or a/c ratio.

2.13
RILEM TECHNICAL COMMITTEE TC 70-OMD

RILEM is an international laboratory practice and research organization based in Europe. In 1983 it set up Technical Committee TC 70–OMD to prepare recommendations on optimized mix design of concrete to meet requirements for specified applications. The committee's brief stated that the mix design methods were to be based, if possible, upon mathematical or physical models because 'in this form the methods are particularly suitable for computer use'. It is now time to report on the RILEM exercise so that it may be realized just how little agreement there is between different systems. The committee had 24 members from 12 countries. The results are given in Table 2.7.

The situation can perhaps best be conveyed by selected quotations from the Committee's own reports:

> 'In order to compare the predictions of various mix design models, committee members were asked to calculate, using their mix design models, the batch weights for concretes with target mean compressive strengths of 30 and 50 MPa and slumps of 0 to 10 mm and 60 to 120 mm using cement and aggregate.'
>
> 'All the methods proposed are applicable to OPC concretes, two are applicable to OPC-pfa concretes, but none are applicable to OPC-slag concretes. A number of the methods allow for air entrainment but only one takes into account a property other than strength or workability.' 'Twelve replies were received. The extreme mixes are given in Table 2 [Table 2.7 of this book]. The predicted cement contents and w/c ratios varied by over 300 kg/m and 0.2 respectively. The Committee were surprised that the predicted mix proportions varied so widely. It is clear that some of the predicted mixes would result in "unworkable" concrete or concrete which would give a compressive strength markedly different from that specified.'

Whichever of these answers is nearest to the truth, it is clear that virtually any technical manager of any ready mix producer would be able to get closer to that truth 'off the top of his head' than at least 25%, if not 75%, of the assembled 'experts' (of which the current author was one). What is less certain is **which** 25% of the experts are worse than the 'rule of thumb' or empirical practitioner. It can certainly be seen why theoretical mix design systems are treated with great suspicion by the industry.

Table 2.7 The RILEM exercise: extreme mixes

	Concrete			
	1	*2*	*3*	*4*
Target 28–day strength (MPa)	30	30	50	50
Workability, slump (mm)	0–10	60–120	0–10	60–120
Cement content (kg/m^3)	187–361	247–460	290–640	356–828
Water content (1/m)	147–180	178–230	150–201	185–275
Free w/c ratio	0.79–0.50	0.72–0.50	0.52–0.33	0.52–0.33
Total aggregate content (kg/m)	2037–1871	1879–1763	1928–1794	1781–1645
Grading 0–2 (kg/m^3)	954–733	752–691	675–555	623–441
Grading 2–8 (kg/m^3)	357–145	376–137	675–117	623–97
Grading 8–16 (kg/m^3)	726–993	752–935	578–1122	534–1107

2.14
ASSESSMENT OF ALTERNATIVES

In the past it has been necessary to oversimplify mix design and to work from tabular or graphical design aids in order to reduce the time and effort involved to an acceptable amount. Virtually all mix design systems end with a recommendation to prepare a laboratory trial of the designed mix and to adjust it by eye. Thus the 'accuracy' of the finally adopted mix is dependent on the skill of the trial mix conductor rather than the design system used. Under these circumstances it is clearly not worthwhile subjecting oneself to a lengthy and arduous theoretical design process.

This situation is completely transformed by the advent of computers. There is now no reason to avoid extreme mathematical complication if there is any prospect that it may increase accuracy. It should also be possible to create a system which can evaluate an input mix and also a system which can 'learn from its experience'. Thus if a mix gives a higher strength than is predicted by the computer, the system should provide for the input of an adjustment factor by which to scale future predictions. Except for research purposes, and to develop new types of concrete, the use of trial mixes has become superfluous. A mix system needs to be able to produce usable concrete without a trial mix (i.e. concrete with acceptable wet state properties) and the fine tuning can then be done more accurately by feeding back test data on job concrete than by any number of trial mixes.

There have been many comparison exercises carried out between various mix design systems. Some of these have been accompanied by rather fatuous conclusions, such as that one system yields more economical mixes than another. Few have considered whether the competing systems have relevant features rather than whether they 'give the right answer'.

A mix design system should enable the user to:

1. Select which of the available aggregates will produce the most suitable concrete (which should generally mean the most economical concrete having the required properties).
2. Determine the most advantageous relative proportions of the selected aggregates.
3. Determine the water requirement of the concrete.
4. Determine the required cementitious content of the concrete.

If the system is capable of satisfying the other three requirements with reasonable accuracy, without requiring other than the normally available specific gravity (SG) and grading data for aggregates, the first requirement will be readily met.

It may be acceptable, assuming the ready availability of a combined grading display for any input combination of relative proportions, for those relative proportions to be determined by trial and error on a computer screen. However, the third requirement of being able to calculate water demand for any mix will require a means of assessing the effect of different aggregates and proportions on water requirement. The means will need to be in the form of an equation rather than a table. This is partly to suit the use of a computer and partly because there will be too many other factors involved (e.g. at least slump, temperature, air content, silt content of sand, cement content) to permit use of a tabular system.

The parameters available for use in a water prediction equation include fineness modulus, specific surface and the flow time and/or void content of the dry fine aggregate.

The determination of the necessary content of cementitious material (including cement, fly ash, blast furnace slag and silica fume) is quite simple if the relative cementing efficiency of these materials is known. Certainly the system must include scaling factors to allow for varying efficiency of these materials and must also take account of the effects of incomplete compaction, air content and declining strength gain per unit increase of cement at higher cement levels.

There seems little doubt that the packing techniques of Dewar and de Larrard are able to present a more complete and accurate model of a concrete mix than the specific surface technique of the author. Both are also computerized to avoid the user having to even see much of the mathematical background. However, specific surface operates with less data input and a more easily understood theory and appears to produce similar results in most circumstances. It remains to be seen in practical use which gives the more satisfactory performance.

Specific surface has been shown to simulate quite well the mix ranges currently in use by several major concrete producers. Since this is so, the method can clearly be of assistance in optimizing mixes in the face of varying materials and conditions. It remains to be seen whether packing techniques can go further than this and demonstrate that a range of mixes which have been optimized according to specific surface theory can be made more economical or otherwise improved by the use of packing theory.

3
The Conad Mixtune method of mix design

There are now at least two mix design theories (Dewar 1999, de Larrard 1999; Chapter 2) which are certainly more rigorous and comprehensive than that of the author. They give an even more accurate prediction of the effects of shape and grading variations, and are also computerized so as to be relatively simple to use. The justification for the continued use of the Conad system is that it requires less data, is a simple concept requiring little mathematics, always results in acceptable fresh properties of concrete, and integrates well with the Conad quality control (QC) and mix adjustment system and with typical current practice in the industry.

In the period since the first edition of this book there has been some change in the author's concept of mix design objectives. A mix was always seen as requiring adjustment as the properties of the constituent materials and the conditions of production (e.g. temperature) varied. It was however seen as an individual entity which may be related (e.g. by a common MSF value) to a series of other mixes. In the current concept the series of mixes is seen as the usual basic unit. Producers simply have too many mixes in simultaneous use to give them individual attention.

Although not yet implemented, the ultimate concept is to revert to a single mix formula, but one that covers all mixes in use. If it can be shown, as it is in this chapter, that a table of mixes can be generated from a formula, then the table can be replaced by the formula and any mix in the table generated as needed. The big advantage of this approach is that the mix generated can take full account of current data including temperature, air content and required slump, as well as current aggregate gradings and cement performance. The formula may also include factors such as MSF (i.e. sandiness, section 3.3) and the relative proportions of aggregates, which essentially make it capable of generating all the tables of mixes in use (e.g. for normal, pump or paving mixes and even for spray mixes or no-fines concrete). The snag in implementing such a concept is in the analysis of test data for QC purposes. The author is currently working on this problem and believes a technical solution to be in sight. The real problem will be in gaining official acceptance.

Conad does not rely greatly on initial accuracy and does not advocate trial mixes (although it does provide assistance in carrying them out if they are used). Rather it concentrates on the accurate adjustment of mixes in production, producing concrete of the desired fresh properties and relying on feedback of production test data to achieve the precise strength targetting.

3.1
VALIDATION OF THE METHOD

The typical ready mix producer has a number of tables of mixes setting out the variation of cement and fine and coarse aggregate content to be used over a range of mix strength requirements. For some the first column of the table will be strength grade. For others the first column will be cement content in 5 kg steps and the row corresponding to any required strength will be selected on the grounds of current strength performance. A table may or may not have had some theoretical basis at some time in the past but the individual mixes are likely to have been adjusted by eye over a considerable period of time to give the same required fresh properties in each strength grade. The producer may have many such tables corresponding to different types of concrete (pumped, skip placed, paving, low shrinkage, flowing, etc.), to different aggregates, to variations in grading of those aggregates encountered from time to time, and perhaps even to the preferences of particular clients or particular specification writers. A large producer with many sources of fine and coarse aggregates may have a very large number of resulting mix designs. One producer has claimed to have as many as 5000 such mix designs and a figure of 1000 is not uncommon. Obviously it is quite difficult to maintain adequate records of such a situation and even more difficult to make logical adjustments to a range of mixes rather than descending to individual *ad hoc* adjustment.

The author has examined many such tables and has found that, allowing for the obvious likely degree of error, all the mixes in a table tend to have a factor, which the author calls a Mix Suitability Factor (MSF) in common. The MSF permits such a table to be condensed into a single formula, from which it can be re-generated at any time. The formula does not have to be revised when aggregate gradings vary but rather results in changed aggregate proportions that retain the mix properties unchanged. Furthermore, a change from one table to another (i.e. one type of concrete to another) may only be a matter of changing the MSF value in the formula.

The MSF could be regarded as a measure of the cohesiveness or sandiness of the mix of far greater significance than the actual sand percent age. It is calculated from the surface area or specific surface of the coarse and fine aggregates, adjusted for the effects of cement content and entrained air percentage. The MSF did not arise from a study of mix tables but from the study of the effect of sand grading variations on a single mix in the period 1952–4 (Day, 1959). The author has used this as the sole basis for his mix designs for many different kinds of concrete, in many countries, over the past 45 years. It has permitted him to provide a mix design service over the telephone to contractors in the wilds of SE Asia and elsewhere, when the only information available was a sand grading and the site engineer's description of the appearance of the coarse aggregate, and the concrete was required in the actual structure the following day.

Since the concept takes account of only one facet of aggregate properties, it is not to be supposed that it can cope accurately with as wide a range of aggregates as the more sophisticated concepts of Dewar and de Larrard. This shows itself when dealing with very coarse sands combined with low cement contents, with very high strength requirements, with badly shaped coarse aggregates, and through a reduced precision in predicting water requirement and the contribution to strength of pozzolanic and fine filler materials. The Conad system copes with all this by using semi-automatic feedback of obtained results to produce correction factors. The overall result may be less scientifically satisfying but can lead to mixes being settled into accurately adjusted production in less time than it takes more rigorous systems to commence production use.

There is still the situation in some countries, including France and the USA, where adjustment of mixes in production is not permitted. In such circumstances it will often pay to adopt more rigorous design and experimentation methods prior to commencing production. However, the net result of the restrictive specification and the more rigorous mix design is a need to either provide a larger margin for variability or to very rigorously control all input materials. It is unlikely that such a combination would ever achieve as low a variability or as precisely targeted and cost-competitive a solution as the Conad system with appropriate feedback.

Another aspect of the design of mixes is that these are often influenced by factors other than concrete technology. For example, it is common in Melbourne, Australia, to have coarse aggregates separated into 20 mm single-sized and 14 mm graded to 7 mm and to use them in little short of 50/50 proportions. Now, it does not require a sophisticated design system to know that 10 mm is a better filler for 20 mm than is 14 mm and that the ideal proportion is probably closer to 2 of 20 mm to 1 of 10 mm than 50/50. However, many plants have only two bins available for coarse aggregates, receive orders for 14 mm maximum size mixes on occasions, and would run out of 20 mm material if they used it in higher proportion. Large and otherwise sophisticated plants have been seen in Singapore and New York using only a single coarse aggregate graded from over 25 to 5 mm. Segregation in batching is inevitable in such a situation. Add to this that many plants in the world use only a single sand and it is seen that greater sophistication than the Conad system provides is essentially pointless in many if not most cases.

These are some of the reasons why the author has chosen to stick with the very simple, quick, and easy to use and adjust, Conad system rather than seeking greater sophistication. A final reason is that any more sophisticated method requires more input data (especially bulk densities or unit weights) than the grading and SG needed for Conad and this has rarely been available from the author's clients.

3.2 PROVIDING SERVICE TO AN EXISTING MAJOR PRODUCER

As described above, an established major producer may have hundreds, and possibly thousands, of mixes on record. There will be no welcome for a person who suggests that these be thrown away and replaced by a newly designed set. What may be acceptable is a logical framework that can be slid into place under the existing mix structure, one group of mixes at a time, replicating almost exactly the main mixes in current production. Such a structure will no doubt reveal minor illogicalities between mixes in each set, which can be corrected. Some mixes in the set, especially if they have not been used extensively since the last change in aggregate grading or shape, may require larger adjustments. Comparison between different sets of mixes is likely to reveal at least small further illogicalities for correction.

The resulting mix structure can then be compared with an absolute standard, distilled from many such comparisons and extensive use elsewhere. Such a comparison is likely to suggest that at least small improvements or economies would result from an adjustment in one direction or another. Such an adjustment, if substantial, may be made in steps to avoid upsetting current customers and to confirm that the theoretical improvement is being experienced.

The producer would then be in a strong position with a sound theoretical basis for his entire range of mixes. If and when changes in current aggregate gradings were experienced, or a need or opportunity arose to use a completely new aggregate, merely entering the new aggregate details in the system would cause an automatic revision of all mixes using it. Similarly a downturn in cement or other mix factor to be countered, or an upturn to be taken advantage of, would involve entering only a single number to effect a change in all mixes. Since the resulting mix proportions could be precisely established prior to use, it would be easy to assess the economic merits of any new material offered to the producer.

A major reduction in record keeping would have been achieved. A significant factor is the integration of records for mixes and those for material properties. If it is known that certain mix adjustments will automatically be made when material properties change, then records of the material property variations essentially constitute records of mix changes. Such records will be far more condensed, even if not fully automatic. For example, a simple note that a revised specific surface of a particular sand was entered in the mix system on a particular date could replace recording revised batch quantities for hundreds of mixes.

All this is not to deny that a producer may find that a few mixes in the range are giving trouble, i.e. are not conforming to the overall pattern. For example, a very coarse sand may produce good concrete in mixes with cement contents over 350 kg/m but experience bleeding, segregation and lower than calculated strengths in lower cement content mixes. It may be obvious to an experienced person that a proportion of a finer sand must be introduced. Conad may offer an automatic warning that the calculated sand percentage is too high. However, the more rigorous systems of Dewar and de Larrard can actually calculate quite precisely what proportion of a particular fine sand would give an optimum result.

3.3 THE CONAD CONCEPT: MSF, EWF AND SS

As noted above, the basic concept of Conad is that of Specific Surface (SS) of aggregates. Essentially the concept is to adjust the sand percentage to provide the desired overall SS. Unfortunately it is necessary to complicate the situation by introducing two further terms. These are the Mix Suitability Factor (MSF) (Table 3.1) and the Equivalent Water Factor (EWF). This is because additional cement produces a similar effect to more sand and because entrained air both increases cohesion (as does more or finer sand) and reduces water content (which is the opposite of more or finer sand).

So we have:

$$\begin{aligned}&\text{Degree of cohesion} = \text{MSF}\\&= \text{SS of aggregates} + \text{effect of cement} + \textit{effect of entrained air}\end{aligned}$$

and, water requirement proportional to:

$$\text{MSF} - \text{effect of entrained air} = \text{EWF}$$

or

$$\text{EWF} = \text{SS} + \text{effect of cement}$$

Table 3.1 Mix suitability factors

MSF	*Slump range*		*Remarks*
mm	*in*		
16			Unusable, too harsh
16–20			Harsh mixes, only suitable for zero slump concrete under heavy vibration
20–22	0–50	0–2	Hard wearing floor slabs, precast products under good external vibration
22–25	50–90	2–3.5	Good structural concrete
25–27	80–100	3–4	Good pumpable concrete. Fine surface finish. Heavily reinforced sections
26–28	90–120	4–5	Pumpable lightweight concrete
27–31	200+	8+	Flowing superplasticized concrete

But by 'cement' here we mean all cementitious, pozzolanic, and fine filler material. Each such material has to be given a coefficient to convert it to its 'cement equivalent' so that we have:

$$\text{Equivalent cement (EC)} = \text{C} + k_1\text{C}_1 + k_2\text{C}_2 + \ldots$$

(Note that, unfortunately, two sets of k values are required, since equivalent cement from the viewpoint of workability, as above, is not the same thing as equivalent cement from the strength viewpoint.)

Now putting empirical constants to these we have:

$$\text{EWF} = \text{SS} + 0.025(\text{EC}) - 7.5 \tag{3.1}$$

$$MSF = EWF + 0.25(air\ \% - 1) = SS + 0.025(EC) - 7.5 + 0.25(air\ \% - 1) \tag{3.2}$$

(Note that the coefficient 0.025 applied to equivalent cement is a revised value from further experience—the coefficient in the first edition was 0.02 and the constant–6 in place of –7.5 used above—really the original and new values were 0.02(EC–300) and 0.025(EC–300).)

It will be noted that specific surface is being used for two quite different purposes. One of these is to determine the desirable percentage of total fine aggregates to total coarse aggregates to provide an appropriate degree of cohesion. The other is to estimate water demand. In this respect it is clear that a higher MSF yields a more cohesive mix which can be used at higher slump and is easier to place but is more expensive because it has a higher water demand and therefore a higher cement requirement.

3.4
SPECIFIC SURFACE

Specific surface is the surface area per unit mass. It would actually be better for it to be the surface area per unit solid volume but this would destroy the current database of published and remembered values which are all in terms of square centimetres per gram. This makes little difference unless there is a substantial difference between the SGs of the fine and coarse aggregates, in which case it is important to make a correction.

The use of specific surface in mix design is not a new idea. In 1954 Newman and Teychenné set out the process by which specific surface figures can be used to replace the type grading curves of Road Note 4 (Newman and Teychenné, 1954). They stated the principle that:

> ‘if the combined aggregate grading is changed in such a way that the overall specific surface is changed, concrete of different properties will be obtained; but if the combined aggregate grading is changed so that the overall specific surface is kept constant, then concrete having the same properties **can** be obtained.’

The inference, strongly supported by the author, is that specific surface is a suitable parameter to specify, along with slump and strength, in defining a desired mix. In fact this is the missing parameter numerically defining the difference between cohesive, pumpable, sandy mixes and harsh mixes suitable for heavily consolidated, low slump mixes. However, as noted above, it does require modification for cementitious materials and entrained air. It will also be seen later that it over-estimates the effect of the finer sieve fractions.

The published discussion of the Newman and Teychenné paper contains references to earlier use (1918/19) of the concept by Edwards (1918) and Young (1919) in the USA. It further notes that Edward’s paper refers to the concept being at least 25 years old at the time he started to work on it.

Edwards had no means of measuring surface area and actually counted numbers of particles in each sieve fraction to start his calculation of surface area. Newman and Teychenné measured surface area by determining the permeability of each sieve fraction using a method described by Loudon (1952). Obviously a substantial obstacle to the widespread use of specific surface was the difficulty of measuring it. Another obstacle was that it appeared to over-estimate the effect of the finer part of a sand grading. Also many technologists felt that specific surface could not be the key to quantifying the effect of aggregate gradings since the specific surface of cement so greatly exceeded that of the aggregates that it must mask any marginal variations in the latter.

3.5 SURFACE MODULUS

The Newman and Teychenné discussion (1954) reports Young as having developed the **surface modulus** concept. This uses the fact that the normal series of sieve openings decreases in geometric progression and therefore, assuming spherical particles, their surface areas increase in geometric progression (i.e. each is double the preceding one in the series 38 mm, 19.0 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, 0.60 mm, 0. 30 mm, 0.15 mm (Table 3.2).

This is a simple enough technique. It was later taken up by Stewart (1951) and the author. It is not clear why the technique did not gain wider acceptance at the time. It may have been due to the previously mentioned considerations. It is also possible that the technologists of the time were too worried that it failed to take the shape of the particles into account.

3.6 PARTICLE SHAPE

The shape of a particle obviously affects its surface area at a given particle size. In the case of fine aggregates in particular, it certainly also affects water requirement. The question is whether it would be useful to correct the surface modulus (see above) for this effect, **even if it was easy to quantify accurately**.

It should be borne in mind that there are two objectives for the exercise. One is the determination of water content, for which such a correction would be helpful. However, the other objective is more crucial. This is to use the specific surface as a basis for varying fine aggregate content in a mix design. The reduction of fine aggregate percentage to counter increasing fineness depends upon three effects. A primary motivation is to counter increasing water requirement. This is possible since a smaller percentage of a fine aggregate still gives the same cohesion or segregation resistance as a larger quantity of a coarser fine aggregate. However, there remains a further necessary justification for the reduction. This is

Table 3.2 Surface modulus: area/volume relation

Diameter (mm)	*Surface area (mm^2)*	*Volume (mm^3)*	*Area/volume (mm^{-1})*
D	4πR2	(4/3)πR3	3/R
38	4356.47	28 730.98	0.16
19	1134.12	3591.37	0.32
9.5	283.53	448.92	0.63
4.75	70.88	56.12	1.26
2.36	17.5	6.88	2.54
1.18	4.37	0.86	5.09
0.6	1.13	0.11	10
0.3	0.2827	0.01414	20
0.15	0.0707	0.00177	40
0.08	0.0177	0.000221	80

that a finer aggregate causes less disruption to the packing of the coarse aggregate.

A closer packed coarse aggregate requires less fine aggregate to fill its voids and therefore there can be a similar amount of free or apparent fine aggregate (actually mortar) in the two cases. More angular particles have a higher specific surface but cause more, rather than less, disruption to coarse aggregate packing. Therefore a more angular particle shape would **not** justify a reduced fine aggregate content. So a surface modulus which took particle angularity into account would thereby be a less satisfactory criterion by which to evaluate the desirable fine aggregate percentage.

The answer to this dilemma is not to include any shape factor in the specific surface but rather to separately allow for the effect of shape on water requirement.

3.7
MODIFIED SPECIFIC SURFACE

Strictly this should be referred to as a modified surface modulus but it is thought that specific surface is the better understood of the two terms.

Eliminating, as discussed above, the effects of particle shape and silt content, there remains the problem that specific surface over-estimates the effect of the finer sieve fractions on water demand. There is a clear explanation of why this is so. A sand being wet may Involve a uniform thickness film of water on its entire surface area. If this is what happens, then it is very clear why water requirement should be exactly proportional to specific surface. However, a sand is also wet if all the voids in a mass of it are full of water. There is no size effect involved in the percentage voids in a sand. A single-sized 40 mm aggregate will have the same voids content as a single-sized 150 µm sieve fraction if the particle shapes are the same. So the amount of water required to fill the voids sets an upper limit to the water, which will apply when less than that calculated from surface area. This is why the smaller sieve fractions can be assigned only a limited effect on a 'modified specific surface' if this is to be a predictor of water requirement.

Since the values are empirical, a difficulty arises when a different series of sieves is used. This has previously limited the international applicability of the system. Recently the original values have been plotted on an SS value *versus* log mean size graph and a smooth curve drawn through them. Provision of a look-up table of interpolated values from this curve enables an SS value to be attributed to the log mean size of the material between any two sieves. It is too early yet to be sure that the interpolated values will work as well as the original values but initial indications are that they will. Interestingly, drawing the curve suggested a slight change in one of the values used for so many years would give a smoother curve. In effect the suggestion was that the departure from the pure surface area relationship started slightly earlier.

The author's 'modified specific surface' is therefore to be calculated from a normal sieve analysis by arbitrarily assigning a value to each sieve fraction as in Table 3.3. Alternatively a computer program is now available which permits any range of sieve sizes (expressed in millimetres) to be entered in the first column of a table and the percentage passing in the second column. The computer will then determine the appropriate factor for each sieve and calculate the SS value of the material.

The values in Table 3.3 can be used either on an individual aggregate or a combined grading. If used on individual aggregates the combined figure may be readily determined as follows:

$$SS_{ca} = \frac{SS_f \times \text{sand } \% + SS_c \times (100 - \text{sand } \%)}{100} \tag{3.3}$$

where SS_{ca}=combined aggregate SS
SS_f=fine aggregate (sand) SS
SS_c=coarse aggregate SS.

See section 7.1 for further details.

The SS values for the four Road Note 4 (RRL, 1950) type grading curves are shown in Table 3.4.

3.8
SILT CONTENT

Fine silt or clay in a concrete mix certainly has the potential to cause a very substantial increase in water requirement. It would appear from the author's experience that there is a threshold effect, with the first few per cent, say 6% by volume, not showing any increase. Above this level, the effect appears reasonably linear.

Table 3.3 Modified specific surface values

Sieve fraction	*Author's modified SS values*	*Approx. true specific surface (cm²/gm)**	*Surface modulus*
>20mm	2	1	1
20–10	4	2	2
10–4.75	8	4	4
4.75–2.36	16	8	8
2.36–1.18	27	16	16
1.18–0.600	39	35	32
0.600–0.300	58	65	64
0.300–0.150	81	128	128
< 0.150	105	260	256

*According to B.G.Singh (1958)

Table 3.4 Specific surface of Road Note 4 type gradings

Curve number	*SS value*
1	16
2	20
3	25
4	33

As with particle angularity, the silt increases water requirement without justifying a compensating reduction in sand percentage. Therefore, if specific surface is to be used for aggregate proportioning, the silt content effect also should be separately allowed for rather than included in the specific surface figure.

The water increase caused depends both on the amount of silt and on its nature. A given weight of very fine clay will cause about three times as much increase in water demand as the same weight of fine crusher dust. Modern testing laboratories tend to dismiss the old field settling test (BS 812, 1960) in favour of washing over a 200 mesh (75 micron) sieve. However, the silt percentage by weight, accurately measured by the latter test, is only half the story. The settling test appears to integrate both factors and give good proportionality to the increase in water demand. This is an excellent example, important for the younger student of concrete to remember, of a relatively inaccurate measurement of the truly relevant property being preferable to very accurate measurement of an only partly relevant property. Another example of the need to assess relative importance relates to fine aggregate testing in general. The inaccuracy in determination of either silt content or grading is often less important than the difficulty of obtaining a truly representative sample. Some fine aggregates are extremely consistent throughout a given delivery but others, particularly if conditions are primitive, can vary greatly. Where variation can be significant, frequent relatively rough testing may be more valuable than the same expenditure on fewer very accurate determinations. If a test indicates a significant change, the first action should be to repeat it on a second sample for confirmation.

Chapter 7 should be consulted for more detail, particularly of the use of marginal or sub-standard fine aggregates.

3.9
PACKING CONSIDERATIONS

At the time of writing the first edition it was seen that particle shape in coarse and fine aggregates had quite different effects. Poor shape in a coarse aggregate required an increased sand content whereas poor shape in a fine aggregate increased water requirement. It is now seen that this is the same effect in both cases, i.e. a badly shaped (or badly graded) material will have a higher percentage of voids and will therefore require more of whatever has to fill those voids. In the case of coarse aggregate, the filler is mortar and, if water and cement contents are considered fixed by other considerations, the variable material is the fine aggregate. In the case of fine aggregate, the filler is cement paste and if cement is fixed by other considerations, the variable material is water.

The above is an over-simplified view since more water will require more cement at a given strength and more sand will increase water requirement. However, these additional effects are automatically taken care of by the system when the two basic adjustments are made.

Understanding this situation resolves the divergence in water content calculation (in the first edition) between the author's specific surface technique and the methods of Dewar (section 2.9), and it does so in favour of Dewar. As the sand becomes coarser, it is likely to contain more voids and specific surface will also call for an increased percentage of the sand. Furthermore the lower the cement content and the higher the specific surface required to provide a given MSF (degree of cohesion), and therefore again the higher the sand content. So we have more and more sand voids to fill and less and less cement to fill them. This leads, as Dewar (section 2.9) has always said, to an increase in water requirement at very low cement contents—the opposite effect to that obtained by pure surface area considerations. However, the additional void content may be filled with entrained air when this is used at low cement contents.

At high cement contents, there will be insufficient sand voids to contain all the cement and therefore there will be excess cement paste and therefore excess water over that calculated on surface area grounds. The author now concedes that his calculated water content will generally only apply with middle range cement contents, certainly 300 to 350 kg/m^3 and often 250 to 400 kg/m^3. Outside this range, additional water should be allowed in calculations. A suggested figure is 2 litres per 10 kg of cement outside the middle range. But which middle range, 300 to 350 or 250 to 400? This probably depends on particular particle shapes and gradings. The current Conad system allows the user to nominate both the rate of increase of additional water and the range outside which it will apply. Furthermore, if the user inputs actual data of water content together with cement content, strength, and density, the system automatically revises its prediction equations to accord with the input data.

The author's view is that the advantage lies with using a quick simple system and obtaining accuracy not by precision of theory but by feeding back actual data which the system has been designed to accept and incorporate corrections for. What matters is the quick revision ability for changes in sand grading and cement content that is conferred by the specific surface approach. Also the ability to include specific surface in the QC graphing system, so as to explain observed results prior to such changes.

3.10
TEMPERATURE/TIME EFFECT

Water requirement for equal slump at time of use increases with temperature. The actual phenomenon is that cement hydrates more rapidly at higher temperatures causing greater slump loss, even in the first few minutes (Mather, 1987). The effect is not linear but rather increases at an increasing rate as temperature rises. As with air content, a purely empirical term is used in the current system. In fact in both cases there are two terms using an arbitrary constant together with the first and second power of the variable in question. The terms used are:

$$\text{Water adjustment due to temperature} = 0.02 \times \text{temp}^2 - 0.1 \times \text{temp}$$
$$(\text{temperature in } ^\circ\text{C}).$$

Popovics (Popovics, 1992) gives the relationship shown in Fig. 3.1.

When correctly viewed as a slump loss effect it becomes apparent that the above term in the water prediction equation is some distance from the ideal answer, in spite of having given good results in the past. The term should take into account both time and temperature. Ideally it should be based on 'equivalent age' (section 12.1.3). It also becomes obvious that the term should be different for different cements and admixtures. Whilst the need for revision is clear, data are simply not available for a reliable immediate revision. The author has originated the concept of an 'equivalent slump' for general use (Day, 1996b). The concept is that a slump, which should always be accompanied by a recorded temperature and time, should be converted into the value it would have given had the concrete been tested 30 min after mixing and at a temperature of 20°C. This will hopefully at least partly overcome the difficulty experienced by Shilstone (1987) of extracting any value from a slump test, since much better correlation with strength can be anticipated.

Where test data are being entered into a spreadsheet or other computerized system, no effort whatever will be involved in the transformation once the relevant formula has been input. A field technician would require either a notebook computer, or a rule of thumb determined in advance for the particular concrete, in order to be able to attach appropriate immediate significance to his measurement of slump.

If the above dissertation achieves nothing else, it should at least make it clear that the specification of an absolute slump from the strength viewpoint and/or the rejection of concrete for minor infringements of such a specification is not reasonable. However, slump limits at the point of use of the concrete may be reasonable from the viewpoint of segregation or bleeding tendency.

The effect on the Conad system will be that the temperature term will disappear, being incorporated in the slump term. However, since the latter is currently purely empirical, it will no doubt require amendment. (At the time of writing of the second edition the author has still been unable to persuade clients to try this approach but remains convinced of its potential value).

3.11
WATER FACTOR (INCLUDING WATER-REDUCING ADMIXTURE EFFECT)

As noted earlier, a basic tenet of the system is that it should be able to 'learn from experience'. One of the ways in which this is made possible is the use of a 'water factor'. This is a constant by which the theoretically calculated water requirement must be multiplied in order to obtain the actual water content.

Part of the philosophy is that if the actual water content of one or more batches using a given set of ingredients is accurately established, then the water factor can be calculated and applied to all other batches using the same ingredients. The water factor, used in this way, could be seen as automatically allowing for

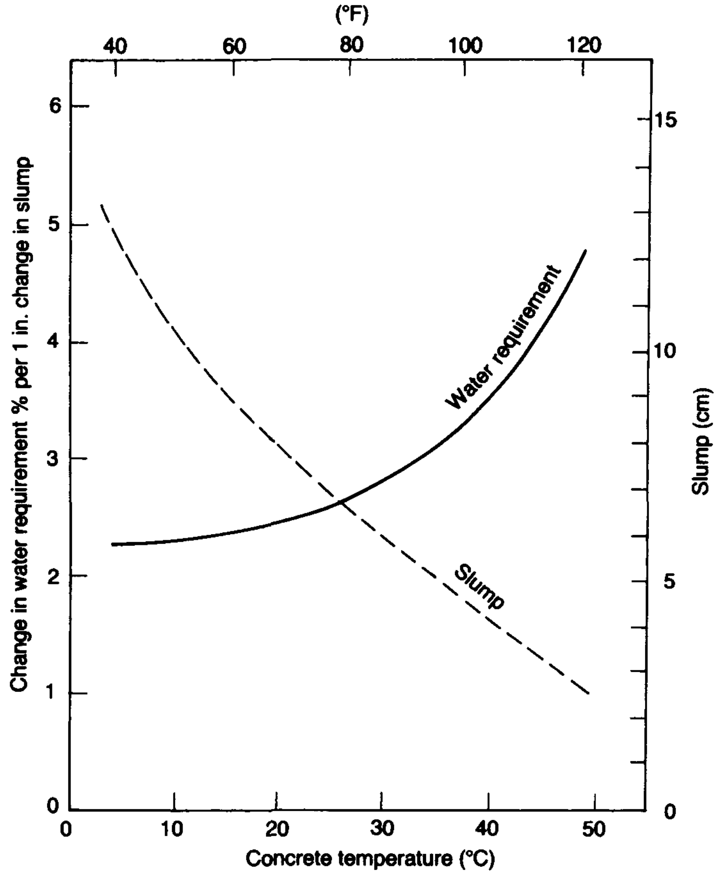


Fig. 3.1 Effect of temperature on water requirement.

all influences not separately accounted for, as well as any inaccuracies in the theory or assumptions. These would include particle shape and admixture use.

In fact the situation is in some ways better and in some ways worse than the foregoing suggests. The value of the factor would only be accurately transferable if the terms in the prediction equation were correct except for consistently different features of particular materials. If the assumed effect of temperature or slump were inaccurate, the value of the factor could not be accurately transferred.

On the other hand the range of effects caused by particle shape and by various admixtures are reasonably predictable. This means that a reasonable prediction of water requirement can be made in most circumstances, rather than being solely dependent upon the feedback of an actual value. Each individual influence which might affect the value of the water factor should be separately considered, and they should then be combined, i.e. if influence A gave a 5% reduction and influence B a 7% increase, the overall factor would be 0.95×1.07=1.02 (approx.).

3.11.1 Admixtures

In considering what actual numbers to select, it must be borne in mind that the effect of entrained air is to be separately allowed for. Water reducing admixtures usually gain part of their reduction by entraining a small percentage of air (perhaps 2%). The water reduction figure to be entered in respect of such an admixture will be the **additional** reduction over that given by using a pure air entrainer to entrain 2% air. Typical figures are 4 to 6% rather than the 10 to 13% likely to be claimed by the admixture purveyor as the total reduction.

Superplasticizing admixtures (high range water reducers) may not entrain any air and may give as much as 20 to 30% water reduction at high dose rates. In general, normal water reducers have a set or only slightly variable dose rate and therefore give a fairly specific water reduction. Superplasticizers on the other hand can be used over a wide dosage range to give a desired degree of water reduction. Such admixtures are often added on site after the concrete has already been mixed. The mix may be designed by omitting the effect of the superplasticizer in selecting the water factor and designing for the initial slump **prior** to adding the superplasticizer. Alternatively the water factor may be reduced and the design based on the slump **after** addition of the superplasticizer. Either method should give the same total water figure and the identical mix design. The addition of a superplasticizer at a given w/c ratio should achieve marginally higher strength due to better dispersion and better compaction.

3.11.2 Angularity and fines

The effect of angularity in fine aggregate can range up to 9% water increase for a badly shaped crusher dust (with the actual dust content still separately allowed for by the settling test and the grading by its specific surface). A figure of 7% may be more normal for good crusher fines and 2 to 4% for a very angular ('sharp') natural sand. A very rounded fine sand, such as a wind-blown dune sand, can act like ballbearings, effectively lubricating a mix, whereas its grading may suggest a substantial water requirement. Such a (relatively rare) situation may be better handled by an arbitrary reduction of the order of 5% in the specific surface value calculated from the grading. This will cause a higher proportion of such a sand to be permissible according to the system. It would only be done if the fine rounded ('dune') sand were cheaply available, but this is normally the case with such sands. The figures quoted are from the author's own experience. It should be noted that hearsay evidence from experimenters with the sand flow cone (section 3.14) suggests that angularity and surface roughness can add over 20% to water requirement.

3.11.3 Coarse aggregates

The effect of particle shape in coarse aggregates is rather different. There may be some small direct effect on water content (say, 1 to 2% plus or minus) but the main effect is to vary the desirable fine aggregate percentage. A very badly shaped crushed coarse aggregate or a contractor's strong preference for oversanded mixes can require the use of an MSF value (section 3.1) 1 or 2 units, perhaps even 3 units, higher than would otherwise be used. This would increase the sand content and so increase water requirement. Similarly a very rounded coarse aggregate can justify reducing the MSF value by 1 or at most 2, compared to standard (which is taken as a well shaped crushed aggregate).

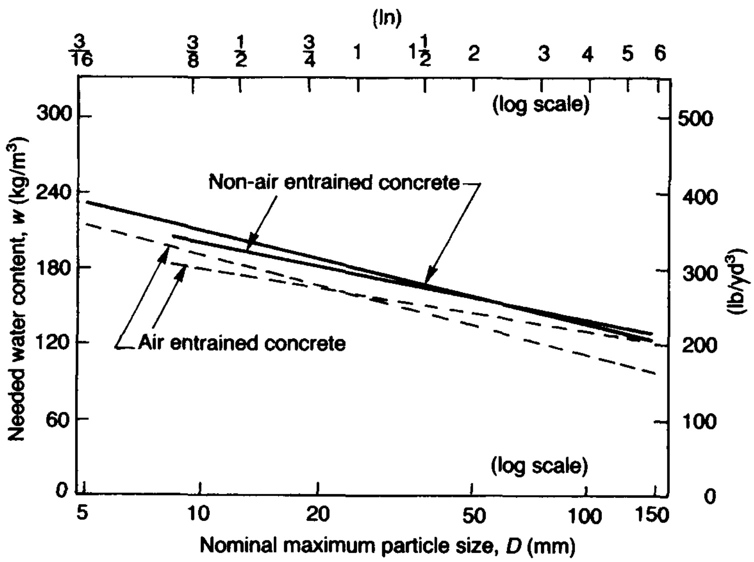


Fig. 3.2 Effect of maximum aggregate size on water requirement.

The effect of smaller or larger coarse aggregates is similar to that of particle shapes. The standard is taken as 20 mm. A reduction in MSF would be made for larger aggregates and an increase for smaller. The variation may be 1, or rarely up to 2, in MSF value. As a further alternative, Fig. 3.2, which is due to Popovics (1982) can be used.

In the first edition, an unusually high or low water requirement of the cement was to be accounted for in the water factor. It is now regarded as preferable for each cementitious material to carry its own water adjustment and this is based on the w/c ratio for normal consistency in cement testing. An overall water factor is still required but it is easier to keep track of the effect of any new material if the normally anticipated value is affected almost entirely by admixture usage. The system makes provision for the separate insertion of a direct water reduction or increase in terms of litres of water per 100 kg of cementitious materials. For example most but not all fly ashes will normally cause a distinct water reduction, perhaps more than 10 litres per 100 kg of ash.

Silica fume requires caution. It should almost never be used without a superplasticizing admixture (a possible exception being for shotcrete). In the absence of a superplasticizer the silica fume is likely to cause a substantial increase in water requirement. However, when used with a superplasticizer, water requirement may be lower than with the superplasticizer but without the silica fume. The effect may be visualized as similar to tiny ball-bearings with and without oil. Perhaps more importantly, the function of the silica fume is to disperse so effectively as to fill the interstices between the cement particles. It is doubtful that this can be accomplished without the use of a superplasticizer, especially if the silica fume has been condensed (i.e. deliberately flocculated). There is in fact some concern that, even with a superplasticizer, full dispersion may not be achieved. For this reason additional mixing time should be allowed, especially where 'soluble' paper bags of silica fume are added whole. The author has seen cores from structures still containing visible pieces of such bags.

Somewhat the same effect, but at a much reduced magnitude, applies to sand and superplasticizers. The latter can be used purely as high range water reducers in normal slump concrete. In this case there is no effect on the mix design other than water reduction. However, when used to produce high slump, and especially fully flowing concrete, there is a distinct effect on desirable sand content. To some extent a superplasticizer uses the fines in the mix to create fluidity. If there is a shortage of fines, the water reduction, or at least the desired fluidity, may not occur. Worse, the mix is likely to segregate at higher dosages. So, in a superplasticized mix, a high MSF (i.e. a very sandy mix) is both necessary to resist segregation and does not lead to as much water increase as usual.

3.12
POZZOLANIC MATERIALS AND SLAG

This section basically refers to fly ash, ground granulated blast furnace slag (ggbfs) and silica fume. However, the remarks can apply equally to a second cement or to any other material such as rice hull ash or lime.

The preceding section has already made reference to the effect of such materials. The system makes full provision for any likely effect of any such material. Each requires the entry of:

1. Batch weight.
2. Specific gravity (relative density).
3. Cementitious value (i.e. strength effect) as a ratio to that of the primary cement.
4. Effect on cohesion (i.e. effect on MSF) as a ratio to that of the primary cement.
5. Water reduction, litres per 100 kg of the material (negative values being permissible).

If a range of mixes with and without any particular material is in use, the true values of these factors should be fairly readily apparent. Specifically, when a new material is introduced for the first time into a mix of which the performance is well established, the performance of the revised mix will enable the values of the factors to be determined. The assumption is that these values are constants for the materials in question, so that they can then be used in designing other mixes.

In fact it is not to be anticipated that a truly constant strength factor for a pozzolanic material will be experienced over a wide range of mixes and cement contents, but it is worthwhile to make this simplifying assumption.

3.13
OVERALL WATER PREDICTIONS

The preceding effects are combined into a single equation as set out in Table 3.5. The constants in the formulas are built into the computerized version of the author's mix design system and have given good results. However, it is suggested that the principles on which the system is founded has a usefulness and validity independently of whether the particular constants are correct.

Two stages might be considered in any attempt to improve system accuracy. One of these is simply to originate different factors. The other is to attach a (different) constant multiplier to each of the terms. In the latter way a computer optimization could be undertaken when a large range of results are in hand. The author in fact has made provision to do this.

A particular modification of one of the above factors is the influence of slump. Popovics (1982) has an elegant approach to this involving a 'thinning factor' *K*. This factor is evaluated as:

$$K = \left(\frac{\text{slump } 1}{\text{slump } 2}\right)^{0.1}$$

Since a ratio of the two slumps is involved, they can be in any units (i.e. in, mm, cm). The relative water contents corresponding to slumps 1 and 2 are then given by:

$$w2 = K \times w1$$

This equation is considered to be much more fundamental than appears from the above so that the same *K* value will be obtained for different tests (e.g. flow table and penetration tests) but with the ratio raised to a different power. Readers are referred to the Popovics reference for a very comprehensive dissertation and justification of the above.

Table 3.5 Effects of various factors on water requirement

Source of effect	*Effect on water requirement (ls/cm³)*
basic water content	85
grading effect	+3×EWF
slump effect	+0.36×(slump) −0.0007×(slump)^2
entrained air effect	−5A×250/total cementitious content
concrete temperature (°C) effect	−0.1×(temp) +0.02×(temp)
silt content effect (combined sands)	+ [(silt % −6)×(wt sand)]/300
2nd cement/pozzolan	−factor k_2 × wt of material
3rd cement/pozzolan	−factor k_3 × wt of material
quantity of cement	+ entered factor × amount out of entered range
	= SUM

SUM×Water Factor=Total water requirement (excluding absorbtion)

3.14 FINE AGGREGATE WATER REQUIREMENT RELATED TO VOID PERCENTAGE AND FLOW TIME

The subject of water requirement cannot be left without discussing the sand flow cone test and void %. The author has not had an extensive opportunity of examining this technique in practice but recognizes that it has the potential to provide an amendment to his modified specific surface. Any such amendment would cover the effect on water content of fine aggregate particle shape and of irregularities (gaps, or excesses of single sizes giving particle interference) of grading as distinct from fineness or coarseness.

The technique of the sand flow cone originated in New Zealand (Cleland, 1968; Hopkins, 1971) and independently in the USA (Gaynor, 1968; Tobin, 1978). It has also been examined in Australia (Elek, 1973).

The test consists of pouring a fixed amount of dry fine aggregate through a standardized metal funnel into a container below (Fig. 3.3). The two measurements taken are of the time taken for all the material to pass through the funnel and of the bulk density or percentage voids of the sand in the container at the conclusion of the test. (Hearsay evidence suggests that flow times are significantly affected by the sharpness of the edge of the orifice so that anyone making their own cone may obtain different results from any which have been published. Figure 3.4 shows the extent of the differences which may be experienced).

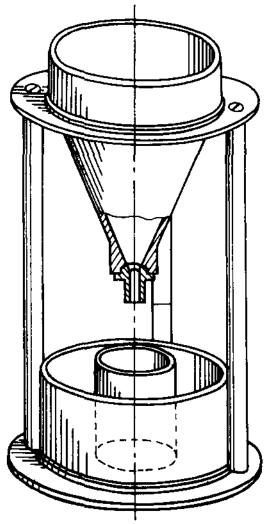


Fig. 3.3 Sand flow cone apparatus.

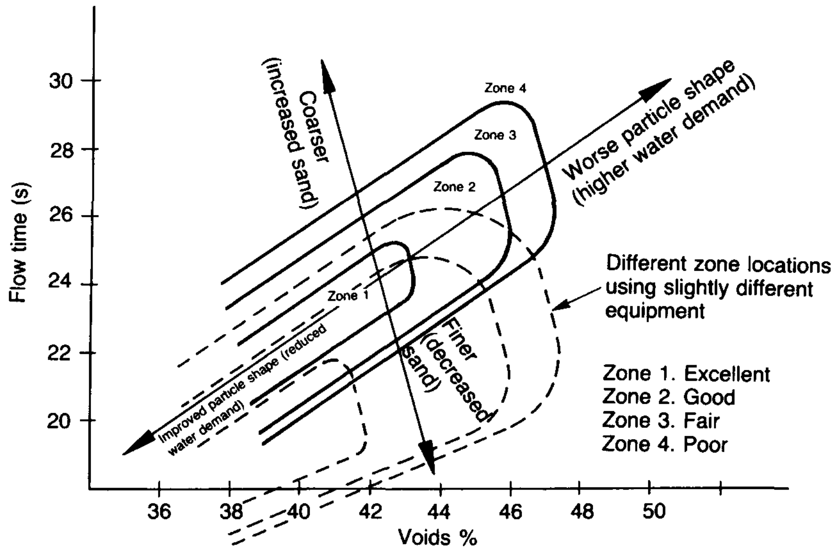


Fig. 3.4 Sand flow cone analysis diagram.

The properties which affect both the flow time and percentage voids are the particle shape, texture and grading. There is quite a good corre lation between these two properties and American researchers appear to consider that they are essentially alternative ways of measuring the same property (Gaynor, 1968; Tobin, 1978). On the other hand, the New Zealand researchers consider departures from the basic relationship to be very significant (Cleland, 1968; Hopkins, 1971). Both tests can be used to measure changes in grading at a constant particle shape with finer sands flowing more quickly and having a higher percentage voids. However, an increase in voids content at a constant flow time is an indication of increasing angularity or surface roughness, or possibly an unfavourable combination of sieve sizes.

Several researchers into the flow cone test conclude that it could be used as a replacement for fineness modulus (determined from sieve analysis). However, this idea does not yet appear to have been used as the basis for a mix design system.

The percentage voids could be used, as proposed by ACI 363 (ACI, 1992), to amend water content predictions. Perhaps a more accurate basis for the adjustment would be the percentage voids less the flow time effect. The latter would presumably remove the influence of grading fineness (for separate treatment) and so more accurately measure the other influences (particle shape, surface texture and grading incompatibilities). The test is also valuable as a criterion of optimum proportions when blending two fine aggregates.

Figure 3.4 shows the type of analysis diagram originating in New Zealand as refined in Australia. Two sets of suitability envelopes are shown to indicate the substantial differences which can occur using different (but nominally identical) apparatus. Clearly it would be necessary to calibrate a newly fabricated funnel with at least four fine aggregates having:

1. FM greater than 3.0.
2. FM less than 2.0.
3. Very rounded sand.
4. Very angular sand or crusher fines.
5. A sand considered ideal.

The test offers a quicker and simpler means than sieve analysis of detecting changes in grading during production use of a sand. In addition it simultaneously checks for any deterioration in particle shape or surface texture. The latter may be considered fairly unlikely to change for a natural sand from a particular location but would be well worth monitoring for crusher fines and would be very difficult to check by any other means.

As the factors monitored can have as much as 20% influence on water requirement (and therefore cement requirement at a given strength) the test would appear to have very substantial economic implications. It is surprising that it is not widely used by those concerned with selecting fine aggregate sources for purchase or development.

A further use for the sand flow cone is in blending two sands. It *is* extremely simple to carry out a set of flow and voids tests with varying proportions of two sands and a graph of each of these properties against percentage blend is very revealing as to the range of compatible proportions.

3.15 STRENGTH CONSIDERATIONS

Basically, the strength of concrete has a close to straight line relationship with **cement/water** ratio. There are two other important influences to consider. One is that it should be water plus voids, not only water. The author's experience is that higher air contents make a larger difference in high strength concrete. The other is that it is well known that strength cannot be infinitely increased by merely increasing cement content.

There are several old-established proposals for a strength prediction equation including:

Abrahams: Strength =

Bolomey: Strength =
Feret: Strength =

Of these, the oldest (1894) and probably the best is Feret, since the others do not take account of air content. In the first edition the Bolomey form (25c/w—8) was suggested as simplest to use over a restricted range and a more comprehensive formula:

$$\text{Strength} = 25c/[w + 0.4(\text{air }\% - 1)\sqrt{c}] - (s/250)^3 - 5 \tag{3.4}$$

devised by the author was incorporated in the actual Conad system. In each case it was suggested that modifying strength multiplication factor be fed back from actual test data.

Recently the author's Mixtable program (section 3.18.3) incorporated all three of Feret, Abrahams, and the author's long formula together with a curve-fitting facility for varying the constants of each to match input strength data. The program displays the graphs of the amended formulas, plotting the input data points. It was found that all three of the equations can be amended to give an almost perfect match to any accurate actual data. Only extrapolation beyond the range of input data shows any substantial difference between the three adjusted curves. It is apparent that it is not useful to have the three alternatives, one would be sufficient. Recently it was decided to use only the Feret formula. Formula 3.4 gave slightly better results, but not enough to supersede the old-established form.

Publications by Dewar (1999) and de Larrard (1999) should be consulted for an in-depth examination of published data on more exact formulas. Dewar quotes Sear as finding that the relationship shows a change at w/c greater than 1.2 and several workers as finding that a change is shown at w/c below 0.4 or 0.38. Dewar also quotes Erntroy on the subject of ceiling strengths for Thames river gravel. However, the present author recalls presenting Erntroy with a cube which registered over 13000 psi (over 90 MPa) a few days after the 1954 symposium, which exploded that particular theory.

De Larrard introduces a concept of maximum paste thickness (MPT) which appears to account well for the effect of excess paste and larger aggregate size on lowering the strength otherwise to be expected of a concrete. He also provides an interesting and convincing examination of the different performance of five widely differing aggregates.

It is not the purpose of the current volume to delve deeply into the above matters. Some guidance is provided at the end of this chapter on the production of high strength concrete but Conad is concerned with the accurate adjustment for maximum economy of mixes in production use rather than their accurate theoretical design. This is not to denigrate the work of such practitioners as Dewar and de Larrard. While the ability to design mixes accurately from first principles may or may not be of great value, the ability to realize why your particular concrete is not giving good results is unquestionably valuable.

Not all cements are created equal, and the whole equation can be considered to be multiplied by a strength factor. The value of this factor will be taken as 1 when there is no information to the contrary and it may vary between about 0.8 and 1.3 in the author's experience to date.

Following this same principle, it is useful to consider supplementary cementitious or pozzolanic materials as 'equivalent cement' so that

$$\text{EC} = c + k_1c_1 + k_2c_2, \text{ etc.,}$$

where EC is 'equivalent cement' (replacing c in the previous equation) c_1, c_2, etc., are masses of cementitious or pozzolanic materials. Here k_1, k_2, etc., are coefficients determined from experience (Table 3.6). However, as noted above, every cement can have a strength factor other than 1 as necessary to obtain the correct strength prediction. Subsequent to the first edition, it has been found preferable to store the appropriate strength factor with each material rather than storing a ratio between the supplementary materials and the main cement. So then we have:

$$\mathrm{EC} = k_1 c_1 + k_2 c_2 + k_3 c_3, \text{ etc.,} \tag{3.5}$$

This is suitable for use with the strength equation but to express the equivalent cement content as a stated amount, or to use it in a stated w/c ratio, it would be necessary to divide through by k_1, to obtain:

$\mathrm{EC} = c + k_2/k_1 c_1 + k_3/k_1 c_2$, etc. (enabling the *k* values to be read from data files)

The concept of equivalent cement content raises two problems. One of these is whether 'the water to cementitious materials ratio' should use the total mass of all cementitious and pozzolanic materials or the mass of **equivalent** cement as defined above. From the strength viewpoint, the latter would be appropriate. However, it is not normal practice and so may be misunderstood. Also from the durability viewpoint, the total mass may be a better indicator.

The other question relates to the second term in equation (3.4). Should the cement content there be replaced by equivalent cement? The answer depends on why it is that the term is necessary in the first place. The author's initial tentative explanation was that the hydration of cement liberates substantial quantities of calcium hydroxide. Such material

Table 3.6 Pozzolanic material coefficients

Material	*Effect on water reqd. (l/100 kg)*	*Likely range of strength*	*k values cohesion*
cement	+/−5	0.8–1.2	0.9–1.1
fly ash	+5 to −10	0.6–1.1	0.9–1.2
ggbfs	+5 to −10	0.7–1.1	0.9–1.2
silica fume	+25 to −5	3–4	3–1

would be the weakest constituent of concrete. A point could therefore arise in the increase of cement where the additional calcium hydroxide reduced strength more than the lowering of the w/c increased it. If this was truly the explanation, then, since pozzolanic materials **make use** of the calcium hydroxide, they should not be included in the term which limits strength increase at high cement contents. The author is now persuaded that de Larrard's MPT (maximum paste thickness) theory is a more likely explanation. This relates the strength reduction to the excess paste content over the amount to fill all aggregate voids. Therefore not only should all cementitious material count but also any increased water or air content. At this stage it is not proposed to make extensive changes in the Conad equation since, with feedback adjustment, it is quite accurate enough for most practical purposes. However, it is now recommended that the second term in equation (3.4) should incorporate all cementitious material.

The conversion of cylinder strength to cube strength is considered in section 1.6 (note that the British Standard figure of cube strength=1.25 ×cylinder strength is *not* realistic).

3.16
YIELD AND UNIT WEIGHT

It is important to realize that the calculated unit weight is in a different category than the water content and strength calculations, both of which include several assumptions and approximations. Unit weight cannot fail to be exactly correct if the assumptions on which it is based are correct. These assumptions consist solely of the relative quantities and specific gravity or relative density of the constituent materials.

The relative quantities of the solid materials are usually quite accurately known. This means that any departure from the anticipated (i.e. calculated) unit weight of the concrete is usually due to the water or air content differing from that assumed; alternatively inaccurate relative densities have been used.

Since cement is the heaviest material in normal concrete, and air and water the lightest, any reduction in concrete density will almost certainly be accompanied by a strength reduction. It is possible to calculate the likely correlation between strength and density depending on which of these three possible causes is considered most likely:

1. A lack of compaction or increased air % is likely to cause a 4 to 5% strength reduction per 1% of density reduction.
2. Assuming a normal density of 2400 kg/m^3, each 1 kg of cement (SG 3.15) variation will cause a concrete density change of (3.15–2.4)/ 3.15=0.75/3.15 or 0.238 kg. If it takes 5 kg of cement to cause a 1 MPa strength change, then a 1% density change (24 kg/m^3), if due entirely to a cement content variation, would correspond to 24/0.238/5 = 20.2 MPa, which could exceed 50% strength change.
3. Similarly a change of 1 litre in water content would cause a density change of (2.4–1.0)=1.4kg/m^3 so that a 1% (24kg) change in density would indicate 24/1.4=17 litres change in water content (approximately 1%) and would be equivalent to a change of say 34 kg of cement (at 0.5 w/c ratio) and be likely to cause a 7 MPa or say 10 to 25% strength change.

What this shows is that density is comparatively insensitive to cement content changes, but this is in any case the least likely cause of the variation. Increased water content is perhaps the most likely cause of a density reduction. It can be seen why density changes tend to have considerable strength significance.

The practice of rounding off density determinations to the nearest 20 kg/m^3 is worth reconsideration. It would certainly not be realistic to claim a higher accuracy than this for an individual determination. However, a cusum graph of density can detect a change of less than 10 kg/m^3 in **average** density (even using the rounded data) and this valuable ability is hindered by the practice of rounding. It is important to consider the subsequent use of data before rounding them. Conversely it is also important to consider the origin and reliability of data before taking action based on them (section 11.6).

3.17
TRIAL MIXES

Another quite important change of attitude is towards laboratory trial mixes. These may still be useful for some kinds of research and development (R&D) work but it may be that the time has come to dispense with them in connection with production concrete. There are two different problems with trial mixes. One is that their accuracy or repeatability (on different days and with new samples) may be inadequate. The other is that a given mix design may produce different results in a laboratory mixer and in a ready mix truck. This is particularly the case when the dispersion of materials such as silica fume and the timing and sequence of batching, including liquid admixtures, may be of significance. However, Mixtune does include a program to assist with trial mix work.

It is a rare experience for a designer to be confronted with a totally unknown set of materials with the only available data being grading, specific gravity (SG), cost and visual appearance. (Such a situation may be dangerous from the point of view of chemical impurities, susceptibility to alkali-silica reaction, etc., but this aspect is not considered here). When this happens to the author he is still confident of the system's ability to select the most advantageous combination of aggregates and for the first concrete produced to be a truck of concrete used in the actual structure. Of course the design has to be conservative in such circumstances, usually being 5 to 10 MPa (say, 1000 psi) stronger than really needed and with 2 or 3% more sand than strictly necessary. Also an opportunity should be taken to supply a truck of any high strength concrete to a foundation or other lower strength and non-critical location several days or weeks before it is actually needed in the columns, or wherever the high strength is actually needed.

A more usual situation is that the materials to be used, or most of them, will already be in use by others. It may be possible to obtain mix details and test results on such production concrete. It may even be possible to see this concrete in use. If available, such data can be entered in the computer system and will establish the correlation factors, the lack of which may otherwise result in slightly uneconomical concrete being used for as long as 4 or 5 weeks.

Where a single new material (whether it be an admixture, a cement, or an aggregate) is being considered for introduction into an operating plant, the most advantageous situation is if it can be available in sufficient quantities to make one or two trucks per day containing it for some weeks prior to its full introduction. Each truck would be sampled and tested with far more accurate and relevant results than laboratory trials. This process is much assisted by the comprehensive nature of the mix design/evaluation process. It is much easier to see how the new material compares when variations of slump, temperature, density and even cement content and changes of grading or silt/clay content in other materials have been compensated for by the system.

The archaic situation is still encountered where a public authority (for example) demands a trial mix before permitting a small mix revision for concrete in continuous production use. Suppose for example that a strength of 40 MPa is required and that 44 MPa is currently being obtained. The mix control program may indicate a cement reduction of the order of 25 to 35 kg/m^3, based on the cement content which had been used to attain the current 44 MPa. It is very unlikely that a 25% error would occur in estimating the amount of cement required, yet even such an error would only represent a 1 MPa inaccuracy in attaining the required strength. On the other hand it is unlikely that a laboratory trial mix would give a strength within 1 MPa of the same mix in production. It can be seen to be ridiculous to require the laboratory trial before permitting the adjustment, yet such a situation has been encountered in locations as far apart as New York City and SE Asia. A more sensible course is to require a conservative basis to be used for a calculated adjustment. For example, the author's practice is to assume a requirement of 10 kg/m for each 1 MPa of extra strength required in an understrength mix but to permit a reduction of only 5 kg/m for each 1 MPa of excess strength when reducing the strength of an over-strong mix (8 kg increasing and 4 kg reducing may be better where cement is very good, or 10 kg up and 4 kg down if you want to be very conservative!). This has the effect of making downward strength adjustments in at least two conservative steps but immediately making up for any shortfall in strength and then approaching the desired ideal value from above.

3.18 IMPLEMENTATION SYSTEMS

The specific surface concept has now been incorporated into several systems designed to fulfil different functions:

1. **The basic system,** suitable for designing and controlling a single important mix in continuous production and heavily integrated with the Conad QC system.

2. **The Automix system,** a very user-friendly system suitable for the rapid initial design of a single mix by an inexperienced person.
3. **The Mixtable system,** a system able to rapidly generate a whole table of mixes with cementitious contents from 200 to 500 kg/m^3. The table can mimic a supplier's existing table of mixes, use a directly input specification, or import and expand a single mix from the basic or Automix programs. The system can also automatically adjust its strength and water requirement predictions to accord with input real data and can automatically cost optimize aggregate proportions.
4. **The Cement Margins system,** a system able to extract from the Conad QC system a large mass of data covering many aggregate and cement groups in use for a wide range of mixes at many plants and to analyse this for conformance to prespecified strength targets. The relative performance of different materials is clearly shown and the intention is to enable accurate adjustment of a huge range of mixes to minimize cement contents throughout the range.
5. **The Benchmark system,** a system similar in some ways to Cement Margins but aimed at examining the performance of mixes in a comprehensive manner, not only relative to strength targets.

It is envisaged the Cement Margins or Mixtable program might be used by a company's area technologist to attain minimum cost operation, while Benchmark might be used by a large, possibly multinational, company's head office to compare the performance of all its areas and materials (and its area technologists!).

3.18.1 The basic system

The whole intention of Conad Mixtune is that it should provide the means by which mixes **in production use** are 'tuned', i.e. adjusted to give concrete of the desired properties more precisely and economically. To do this it is helpful to recognize that a given mix usually exists in three forms:

1. As the designer intended it to be.
2. As it was actually made.
3. As the designer now wishes it to be.

The difference between the first and second forms arises from inaccuracies in batching (including water), differing properties of materials, and different climatic conditions. If the actual batch quantities, aggregate gradings, slump, concrete temperature, etc., are established, and revised predictions for water content and strength produced, it is then legitimate to compare these predictions with the actual test results. This permits residual adjustment factors to be fed back into the mix design system. In the Conad system these factors are called the 'water factor' and the 'strength factor'. The water factor will not usually have unit value as it incorporates effects such as the effect of admixtures. The strength factor can be regarded as essentially a cement quality factor and is to be taken as unity unless there is evidence to the contrary. A policy change from the first edition is that each cement and pozzolan should carry its own strength factor in the database so that the overall strength factor is intended to always be approximately unity. If it has some other value the cause should be examined and the strength factors of the individual constituent materials adjusted.

Using the newly established strength and water factors, it should then be possible to obtain very accurate predictions, i.e. to establish exactly what mix proportions are necessary to provide the desired properties.

Of course, the mix in use may also be considered to be too sandy (i.e. too cohesive) or not sandy enough. An adjustment to this requires only an adjustment to the input MSF value. All consequent adjustments to maintain correct yield and unchanged strength are made automatically by the system.

The major objective of recent developments in the Conad basic mix system have been to integrate it with computer batching systems, a database of all input materials, and with its own quality control (i.e. test result analysis) system. The ultimate objective of this integration is to have a concrete batching system which is able to automatically adjust its own mixes in accordance with input data on material properties (e.g. gradings, silt contents, cement quality) and test results on output concrete. So what you see and can currently use is a mix adjustment system for a single mix, but the importance of the system is that it is working towards being automatically applied to every single mix in the system.

If the system operates automatically, it does not matter how complex the calculation enabling it to do this. However, there is certainly a need for a simplified system to enable quick and easy manual use. Such a system would not necessarily produce inferior results in the longer term, if it provided inbuilt feedback correction.

Figure 3.5 shows the main screen of the basic system. The screen is capable of an almost infinite number of variations according to the wishes of the user. As printed, the left-hand side shows three columns entitled Design, Actual and Proposed (abbreviated). Design is the previously designed mix automatically extracted from file. Actual takes actual average batch quantities over any specified period from the stored batch plant data, actual material properties from the materials control files (also over any specified period), and concrete test data from the QC files (also over any specified period). The main objective of this is for the program to calculate two factors, the ratio of calculated to actual water content and of calculated to actual strength.

The Actual column, complete with strength and water factors, is then copied to the Proposed column. In this column almost anything can be done to the mix and the system will react according to various settings on the screen. Batch quantities can be changed and the screen will display the resulting properties. Properties (e.g. strength, MSF, yield) can be input and the screen will display the batch proportions required to achieve them (Fig. 3.5). Yield can be set to an exact cubic metre (enter 1000 litres) or any other desired figure, or allowed to vary according to input batch quantities. Temperature, slump, or air content can be changed and the effect on water content, density and strength will be displayed.

Next to the Design, Actual, and Proposed columns the material properties are displayed (e.g. SG and specific surface) each with different values, if appropriate, for the Design, Actual, and Proposed situation. This section of the screen extends under the next two sections and can be seen in entirety either by scrolling it or by switching off the mix property and graph overlays. The mix properties are largely self-explanatory. The three graph ‘thumbnails’ can be expanded to full screen depth by right clicking on them.

At the foot of the screen is water content, again in its three variants. Next to this are shown the water corrections calculated for each influence of which the system takes account (MSF, slump, temperature, silt content of sand, entrained air %, cementitious materials [properties and quantity]).

The system does include a correction mechanism to allow for particle shape in both fine and coarse aggregates but the correction is largely empirical and not anticipated to be as precise as use of one of the packing theory programs.

It can be imagined that a complex hierarchy of rules of precedence are involved ‘behind the scenes’ but the system is simple to operate once the user has ensured that all controls have been set as intended and the necessary data is present in the database. This program requires no expertise in concrete technology, but does require more care in operation than it sometimes gets. It is ideal for regulating a small number of

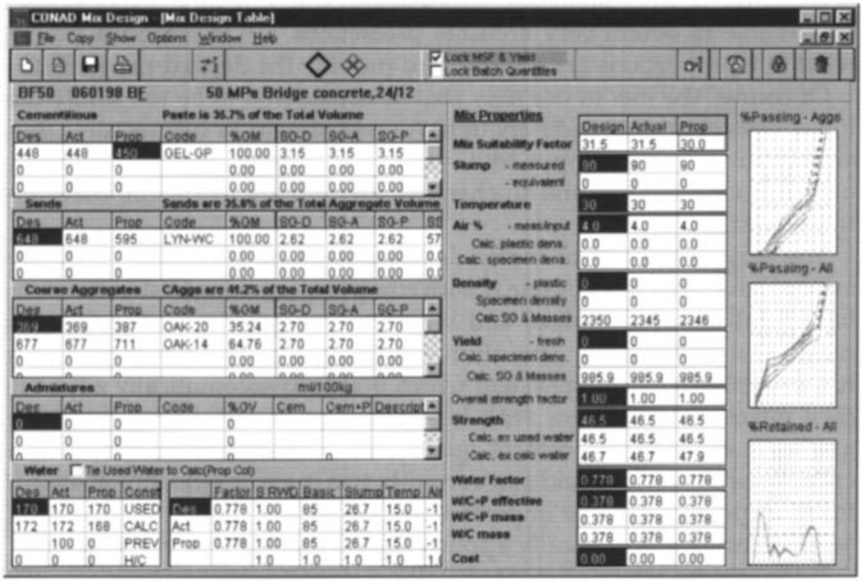


Fig. 3.5 Main screen of basic Conad mix system.

highly critical mixes in the on-site plant of a major project. (Full details of this system are available in the system manual on the supplied CD).

3.18.2 The Automix system

This computer program is aimed at providing a very user-friendly design program at the cost of some loss of features compared to the basic Mixtune program. The main features lost are feedback of production data and shape correction. However, feedback of test data is achieved in a different way through transference to the following Cement Tables program.

The program goes some way towards being based on ideal gradings for those who do not feel comfortable with complete freedom to nominate the relative proportions of several coarse aggregates or two sands to each other. However, it still uses specific surface to determine the ratio of total sand to total coarse aggregate.

In one concept, an ideal sand grading is one whose grading is normally distributed on a logarithmic scale, i.e. when plotted in terms of percentage retained on the normal sieve size X axis, the result is a normally distributed histogram. No numerical penalty is known to be incurred if the grading is not normally distributed, but there may be a greater risk of segregation, bleeding or increased water requirement. The question of suitability may be better assessed in terms of percentage voids or flow time (section 3.14) but the normal distribution concept may be of some assistance in assessing the optimum combination of two sands where neither of these tests is available. For any given mean size (i.e. logarithmic mean size) it is possible to nominate a desired percentage passing the 75 mm sieve or alternatively to nominate standard deviation or coefficient of variation. Any of these permits calculation of a family of normally distributed gradings, one for each mean size.

Another alternative criterion is the old UK sand grading zones (Fig. 3.7 and Table 7.3).

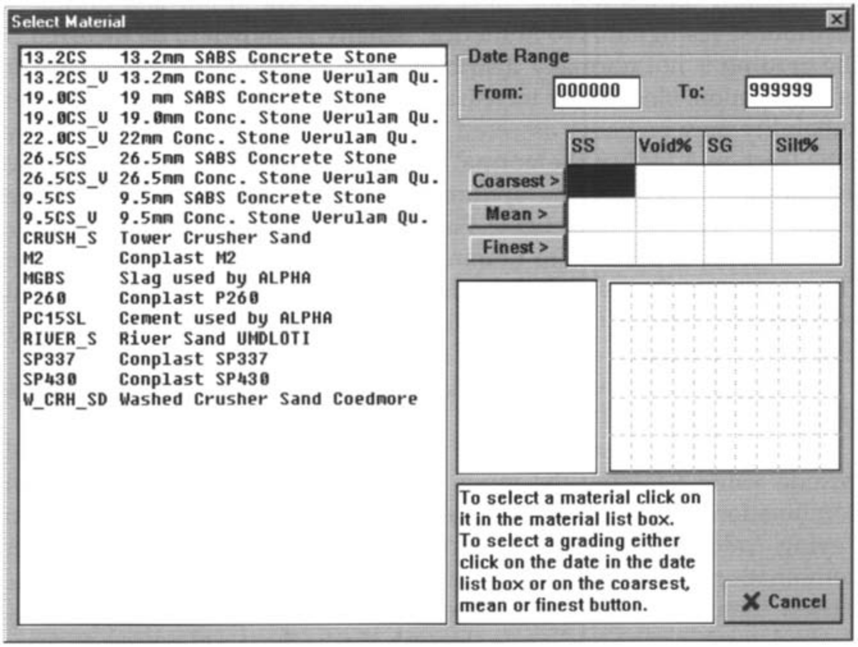


Fig. 3.6 Materials selection screen.

The Automix program is able to cycle through these alternative sets of criteria while retaining the input individual gradings and the current combination on screen. For any set of guidance curves, Automix, on keying 'calculate' will cycle through each curve and every integer combination of the two sands from 4 to 1 to 1 to 4 to find which gives the closest match to one of the curves. However, the user can input any desired combination and cycle through the background curves to form an independent opinion of its suitability.

For coarse aggregates no theory is advanced and the user merely selects from the available four options of continuous, semi-continuous, semi-gap, and gap. The curve resulting from the combination selected is shown superimposed on the four optional curves. Again the user is able to input an alternative combination.

The user now goes to a second screen (Fig. 3.8) where the mix will actually be designed. The desired type of concrete is specified in terms of its MSF from a pull-down menu. This menu describes the type of concrete which will be produced alongside each MSF number (e.g. Harsh Mix for low slump precast at 22 and Sandy Flowing at 30). However, these are to some extent matters of opinion and users should feel free to nominate their own preference of MSF number for the particular work in hand once they become familiar with the fresh concrete properties to be anticipated from a given MSF number (any desired number can be keyed in rather than selecting one from the table and the descriptions in the table can be edited by the user).

Appropriate values are entered for slump, air %, and concrete temperature. The default figure for water factor is 0.95 (i.e. a 5% water reduction appropriate to the use of a normal water reducer); this value may be overwritten as desired on the basis of the user's own experience with the proposed materials.

The program makes an assumption that the water requirement calculated will apply over a limited range of cement contents, which the user is able to specify. A conservative range is 300 to 350 kg but a wider range may apply. If a lower cement content is used, the assumption is that there may be an inadequate amount of cement paste to fill the voids in the sand; the program therefore assumes that a higher water content will result (Dewar, 1999). Again the user is able to specify a rate per 10 kg of cement at which

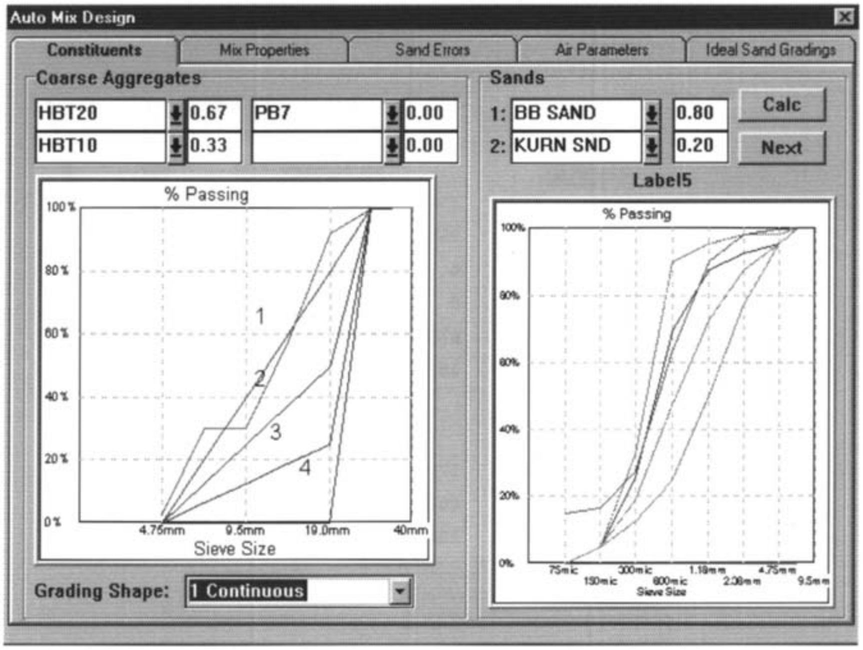


Fig. 3.7 Automix constituents screen.

water content will increase but a default value of 2 litres per 10 kg is suggested. If, on the other hand, cement content is higher than the 'ideal' range, there will be more cement than can be contained in the sand voids and this additional cement will require additional water. For simplicity, it is assumed that the same rate of increase as in the inadequate cement case will apply.

When air entrainment is employed, it is assumed that, in addition to reducing water content generally, the air will assist in filling the sand voids and so will avoid the increased water content otherwise to be anticipated in the low cement case (but not in the high cement case).

It now remains only to key 'calculate water' followed by 'calculate' for the program to proportion the mix. It does so by calculating the proportion of combined sands to combined coarse aggregates so as to yield the specified MSF. The program compares this result graphically with the grading resulting from combining the two curves the program was trying to match. The gradings resulting from both the calculated mix and the target grading are shown on three thumbnail graphs: percentage passing, aggregates only; percentage passing all materials; and individual percentage retained, all materials. Each thumbnail may be expanded by right-clicking on it. The expansion will revert on releasing the right-click key but may be retained on screen by moving the cursor off the expanded graph before releasing the key.

This system is intended to provide guidance and simplicity of operation for new or inexpert users. The mix designed can be saved in a database and recalled into the Mixtables program for expansion into a whole range of mixes. However, with practice and expertise, the user may go straight to the Mixtable program.

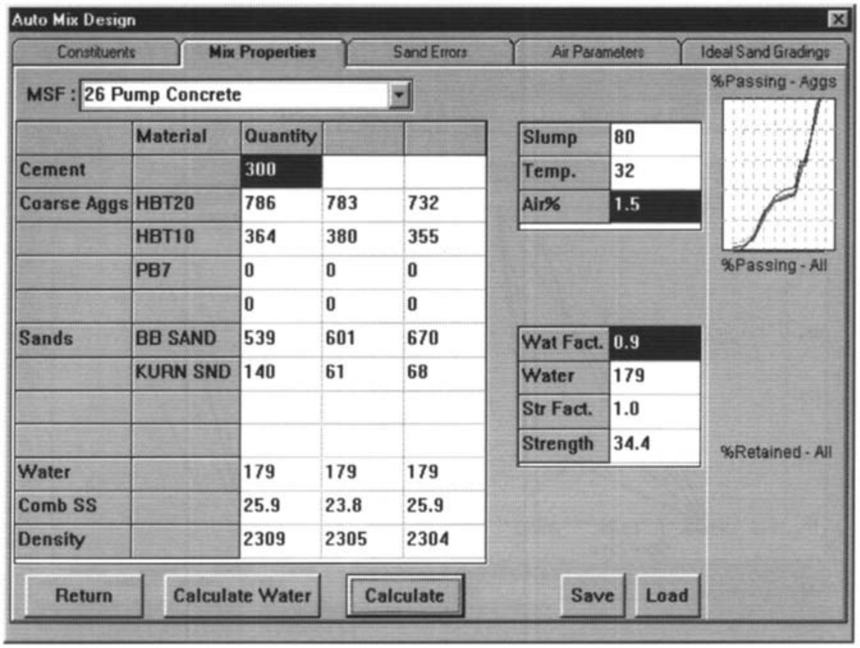


Fig. 3.8 Automix mix properties screen.

3.18.3 The Mixtable system

The author would like to see the practice of batching mixes from a table discontinued in favour of building a mix design facility into computer batching plants. However, the customer is always right and Mixtable is a program to generate a table with cement content ranging from 200 to 500 kg of cement (or cementitious material) in whatever cement content steps the user chooses, or in steps of 1 MPa. The program has value independently of actually being used to batch concrete, since it enables easy examination of a range of possible mixes and permits actual data to be compared, and used to modify the water and strength predictions of the system.

This system automatically produces a range of mixes from 200 to 500 kg content of cementitious materials. The user specifies (Fig. 3.9) the ratio of up to three coarse aggregates to each other and up to two sands to each other (or imports this from the Automix or Basic Mix Design programs). Requirements of MSF, slump, temperature, and air % are specified. The amount of fly ash, silica fume or slag can be specified as either a fixed amount or a percentage. The generated table can be in steps of 1, 5, 10, 20, 25, 50 or 100 kg.

The table (Fig. 3.10) shows batch quantities and density and gives two estimates of compressive strength using the Feret formula (section 3.15) and equation (3.4).

The system permits actual concrete test data of cement content, water content, strength and density to be entered (or automatically obtained from the database on nominating the production mixes to use) and then has the facility to optimize the constants in the water prediction equation, and also the strength equations, to give the best match to the input data. Graphs are displayed of strength *versus* cement content, strength *versus* w/c ratio, density *versus* cement content, and water content *versus* cement content. On each of these graphs the entered data points appear in addition to the graphs from the optimized equations. Three thumbnails of

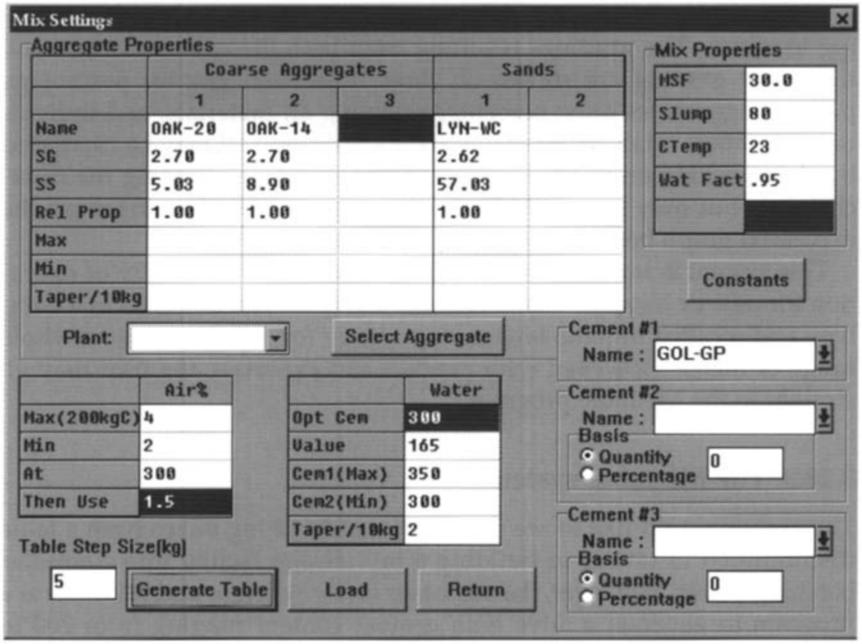


Fig. 3.9 Mix Table 1: input design instructions/data.

grading graphs are also provided. These expand on right-clicking and are automatically printed out with each table of mixes. Figure 3.9 shows the input screen, Fig. 3.10 a typical table of mixes, Fig. 3.11 the retrieved production data, and Fig. 3.12 the displayed screen of graphs.

The concept was originally relatively simple until some of the implications of actual use were considered. It was then realized that mix amendment needed to be done on the basis of early age strength predictions; that even so it had to be done on a very limited number of results; and that, to reduce the necessary testing frequency, results of dissimilar mixes (e.g. pump and structural) using different cemetitious combinations (e.g. with and without fly ash, slag and silica fume) had to be adjusted so as to appear on the same graphs. It was relatively easy for an expert to carry out the adjustment after downloading the data into a spreadsheet, but the challenge, as usual, was to automate the process for use by non-experts.

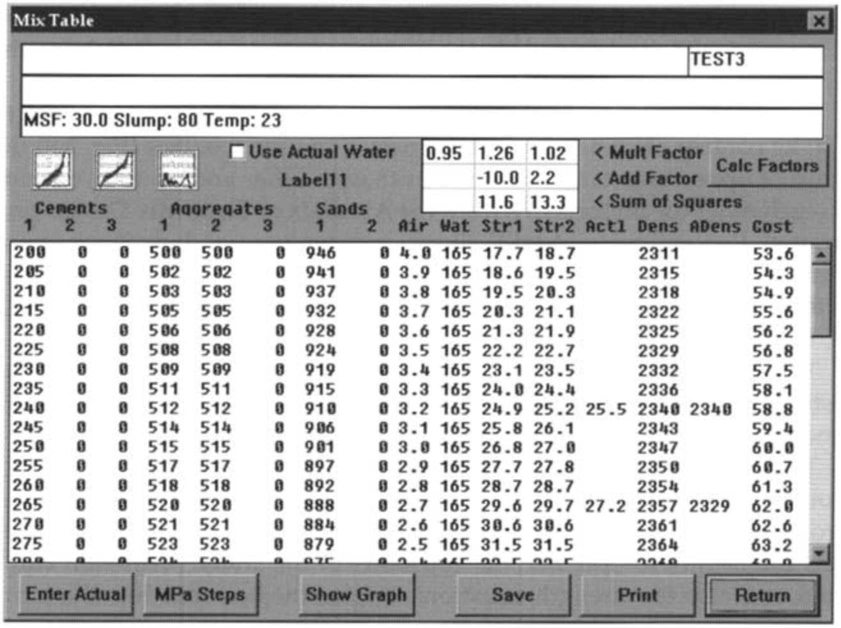
3.18.4
Cement Margins program

The concept of the Cement Margins program is to examine past results to see whether or not they are giving the desired target strength. It is designed to help the operator quickly notice areas where either a saving of cement can be made, or an increase of cement is required to reduce the risk of rejection.

It serves two purposes:

1. To fine tune cement contents for maximum economy.
2. To serve as an initial alert on problems requiring investigation.

The program separates QC Test data into groups with the same



Cements 1	2	3	Aggregates 1	2	3	Sands 1	2	Air	Wat	Str1	Str2	Actl	Dens	ADens	Cost
200	0	0	500	500	0	946	0	4.0	165	17.7	18.7		2311		53.6
205	0	0	502	502	0	941	0	3.9	165	18.6	19.5		2315		54.3
210	0	0	503	503	0	937	0	3.8	165	19.5	20.3		2318		54.9
215	0	0	505	505	0	932	0	3.7	165	20.3	21.1		2322		55.6
220	0	0	506	506	0	928	0	3.6	165	21.3	21.9		2325		56.2
225	0	0	508	508	0	924	0	3.5	165	22.2	22.7		2329		56.8
230	0	0	509	509	0	919	0	3.4	165	23.1	23.5		2332		57.5
235	0	0	511	511	0	915	0	3.3	165	24.0	24.4		2336		58.1
240	0	0	512	512	0	910	0	3.2	165	24.9	25.2	25.5	2340	2340	58.8
245	0	0	514	514	0	906	0	3.1	165	25.8	26.1		2343		59.4
250	0	0	515	515	0	901	0	3.0	165	26.8	27.0		2347		60.0
255	0	0	517	517	0	897	0	2.9	165	27.7	27.8		2350		60.7
260	0	0	518	518	0	892	0	2.8	165	28.7	28.7		2354		61.3
265	0	0	520	520	0	888	0	2.7	165	29.6	29.7	27.2	2357	2329	62.0
270	0	0	521	521	0	884	0	2.6	165	30.6	30.6		2361		62.6
275	0	0	523	523	0	879	0	2.5	165	31.5	31.5		2364		63.2

Fig. 3.10 Mix Table 2: resulting table of mixes.

- Month.
- Product Code (mix).
- Plant.
- Cement Group (a different group is automatically set up for every combination of cementitious materials, when processing batch data).
- Aggregate source group.
- Admixture, which may be included later.

The screen display and the basic printout are in order of the MPa deviation from target. This therefore highlights, at opposite ends of the list, groups posing a risk of failure (or requiring further investigation) and groups where an opportunity exists for saving cement. The group summaries clearly show the relative economy of alternative materials.

The quality of information produced from the computer analyses is dependent on the quality of data entered. The program will reliably indicate excessive and inadequate margins but it may require operator expertise to determine their significance. Groups may vary from target due to factors other than a currently incorrect choice of margins and future mixes should not be adjusted for factors which may not apply in future. Such factors may include slump or temperature variation and testing error. In an effort to overcome these potential sources of an inaccurate analysis result, the following checks have been built into the analysis.

1. The analysis separately examines the most recent results and a weighted mean over the past three months.
2. The analysis separately examines actual 28-day results and 28-day result predictions from 7-day results (to highlight any testing errors in addition to giving greater immediacy).

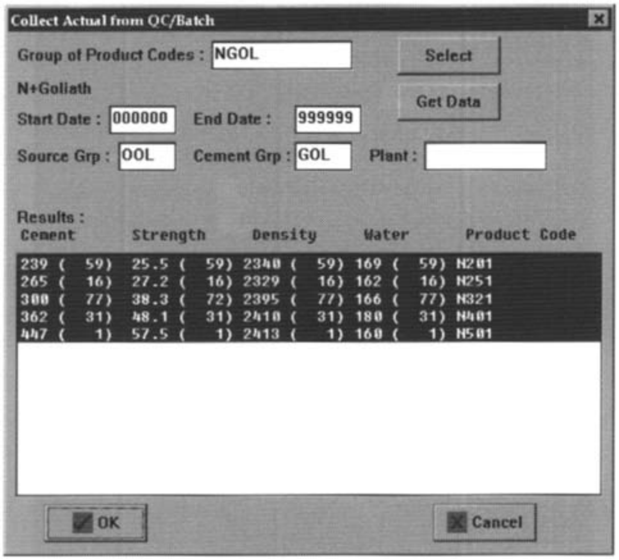


Fig. 3.11 Mix Table 3: retrieved production test data.

3. In addition to ‘actual minus target’ strengths, the program also displays ‘actual minus calculated’ strengths. This alerts the operator to deviations caused by abnormal slumps or temperatures (which should not be allowed to affect margins for material quality variation). The Conad program is capable of generating mix revisions for individual mixes to take into account such circumstances, if foreseen, but few clients currently make use of this facility.
4. The program also generates strength data adjusted for these deviations for use in determining desirable adjustments to future mixes. Such adjustments are obtained by graphing (actually fitting an equation to) the results to smooth out variability and reading revised figures from the graph (or generating them from the equation).

3.18.5
The Benchmark system

The Benchmark system is designed to compare the performance of a large number of mixes in production use over a wide area (perhaps internationally) by a major concrete producer. However, it could also be used as an absolute comparison standard by small producers.

The input mixes may represent different

- Aggregate sources (crushed, rounded, smooth, rough).
- Cements.
- Grades of concrete (strength levels).
- Types of concrete (workability requirements).
- Climatic conditions.
- Design philosophies (degree of sandiness, continuity of gradings).

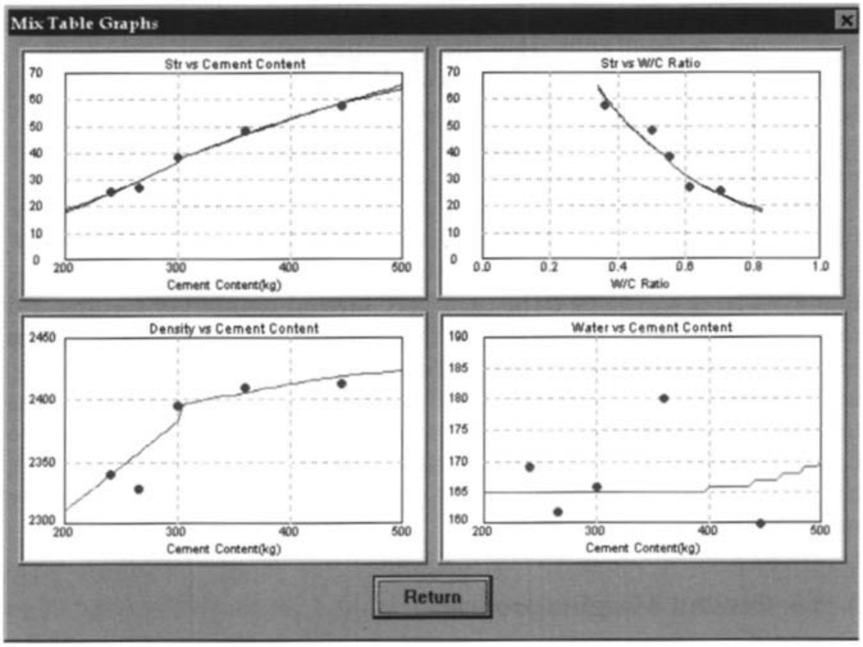


Fig. 3.12 Mix Table 4: system equations optimized to retrieved production data.

The concept is to employ an absolute standard provided by the MSF (mix suitability factor or degree of sandiness) concept, together with the water content and strength calculations forming part of the Conad system, to compare the performance of the input mixes.

The program generates eight sets of graphs (Fig. 3.15):

- Cement content *versus* MSF (with or without a 'shape correction factor').
- Cement content *versus* Strength (actual or calculated).
- Cement content *versus* Water content (actual or calculated).
- Cement content *versus* Cement effectiveness (kg/MPa).
- Cement content *versus* Strength ratio (actual/calculated).
- Cement content *versus* Water ratio (actual/calculated).
- Calculated water *versus* Actual water.
- Calculated strength *versus* Actual strength.

If a wide range of data is in fact available, users will not be at the mercy of the author's opinions of what is good, but will effectively be using their own data for the comparison.

- The graphs of MSF *versus* cement content will reveal any differences in design concepts or aggregate properties.
- The graphs of actual strength *versus* cement content will show relative cost efficiency
- The graphs of actual water content and strength *versus* calculated values will reveal whether cost efficiency variations are due to material characteristics, climatic conditions, or design philosophies.

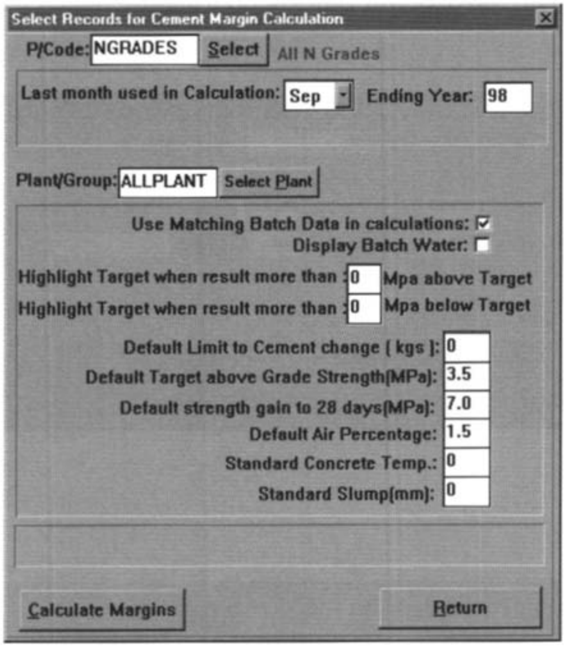


Fig. 3.13 Cement Margin record selection screen.

The program should enable users to highlight uneconomic material sources and mix design practices, enabling differentiation between these two very different causes of excess cost and allowing for regional climatic variation.

3.18.6
The Trial Mix program

While the author is generally opposed to the use of trial mixes, there are exceptions to this, and it is easy to write a computer program which will assist.

Such a program must have two basic functions:

1. The ability to scale quantities per cubic metre and adjust for moisture contents for any desired size of trial mix.
2. The ability to take the actual quantities used in the trial mix (which may have been accidentally or deliberately altered) and reconvert them into quantities per cubic metre.

It helps if the program can also predict the likely results of a mix from constituent details, in terms of water requirement, strength, and density. The operator may then be able to see the reason if the results are other than as anticipated. This can be of substantial assistance, if only because it provides a means of comparing mixes which are not quite as intended, or not otherwise comparable due to differing slump, air content or temperature.

Figure 3.16 shows the Mixtune trial mix screen. Initial mix data must be entered via the basic Mixtune program but any mix may be entered directly into the first column of basic Mixtune or read in from any of

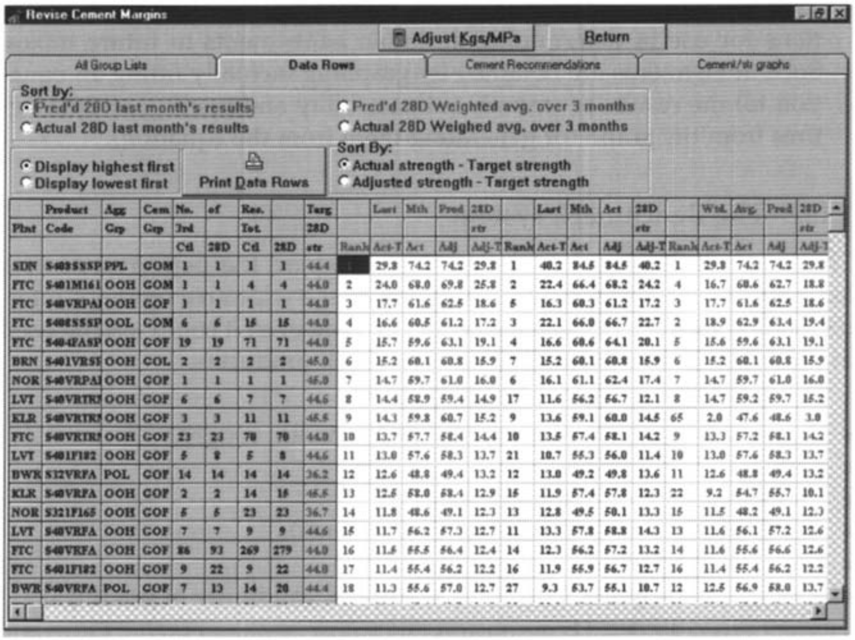


Fig. 3.14 Cement margins: full screen view of data rows.

the other Mixtune design programs. It is necessary that aggregate details have been previously entered via the Material Control screens (Figs 5.1 and 5.2) and that the intended mix properties in terms of slump, temperature, and air % are entered.

The next step is to enter the aggregate moisture percentages, then the size of trial batch required, and then key 'calculate'. This will cause the required batch quantities to be displayed in the third column and also, for convenience, they are duplicated in the fourth column but they may be overridden there as the trial mix proceeds, e.g. extra sand or water may be added. Similarly the mix properties may not be as intended and the actual values of these may be entered in the right-hand column of the properties table, on the right-hand side of the screen (Fig. 13.6).

On keying 'calculate' again, the actual trial mix quantities will be translated into quantities per cubic metre in the second column, alongside the originally intended quantities.

Along the bottom of the screen will be displayed the water calculations performed by the system. It can be seen how much difference in water content the system expects to result from an observed difference in slump, temperature etc.

If the fresh density of the concrete is measured and entered, the system will display the calculated air % to cause such a density, and also show the revised yield. The calculated quantities per cubic metre will also be revised in accordance with the measured density.

If air % is measured, the calculated density will be revised. Yield and quantities per cubic metre will be corrected providing fresh density has not been measured. Obviously only one of the revised yields from air % and fresh density can be accepted. Fresh density must be given precedence on the basis that an incorrect SG may have caused the difference.

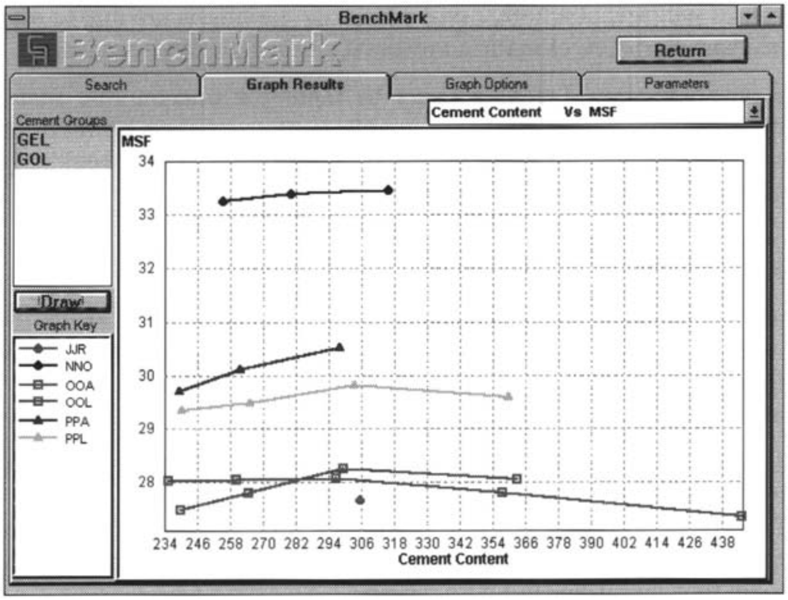


Fig. 3.15 Benchmark sample graph (cement content *versus* MSF).

3.18.7 Demonstrations programs

The CD is intended to enable readers to examine several basic Conad contentions using their own data (Fig. 3.18). Two programs are provided, **QC Demo** and **Mix Design Demo.**

QC Demo

The user is invited to enter data on up to 100 samples of concrete (all of the same grade of concrete). The entered data provided for are:

1. Date of cast (as ddmmyy or as you wish).
2. Slump (mm intended, in or cm may work on cusum but are not good on direct plot).
3. Temperature (°C or °F as desired, we use °C).
4. One 7-day and two 28-day test results, strength and density (MPa or N/mm^2 and kg/m^3 intended, psi and lb/ft^3 may work on cusum but are not good on direct plot. It is also satisfactory to use other than 7 days [e.g. 3 days] as the early result, providing all same age).

The columns provided but not accessible for direct entry are (in order of occurrence):

1. Strength gain from 7 to 28 days.
2. 7-day strength as a percentage of 28-day strength.
3. Predicted 28-day strength using (1) above.
4. Predicted 28-day strength using (2) above.
5. Prediction error using (3) above.

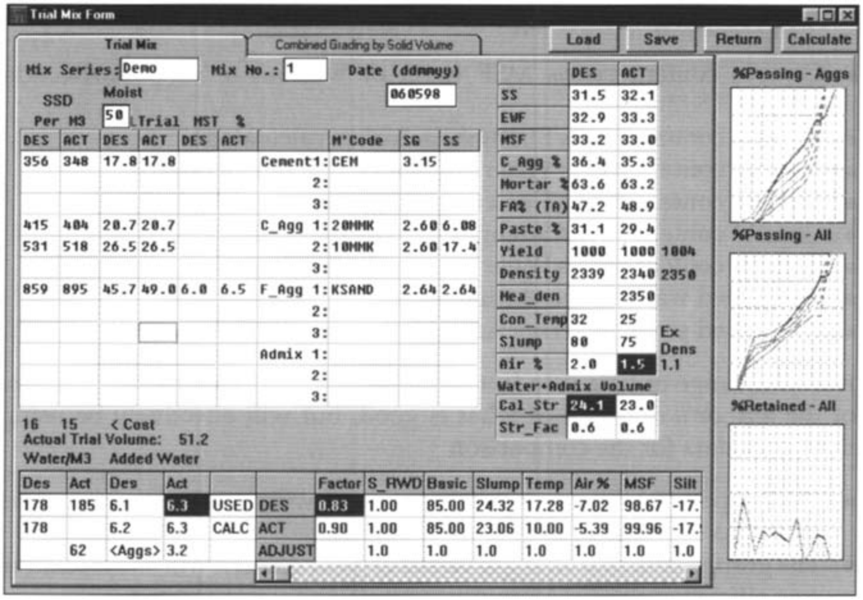


Fig. 3.16 Trial Mix main screen.

6. Prediction error using (4) above.
7. Pair difference between the two 28-day results.
8. Average 28-day strength.
9. Average density.

On clicking the 'calculate' button, the above columns, and the statistics rows at the top of the screen, will be completed.

The expectation is that the user will find that the error in (6) is greater than that in (5), thereby confirming the correctness of the Conad prediction technique.

The average pair difference (7) is a good indication of the testing quality. If it is less than 0.5 MPa (75 psi) it is unbelievable. If it is between 0.5 and 1.0 MPa (75 and 150 psi) testing is good. If it exceeds 1.5 MPa (225 psi) testing is poor and the demonstration may not be significant or convincing.

A special graph is provided of 7 to 28-day strength gain against 7-day strength. If it is correct to use a percentage gain assumption, the points would have to show a strongly increasing tendency to the **right** of the graph. The expectation is that, while there will be plenty of scatter, the trend will be horizontal, with possibly a slight increase at the **left** side.

A direct plot graph is provided. This may not be easy to read if different units are used to those intended.

The graphs include the actual 28-day results and two attempts to predict the 28-day strength from the 7-day results, one based on (3) above and one on (4) above. Both predictions will probably be reasonably accurate (with good testing) for most results but users should especially note which gives the best prediction for especially high and especially low 7-day results.

The cusum graphs should be self-scaling so that each line touches either the top or the bottom of the graph frame at some point. It is anticipated that the 7 and 28-day strength lines and the density line will follow each other closely, while one or both of the temperature and slump lines will tend to be a mirror image of these. The two 28-day strength prediction lines are expected to be on top of the 7-day strength

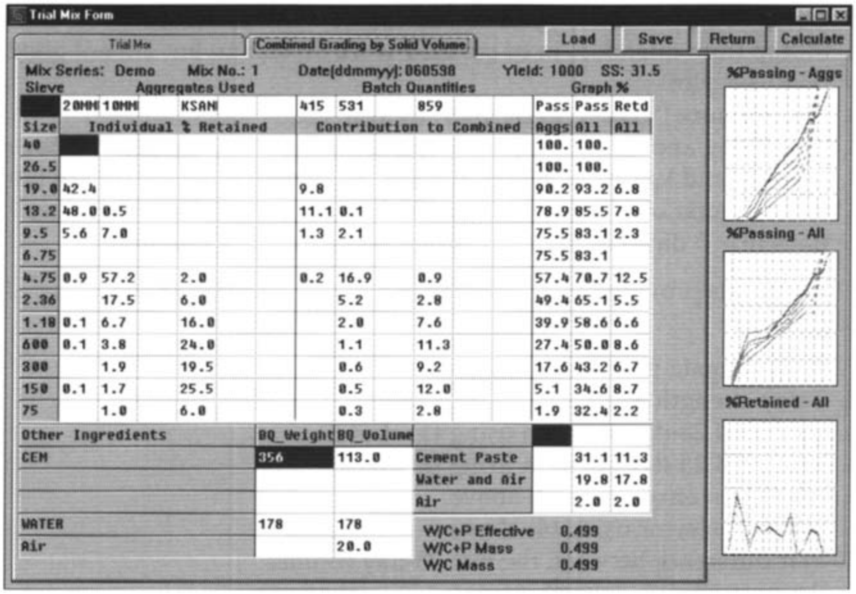


Fig. 3.17 Trial Mix: combined grading by solid volume.

line, since they are calculated from it. If there is very little variability in the data entered, the lines should still go fully to the top or bottom of the screen but may not show any noticeable trends (indicating that sampling or testing error is likely to be the main cause of such variation as remains.

Mix Demo

Two screens are provided (Figs 3.19 and 3.20). Figure 3.20 permits calculation of specific surface by entering percentage passing standard sieves. The 4.75 mm sieve is a 3/16 in sieve and sieve sizes are a series each being half the opening of the one above it.

The system can handle, store, retrieve and transfer to the other screen (Fig. 3.19) only a single grading for each of fine and coarse aggregate. If you have more than one of either, you must combine them for entry into this system. This can be done sieve size by sieve size as in the following example.

S and A has 90% passing a 2.36 mm sieve while sand B has only 40%. You wish to use twice as much A as B. Then the amount of combined sand passing the 2.36 mm sieve will be:

$$[(2 \times 90) + (1 \times 40)]/3 = (180 + 40)/3 = 73.3\%$$

Having keyed ‘use for coarse aggregate’ and ‘use for fine aggregate’ as each has been entered, these should already be in place in the third column of the main screen when you return to it. It is essential also to enter SG values for each material in the second column.

You can now enter any batch quantities you wish in the first column, but these must be in kg (2.2 lb=1 kg).

The fourth column on the screen will show the volume of each ingredient in litres and the fifth column shows the contribution it makes to MSF. Note that it may seem strange that 300 kg of cement makes zero contribution but this actually means that it just offsets the −7.5 constant in the equation used.

The bottom row shows totals. The total volume is the yield. If the yield is exactly 1000 litres then the total batch quantities will equal the density of the concrete.

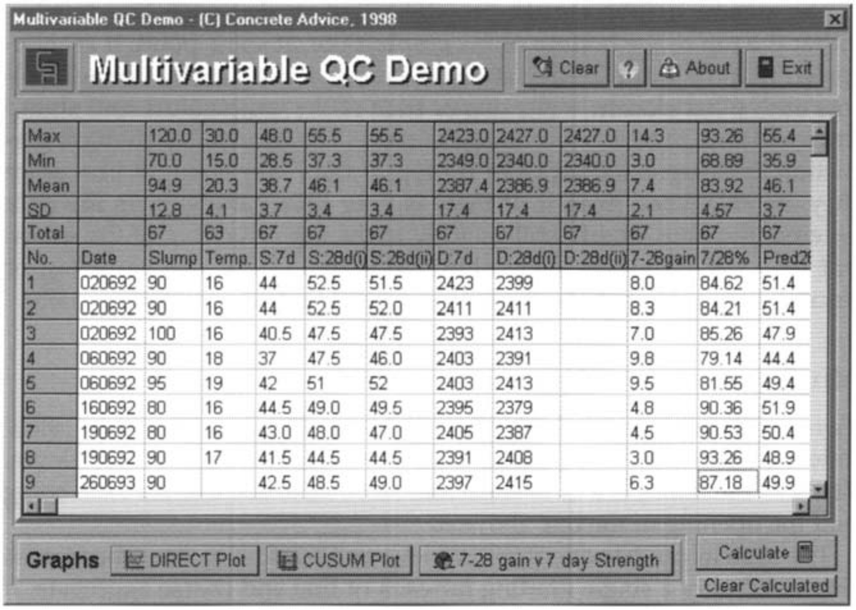


Fig. 3.18 Multivariate QC Demo main screen.

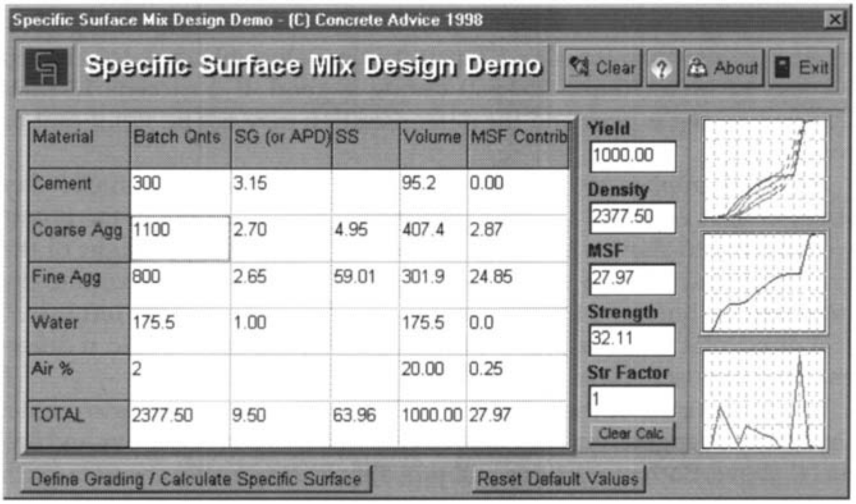


Fig. 3.19 Mix Demo main screen.

The strength prediction is accompanied by a strength factor so that if the strength prediction is incorrect for your materials, it can be amended.

This leaves the thumbnail graphs on the extreme right of the screen.

They are:

1. Percentage passing, aggregates only.
2. Percentage passing, all materials by volume.

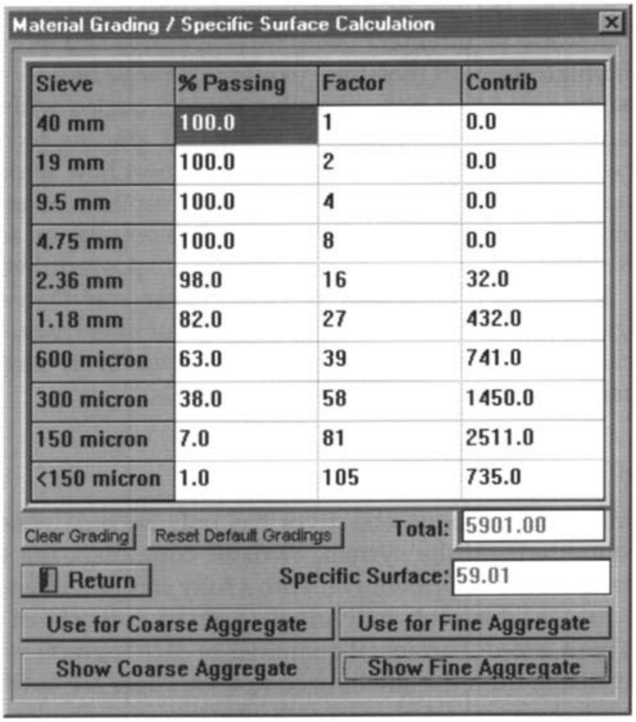


Fig. 3.20 Mix Demo: gradings screen.

3. Individual percentage retained, all materials by volume.

The graphs expand (individually) if the mouse is right clicked on them.

The purpose of this part of the program is for the reader to enter known mixes and see what MSF they have, so calibrating MSF for the reader. It should be found that mixes with the same fresh concrete characteristics but with widely differing cement contents or aggregate gradings have the same MSF value.

Note that the top graphs carry four background curves, being the old UK ‘Road Note 4’ type gradings so widely used as a criterion in the 1950s to 1970s.

By varying the sand content with the cement content at 300 kg the combined grading can be made to travel across these curves, further calibrating the user. It is interesting to note that gradings in popular use today are often finer than the finest of the Road Note 4 set. This is partly due to the presumed need for more workable concrete for faster placing, and partly because the use of water-reducing admixtures makes such basically high water requirement mixes less expensive and more capable of producing high strength than they would otherwise be.

It will also be noted that if the cement content is varied with the MSF kept constant, this will also cause the combined grading to cover the whole range of these background curves. This shows how wrong it is to maintain the same sand percentage regardless of cement content.

These demonstration programs have proved so popular on account of their simplicity that we have now produced a more powerful version we call Conad Mini which is for sale at much lower cost than the main

Conad program. This program includes batch plant interaction and early age features. It can also accept gradings in any series of sieves and has a multilanguage feature (see the CD for more details).

3.19
COMPUTERIZATION

The attainment of the above objectives has only become possible as a result of computerizing the system. Without computers the time and effort involved would be prohibitive. With a fully computerized system it is possible to consider each individual truck of concrete of which a sample has been tested. The actual quantities of each ingredient are automatically available from the batching computer. These are automatically combined with the latest set of aggregate gradings, etc., obtained prior to the truck in question. When combined with the slump and concrete temperature data obtained on the sample, and air percentage either directly measured or calculated from test specimen density, a calculated strength is obtained for comparison with the actual test strength. A very significant point is that strength can be calculated for every batch of concrete produced, not only those which have actually been tested. This aspect receives more consideration in Chapters 4 and 5.

Although the mix design systems can design mixes, their more important function is the rapid and accurate adjustment of mixes already in production use. A second very important function is to provide a comprehensive database of every test result (including concrete and aggregate testing), every mix adjustment made, and the actual batch quantities of every batch of concrete made. Such a record for a busy plant generating 50 000 trucks of concrete and 1000 concrete test samples would occupy two floppy disks for the year's data. Years later, it would take only a few minutes to establish fairly accurately the lowest strength batch of concrete of a particular grade delivered to a particular project on any given date, **even if that batch had not been sampled for testing.**

3.20
HIGH STRENGTH/HIGH PERFORMANCE CONCRETE

High strength concrete has come a long way since the publication of the first edition of this book. Strengths of up to 800 MPa (higher than ordinary steel!) have been registered on cement-based materials known as 'reactive powder concrete' but these are scarcely real concrete. Such strengths require heat and pressure during early curing and the mixes contain nothing coarser than fine sand. Still it is always useful to have results on such extreme cases so that one is interpolating rather than extrapolating when considering more normal concrete. In passing, it should be noted that such high strength material is not necessarily a purely impractical and uneconomic laboratory demonstration. Certainly it is expensive to produce, and cannot be used for *in situ* structures, but possibly economic production of some precast items is envisaged.

At present we may consider strengths of the order of 150 to 200 MPa to be possible with 'real concrete'. However the very term high strength concrete (HSC) is now becoming unfashionable and being replaced by high performance concrete (HPC). High durability, high wear resistance, high elastic modulus, high impact resistance, self compaction, anti-washout capability (suitability for underwater placement), high density, low density, low heat generation, low shrinkage, crack resistance, or a myriad other properties, may be the nature of the high performance.

So what constitutes high performance concrete? With so many alternative properties, common features are not so obvious. Perhaps the most essential constituent is a superplasticizing admixture. Another is at least one kind of pozzolanic material. Scientific formulation and a high degree of quality control in production are other likely features.

To some extent the user may find that the Conad system as it stands permits the design of quite high strength concrete but we are talking of up to 100 MPa rather than 150 or 200 MPa. There are certainly matters that should be pointed out for those without previous experience of strengths over say 60 MPa cylinder strength.

The first item of significance is that there is a distinct limit to how far it is possible to go on producing higher strengths by increasing cement contents. This has been obvious for a very long time but finally a new explanation has been produced. This is François de Larrard's concept of 'maximum paste thickness' (MPT) (de Larrard, 1999).

A second item is that density of packing rather than w/c ratio is the most important criterion.

A third item is the extreme importance of curing. For many years lip service has been paid to the importance of curing for normal concrete, without very much effect on practice. With HSC the situation is much more critical. The w/c ratio is frequently, even normally, lower than the 0.38 which is the lowest at which there is sufficient water for full hydration. This means that even if no moisture is allowed to escape, hydration, and therefore strength gain, will still stop when all the water has been consumed. High strength concrete usually contains a substantial percentage of pozzolanic material. This means that it will normally show a very substantial strength gain after 28 days when water cured. However, it has been suggested that this should not be taken into account since strength gain would rarely continue beyond 28 days in an actual structure.

The mode of failure of high strength concrete is different to that of normal concrete. There is a tendency to what might be described as 'columnar' fracture in which the test specimen splits vertically before crushing. Failure tends to be more accompanied by bond failure between the matrix and the coarse aggregate, or by failure of the coarse aggregate itself, than in normal strength concrete. This fairly obviously means that the parent rock of the coarse aggregate will be of importance, especially in terms of its bond characteristics. An interesting example of this occurred in Melbourne in the 1970s. There are two basalt aggregates available there, being known as 'older' and 'newer' basalt. Both aggregates had long been used to produce excellent concrete but there was a very substantial difference in their mechanical properties. By any test the older basalt gave far better results. It had higher crushing strength, higher elastic modulus, higher LA value (abrasion resistance), lower absorption, lower moisture movement, etc., but it did not work as well in high strength concrete. This was in the days when high strength concrete meant a strength of 50 MPa. The difference was attributed to a difference in bond characteristics. Ten years later, in 1984, the author conducted trial mixes to examine the feasibility of an 80 MPa grade of concrete. Several different coarse and fine aggregates were tried and, for completeness, one mix used older basalt coarse aggregate. Sure enough the older basalt 'failed' by producing only 85 MPa at 90 days whereas all other mixes produced between 100 and 110 MPa. These mixes used fly ash and superplasticizing admixture, neither of which had been available in the 1970s. However, they were prior to silica fume becoming available in Melbourne and it is understood that with this material there is no longer any disadvantage in using older basalt. Again this tends to confirm the bond hypothesis, since silica fume is known to be very effective in improving bond.

Bond improvement is not the only advantageous property of silica fume and this material is extremely valuable in the production of high strength concrete. Indeed it is virtually essential if **mean** strengths of more than about 90 MPa at 28 days are required. The actions involved appear to be partly chemical and partly physical. Having an extremely high surface area (100 times that of cement) it produces high pozzolanic reactivity. However, it is so fine that (properly dispersed) it fills the voids between cement particles, effectively displacing water and producing distinctly denser concrete. This brings back into memory Feret's strength proportionality to the ratio of cement to water plus voids rather than Abrams' w/c ratio (the modern version is the 'gel/space ratio'). These effects are far more involved than can be explained here but one interesting reference is to a comparison between the effects of silica fume and carbon black in concrete (Detwiler and Mehta, 1989). It appears that carbon black has a very similar size distribution to silica fume but absolutely no pozzolanic action. It produced around half the strength improvement of the fume and so it was concluded that the benefits of the fume are roughly 50/50 chemical and physical.

Returning to practical advice for those requiring it, rather than interesting (and possibly questionable) theories, the following guidance is offered:

1. If a cement content in excess of 500 kg/m^3 appears necessary (450 may be a better figure to work to) do something different, or at least use low heat cement if available.
2. The something different could include:

   (a) fly ash substitution—say 20–30%.
   (b) superplasticizer (i.e. high range water reducer).
   (c) silica fume.
   (d) smaller maximum size coarse aggregate.
   (e) aggregate with better bonding characteristics.
   (f) coarser sand.

3. Remember that when putting extra cement in will not work, taking water out always will, providing full compaction is still achieved.

Another important point to bear in mind when first using high strength concrete is that it is even more difficult to test than it is to make. The most essential point is proper curing. Of course this is easy to arrange in laboratory trial mixes and may be one reason why a satisfactory laboratory trial can be followed by disappointing results in the field (another is that high cement content concrete generates a great deal of heat in a truck mixer and thereby its water demand can substantially increase). High strength concrete probably does not contain enough water for full hydration in the first place (but unhydrated cement, if not surrounded by space left by evaporating water or locked in by flocculation, is a strong fine filler with excellent bond characteristics). It is therefore **very** important that it does not lose any of the original water. An interesting point is the difference in the effect of allowing low and high strength concrete to dry slightly in the first 24 hours. The low strength concrete will show a depressed 7-day strength but, if subsequently cured in water for 27 days, will usually show little effect on 28-day strength. This is because the concrete re-saturates and hydration resumes. The high strength concrete on the other hand is likely to show little or no effect on 7-day strength and a much reduced strength gain after 7 days. This is because the high strength concrete, once partly hydrated, becomes relatively impermeable and will not re-saturate even if kept in water for 27 days. The same effect no doubt occurs in actual structures and it is vitally important not to allow drying in the early stages. One sees instances, particularly in the tropics, of formwork being removed at less than 18 hours from massive columns of high strength concrete which, through its self-generation of heat, is too hot to touch. Such concrete is practically blowing out moisture as jets of steam. If only kept from losing moisture until it reaches ambient temperature, such concrete may by then be mature enough (having essentially steam cured itself) and impermeable enough, not to lose much further moisture. Also it probably already has developed most its design strength.

Aitcin (1998) has emphasized that there is an inadequate amount of mixing water for complete hydration and it is insufficient to use a curing compound or polythene sheet to seal in initial moisture. Additional water may be required to attain desired long term strength or to avoid autogenous shrinkage.

A very desirable and interesting recent development has been the use of steel pipe columns filled with high strength concrete. Pipes of the order of a metre or so in diameter can be erected four storeys or more and then pumped full of high strength concrete from the bottom up. Such concrete can be of 80 MPa or more in strength but must be superplasti cized to fully flowing condition (over 200 mm slump—the author recalls having threatened to reject concrete if the slump fell below 200 mm). An equally vital requirement is the absolute avoidance of bleeding. Any bleeding would cause settlement and create havoc at any reinforcement, shear keys or internal projections of any kind (steel beams may project into such columns). The bleeding inhibition is achieved by the use of silica fume which thus serves a dual purpose. It is not difficult to persuade the concrete supplier that silica fume is necessary in 80 MPa concrete but more explanation is needed if an even higher dosage is specified for some 30 MPa concrete in a lightly loaded but otherwise similar situation.

4
Philosophy and techniques of quality control

There are two aspects to controlling concrete quality. One of these is the avoidance of failures and the other the attainment of low variability. Obviously low variability will be of assistance in avoiding failures and vice-versa but it helps to consider the two separately. Equally obviously there will be no failures if there is an adequate margin between the average quality level and the specified minimum.

What is useful is to consider separately those factors acting continuously and those acting intermittently. It is even possible that some of the same factors can fit into both categories, e.g. sand grading is unlikely to be identical from truck to truck but there may be a more substantial change from time to time as extraction location or conditions change. It is a difference between variability about the same mean value and a change in mean value. If a change in mean value remains undetected it causes an apparent increase in basic variability.

The continuous basic variability can be thought of as a feature of the production process. It can only be improved by improving that process or the uniformity of the materials supplied to it. The early detection and reversal of occasional change is a feature of the control system. So the control system **measures** the basic variability and **detects** change points. It also **contributes** to the overall variability to the extent that it fails to detect and correct changes immediately.

Apparent overall variability is also increased by the error in testing or recording data. This also affects real overall variability in that, by partially obscuring change points, it slows their detection.

The analysis system adopted will similarly have an effect on overall variability through the speed at which it is able to react to change points. It may also have a substantial effect on basic variability to the extent that it is able to highlight the causes of that variability in such a way as to enable them to be reduced through maintenance or production system improvement.

The above is worth bearing in mind when comparing and contrasting quality control (QC) and quality assurance (QA). Insofar as they differ, QA is concerned with avoiding problems by pre-inspection of materials and certification of implementation of control and production procedures. It could be considered to be aimed more at eliminating change points rather than at their early detection. This could be counterproductive if it does not prove possible to eliminate change points and results in their slower detection. However this argument could be overpedantic. Quality assurance has also been described as documented QC, suggesting that the main difference is only one of record keeping.

4.1 HISTORICAL EVOLUTION

4.1.1 Changing attitudes

Realism is a quality frequently absent from a consideration of how to control concrete quality. It has tended to be a preserve of desk-bound administrators and statisticians unfamiliar with concrete. Having said this, it should also be said that one such statistician, Dr F.R.Himsworth (1954), shed more light on the subject than had been shed by generations of those within the industry.

For many years the industry and related professions have been relatively uninterested in quality control, confusing it with acceptance testing. There has been a tendency, sometimes justified in the past, to regard concrete producers as ignorant and dishonest persons with whom a firm hand needed to be taken. Today it is gradually becoming recognized that, at least in some parts of the world, most concrete suppliers are conscientiously trying to do the right thing and many are acquiring a high degree of technical competence. There is a growing recognition by both the producer and the purchaser that a single truck of defective concrete can result in a huge expenditure, possibly outweighing the entire profit margin on a whole contract. Nevertheless a competitive market often limits the use of large safety margins and, as structural designs call for higher and higher strengths, these may in any case be technically impractical.

While the respect of purchasers for the concrete supplier has tended to increase, the reverse is less often the case. As his own knowledge increases, the depth of ignorance about practical concrete technology by many of those supervising him becomes more apparent to the concrete producer. The technology of concrete QC is no longer, if it ever was, something which can be learned in the last half hour of a university course.

The time is therefore extremely propitious to take a detailed, realistic look at the control of concrete quality. Such a study must reconsider the basic philosophy of QC, the nature of concrete variability and the reliability of test data. It also needs to take into account the motivation of producers and the effect of specifications on the future development of the technology of production.

It is interesting to note that the science of quality control (of anything, not specifically concrete) was essentially pioneered by three Americans (Shewhart, Demming and Juran) but was only widely accepted in America after it had been used by Japan to decimate such American industries as car manufacture and electronics. The author was privileged to attend a QC course by Juran in 1957 at which Juran advised participants that he had just given a similar course in Japan and had never had a more enthusiastic reception anywhere. He confidently predicted that Japan, which at that time had an atrocious reputation for quality, would lead the world in its application within a few years.

Readers should note the existence of chapters on the testing of concrete and on statistical analysis. Those unfamiliar with statistics, or unconvinced that the testing process is a significant source of the apparent variability of concrete, should read these sections before proceeding in the current section. Those who are well experienced in these matters may nevertheless find food for thought in the author's individualistic views.

There is a strongly held view that a set of data must be allowed to accumulate without intervention until a reliable statistical assessment is possible and only then should an adjustment be made. The risk seen is that, by making more frequent changes, the apparent variability will be increased and the true situation never known. Also the changes may be unwarranted and counter-productive. The author was brought up on this theory and adhered to it for several decades. However, he has gradually come to the conclusion that, in many circumstances, it is not correct. The requirements for early intervention are the reasonable (as opposed to statistical) certainty that the situation being corrected is not a temporary, self-correcting aberration (i.e. that it is still continuing) and that the correction will not result in a larger discrepancy in the opposite direction. Reasonable certainty does not have to be based solely on the strength results, it can also take into account related variables such as density, slump and temperature. It will be seen that it is relatively easy to fulfil these requirements using the author's system, although it may still be difficult to do so using more traditional methods. Obviously the earlier a system provides an effective assessment, the less onerous the requirement to be sure the situation is still as assessed.

4.1.2
The objectives of quality control and quality assurance

In 1958 the author wrote a series of articles on 'Statistical Quality Control of Concrete and Concrete Products' (Day, 1958–9) which contained the following comment:

> 'The only rational objective for any but 100% testing is not to discover and reject faulty products but to ascertain the minimum quality level of the production. A moment's thought will show that if 10% of total production is tested, then for every faulty unit discovered and rejected, nine faulty ones will be accepted. This applies not only to final tests on products but also to each individual batch of concrete produced.
>
> If any reject units or concrete specimens whatever are discovered, a serious situation exists which cannot be met by the rejection of the tested defective units alone and should lead to extra testing on a scale that would normally dislocate the entire production system.
>
> A distinction should be drawn between unsatisfactory and unusable products. No useable product should be rejected and no unsatisfactory product should be paid for as first quality…the specified minimum strength may be 4000 psi but clearly 3950 psi would not constitute unusable concrete…if the absolute rejection limit be maintained at 4000 psi and a large concrete unit contains nine batches stronger than 4000 psi and one at 3950 psi, then it would have to be rejected. This is clearly undesirable.
>
> If 10% of the tests made are below strength, then probably all units made contain defective concrete although on average only one in ten units would show defective on test cylinder results. It would not be satisfactory to reject this tenth unit. If however the results were statistically analysed and it was shown that 10% of results were below specification (but not dangerously so) a cash penalty could be imposed and all units accepted. If one dangerously low result were obtained then probably nine previous units contain dangerously defective concrete and acceptance testing of all production should be carried out.
>
> This underlines the desirability of a zone of useable though unsatisfactory concrete since, in its absence, we have either to regard 3950 psi concrete as dangerously weak, or to allow a manufacturer to produce poor concrete with impunity on occasions.'

The author is still of the opinion that a distinction should be drawn between **structurally** defective concrete and **contractually** defective concrete defined as follows.

Structurally defective concrete is that which is unable to serve its intended purpose and must be removed from the structure or supplemented in some way. It is absolutely imperative that no such concrete whatever be produced (it is not practicable to allow some to be produced and then attempt to ensure its exclusion from the structure).

Contractually defective concrete is that which, while capable of serving its intended purpose, is not quite of the specified quality. A small proportion of such concrete may be incorporated in the structure with little detriment.

There is usually a substantial margin between the two and the author's experience is that if no contractually defective concrete is accepted without some penalty or substantial expense and inconvenience to the contractor, no structurally defective concrete is produced. However, if contractually defective concrete is supplied with impunity, structurally defective concrete is likely to follow.

Quality control has nothing to do with setting a high or increased level of quality and very little to do with acceptance testing. The required minimum quality level should be set by the specification, and quality control or quality assurance are concerned with so regulating production that the required quality level is attained at the minimum cost.

Before commencing production, an assessment must be made of what average quality is needed in order to satisfy the minimum requirement and how that average quality can be provided. The control function then consists of monitoring the situation so as to detect, at the earliest possible moment, when either the average quality or the variability of that quality changes or becomes likely to change. The system should then go on to rectify, or take advantage of, the detected change (depending on whether it was for the worse or the better).

The control system should monitor not only the quality of the resulting product but also the input materials, the production processes, the ambient conditions and the accuracy of the testing process.

The above was all being done as quality control by the author decades before the term quality assurance came into vogue. To some extent 'a rose by any other name would smell as sweet' but in so far as there is a difference between QC and QA, it is that QA is necessarily preplanned and documented as to both procedures and their execution. Quality assurance provides an assurance, in the form of certified records, that the established QC procedures have been carried out in full and it is intended that the system should be sufficiently comprehensive to **necessarily** ensure the acceptable quality of the output. While QC **may** also include the same procedures, this is not necessarily the case.

4.2 THE NATURE OF CONCRETE VARIABILITY

4.2.1 The distribution pattern

Most investigators agree that strength is at least approximately a 'normally distributed variable'. This means that it can be completely described by a mean strength and a standard deviation (SD), i.e. that the percentage of results lying above or below any particular value can be calculated from the mean strength, the standard deviation and a table of values from a statistical textbook.

This is in line with the author's experience except that, almost invariably, the percentage lying 1.65 times the standard deviation below the mean is 2 to 3% rather than the 5% indicated by statistical tables. Why this should *be* so is not of any importance (perhaps through various kinds of control action such as rejecting overslump concrete or badly compacted test specimens) but it is fortunate that it is, because it reduces the amount of unnecessary concern occasioned by the inevitable lower end of the distribution. Interestingly a UK concrete technologist states that his experience is opposite to this and that he typically obtains **more** than 5% of results below the 1.65 SD level. If correct, this may be a result of the slow reaction of UK cusum to downturns (but see Fig. 4.1 and the last paragraph of section 4.2.2).

When the spread of results lying above and below the mean value (strictly speaking the 'mode' or most frequently occurring value) are unequal, the distribution is said to be 'skew'. This is not a frequent occurrence and if it is encountered a reason should be sought. If the spread of results is wider on the low side of the mean, some factor is probably truncating the spread of results to the high side. The cause may be genuine, such as a coarse aggregate of low crushing strength or having a smooth, non-absorbent surface leading to bond failure. On the other hand it may be non-genuine such as a defective testing machine or an operator who is afraid of explosive failures. Similarly when results are truncated on the low side (to a greater extent than the 2 to 3% replacing the theoretical 5% mentioned above) the cause should be investigated to ensure that malpractices or extraneous factors are not leading to an incorrect assessment of the true situation. Another type of abnormal distribution sometimes seen is a double-peaked distribution. This is the result of two separate distributions being combined. It may be that the concrete comes from two ready mix plants operating to different mean strengths. It is possible that there is a difference between morning and afternoon shifts (e.g. temperature, slump preference). It is also possible that different testing officers or testing machines give different results. See the chapters on testing and statistics for more detail.

When a large number (say, 100 or more) results obtained over a period of several weeks are analysed, the assumption is that the mean strength remains unchanged and that variability about it is completely random. If the same number of test specimens were obtained from a single day's concreting, or even more so from a single truck of concrete, it would not be surprising if the variability (standard deviation) were much less. This is because not all the factors causing variability over a period are operative over the single day. In the case of specimens made from the same truck, the variability could be described as the 'testing error' since all the concrete is essentially the same if the specimens have been made from properly remixed multiple samples of concrete spaced during the discharge of the truck.

4.2.2
Factors causing variability

Graham and Martin (1946) were the first to publish an attempt to locate and quantify sources of variability on an actual project. The project was Heathrow Airport, UK, on which 0.5 million cubic yards of concrete were produced and controlled in an exemplary manner.

Full details were given in the paper, with the results in suitable form for others to use. Careful tests were done to establish the sources of variability. Results were expressed as 'possible range in cube strength due to'. Ranges were incorrectly treated as additive to give extreme cases:

	Possible range	
Quality of cement	2100 psi	48.2%
Water/cement ratio	800 psi	18.4%
Sampling & making cubes	500 psi	11.5%
Testing cubes	600 psi	13.8%
Mixing time	200 psi	4.6%
Varying SG of aggregate	150 psi	3.5%
Total variability		100%

They concluded that cement was responsible for 48.2% of the variation of strength that occurred at Heathrow. (And therefore that this was not under site engineers' control).

Himsworth (1954) reanalysed Graham and Martin's results and established that cement was actually only responsible for 10% of the observed variation (section 10.5), water content being a distinctly more important variable. Even more importantly he established theoretically the superiority of workability control over w/c ratio control and showed that uniformity was in the hands of site personnel (the concrete was site mixed, not supplied from a ready mix plant).

Himsworth's paper included a theoretical examination of the factors involved in variable concrete strength and their likely magnitude under different degrees of control. A very interesting comparison was given between the control of moisture content by measurement of aggregate moisture and control by the resulting workability of the concrete. He concluded that even 'good' control by workability gave lower strength variability than 'excellent' control by measured moisture content. 'Poor' control by workability was equal to 'good' under calculated water. This was a full vindication on theoretical grounds of the still frequent practice of permitting ready mix truck drivers to add water 'by eye'.

The conclusion is a little optimistic because it neglects effects of grading, particle shape and temperature on workability. However, control by calculated water, even if giving equally good control of strength, would produce greater range of workability, so the concrete would be harder to place uniformly, less uniform in appearance, etc. The conclusions are still valid today. It is now possible to obtain better control of measured moisture content and this is no doubt the way of the future. However, only the very best equipment, used in the most careful manner, will give better results than control by eye estimation of workability. What is needed is actually a dual control in the highest technology plants. Aggregate moisture should certainly be measured and input water adjusted, but there should be feedback from the job or from a 'slump stand' in the producer's yard at which water is adjusted. If the total water content differs substantially from that calculated, then firstly cement content should be adjusted in subsequent loads, and secondly aggregate and other test data should be examined to establish the cause.

Another significant reference from the past was Erntroy (1960). He analysed results of 100 000 works test cubes produced on the full range of equipment from semi-automatic major batch plants to site mixers charged by shovelled aggregates and bagged cement. The interesting conclusion was that, whilst better equipment on average gave better results, good control was possible with very poor equipment—**the will to control is what really matters**. This is another conclusion still relevant today.

It is helpful also to consider the types of variation which may be encountered:

1. Random variation with no assignable cause. As control improves, the extent of such variation diminishes and an assignable cause is anticipated for any substantial variation.
2. Isolated or non-sustained changes having an assignable cause, e.g. an isolated high slump producing a reduced strength.
3. Sustained changes in mean strength.
4. Changes due to testing procedures (i.e. false changes) which again can be either sustained or isolated.

Strictly speaking, statistics only applies to random variations but what matters is not whether or not the statistics are valid but whether the techniques used enable improved control of concrete. The author's experience is that most sets of results over a period can be broken down into sub-periods of consistent mean strength and of lower variability than the overall set. The variability in the sub-period is the basic random variability caused by such factors as batching inaccuracy (including water) and testing inaccuracy. The overall variability is the combination of this basic variability with the variation in mean strength between the sub-periods. As explained earlier, the latter variations almost certainly do have an assignable cause, whether or not the control system is good enough to detect it. The points between sub-periods, at which

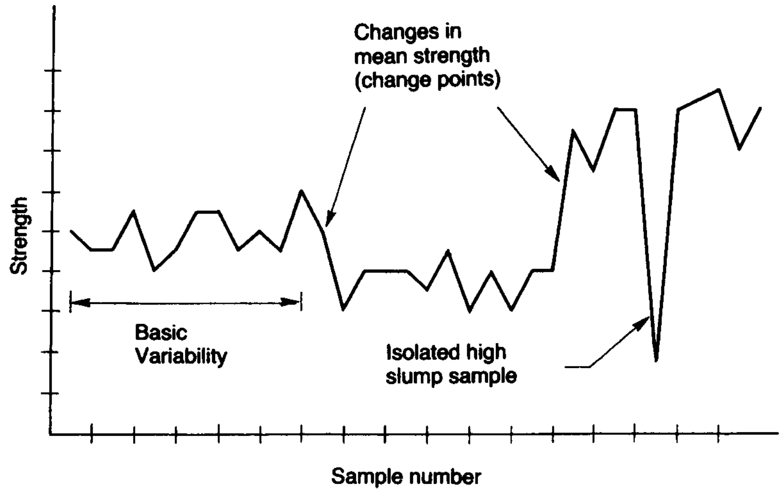


Fig. 4.1 Change points and basic variability.

mean strength shows a sudden or 'step' change, are known as **'change points'** (Fig. 4.1). The typical extent of a change is of the order of 2 to 5 MPa or 300 to 700 psi (which probably only means that changes of much less than 2 MPa are not generally detectable) and it will be seen that their early detection is the basic objective of a control system.

4.2.3
The significance of control action requirements

The basic variability in the period between change points is usually a matter of batching accuracy, especially water batching or slump control. Thus it is essentially a property of the production process. The extent of additional variability added by the changes themselves is more a property of the control system. In the first place it may be possible to detect and allow in advance for changes in the properties of input materials and the effect of temperature variations. To do this requires both that these changes are detected and that their quantitative effect on concrete strength be known.

In the second place, even if the cause of a mean strength change is unknown, its occurrence can be detected and compensated for by a change in cement content. Time is of the essence in making such adjustments. This is firstly because the longer the mean strength remains away from its desired value, the greater will be the effect on overall (longer term) variability. However, there is another aspect to the urgency of making the adjustment, which is often overlooked. If adjustments were made on the basis of actual 28-day strength, this would obviously mean that the adjustment could only be made something more than 28 days after the need for it arose. It is quite possible that a further change could occur during the period between the occurrence of the first change and its detection. It is equally likely that a subsequent change could be in either direction. If the second change is in the opposite direction to the first, then the adjustment being made for the first change could reinforce the second change. In this way it is possible that delayed control action could accentuate rather than reduce variability.

It is useful at this stage to set down two basic requirements for a control system derived from the foregoing:

1. The system must react as quickly as possible to discrete changes in mean strength.
2. The system should as far as possible detect the cause of the change. If this can be done quantitatively, it will be valuable in confirming whether the detected factor is the sole cause.

4.3 QUALITY CONTROL TECHNIQUES

4.3.1 Who should control?

Until recently, a contractor produced concrete in accordance with a specification (which often specified the mix proportions) and a supervisor representing the eventual owner arranged for testing and demanded or negotiated changes (depending on the nature of the contract) if necessary. The vestiges of this approach can still be seen in SE Asia where a batch plant can still produce well over 100 000 m^3 of concrete per annum in a super modern plant and have virtually no control system. Test cubes are certainly cast, and may even be cast by personnel employed by the concrete producer, but they are tested by the client and the results may not even be available to the producer except in the case of failure.

It is only recently (under their new standard AS3600–1989) that Australian consulting engineers have begun to recognize that control should be carried out by the producer. However, for many years most major Australian producers have operated their own testing and control systems regardless of other testing by their clients.

It is instructive to consider the evolution of the current situation in Australia. Strength specification was introduced in 1958 and was based on testing by independent NATA (National Association of Testing Authorities) registered laboratories. In 1973 a new code (AS1480–1973) attempted to hand over control to individual producers using their own registered laboratories. However, an alternative was permitted of continuing independent testing and control. The producer’s control alternative not only entirely handed control to each individual producer (not to a joint scheme as in the UK) but it did not make reasonable provision for notifying concrete users of the situation or require that early age testing be used. In effect it would have been possible for a supplier to produce defective concrete for almost two months (i.e. until a month’s defective 28-day results were in hand) and then only required that the mixes be amended to restore the minimum strength to its specified value. They were not even required to advise purchasers of what had happened, except for the ones who actually received concrete on which tests giving results below the specified strength were obtained. Showing good sense, the consulting engineers *en masse* totally ignored this alternative.

In 1989 a new code (AS3600–1989) made a more definite attempt to introduce producer’s control. This code **required** that producers operate a quality control system **whether or not independent testing was also in use**. It also contained reasonable provision for reporting and required an early age prediction system to be in use. Consulting engineers have been slow to accept even this code, tending to still require control by the previous code. The new code does contain optional provisions for independent testing in addition to the producer’s control but the provision is such that the independent results would have to be very low indeed before they outweighed the producer’s own assessment.

In 1991 the author devised a compromise solution which appears to be working well. The problem is seen as being that there are too few of the independent results to provide a reliable independent assessment of concrete quality and it would be too expensive to increase the testing frequency. The author’s solution is to **use the independent testing to assess the concrete supplier’s testing rather than his concrete**. There has always been a problem with concrete producers claiming that independent samples were not correctly taken or that the specimens were not properly cast or cured. This has been overcome, and the cost of the independent testing reduced, by requiring the concrete producer's laboratory personnel to cast a double set of test specimens at a specified frequency. This may vary from once every 3 or 4 samples to once per day or even once per week. Even a very low frequency will expose the existence of any problem, and the duplication frequency can be increased should a problem be encountered. The second set of specimens is required to be delivered by the producer to an independent laboratory for curing and testing. In addition the producer is required to fax his own test results on the day of test to an independent consultant for analysis.

It will be apparent that this system provides a good knowledge not only of the concrete, but also of the quality of work of both the producer's laboratory and the independent laboratory. It should be remembered that Australia has had its NATA laboratory assessment scheme since the 1940s (far longer than either the UK or the USA) and that it has such a good reputation that it is currently being used as a role model by many other countries in setting up their own systems. Details are given in Chapter 11, but it should be noted that the results of these comparisons certainly justify the duplication procedure. It should also be made very clear that the system has not revealed any hint of dishonesty, only evidence that it is extremely difficult to avoid occasional unmerited low test results. Several times the independent results have been used to show a producer that his own laboratory is producing lower results than merited by the concrete. However, more often the opposite is the case. This is not to suggest that the supplier's results are often biased but arises because, when results differ, the higher result is more often the nearer to the true value. This being so, it clearly does not pay the concrete supplier to skimp on the quality of his testing, the costs of which can be subsidized by his savings in concrete cost. However, **the independent laboratory can only operate profitably at a cost level which the market will bear and at a quality level which the market is able to distinguish**. Where supervising engineers believe that **any** registered laboratory **always** produces an accurate result, and specify independent testing but allow the contractor to choose the cheapest laboratory, a pressure to reduce standards to the lowest able to obtain registration is created.

4.3.2 Quality assurance

If it is not obvious from the above that control should be by the producer (even though some monitoring by others may be desirable) the question is settled by the worldwide trend to Quality Assurance in concrete as in many other fields. Quality assurance requires monitoring of all incoming materials and all production processes as well as ultimate results. This is really only financially practicable if done by the producer himself.

The International Standard ISO 8402 defines a 'Quality System' as '**The organizational structure, responsibilities, procedures, activities, capabilities and resources that together aim to ensure that products, processes or services will satisfy stated or implied needs'**. Clearly this is a much more comprehensive matter than techniques of testing or mathematical analysis. It may be taken as both advice and a warning. The advice is that such a formal and comprehensive preplanned structure has been found to be necessary to achieve a full assurance of quality. The warning is that care is needed to avoid being submerged in form-filling and administration at great expense, and possibly to the exclusion of effective quality control.

It is also necessary to be careful to avoid the 'new player's' assumption that quality assurance and quality control are necessarily different, with the latter being old fashioned and superseded. It may be helpful to think of quality assurance as simply '**documented** quality control'. As an example, consider the specification and control of a sand. It may seem like the correct quality assurance approach to specify grading limits and reject sands not complying. This may be contrasted with the author's approach of saying that almost any grading is useable, providing that the mix is adjusted. The particular technique of 'feedback quality control' may seem the antithesis of quality assurance, since it reacts to the effect of a characteristic of an input material. However, there is nothing wrong, or contrary to the principles of quality assurance, in writing in the quality manual that the mix shall be adjusted if the sand grading changes, or that water content shall be adjusted if slump is found to vary. What makes these actions quality assurance is that they **are** written down, and probably that a nominated individual has to make the change and document it.

4.3.3
Pareto's principle

Vilfredo Pareto was an Italian economist (1848–1923) engaged in travelling from town to town in an attempt to identify the country's sources of wealth. He came to realize that the four or five wealthiest men in a town almost invariably controlled over half its wealth. Therefore, his survey could most efficiently be conducted by first seeking out the right men and then asking his questions, rather than attempting a random survey of a few per cent of the population.

This principle is of great value in QC of all kinds, certainly including concrete QC, i.e. while there may be 100 or more factors causing variability in concrete strength, 70 or 80% of the total variability is often caused by only 2 or 3 of the 100 possible causes. Often only one single factor will cause more than half of the total variability. It will not be the same principal factor in all cases, nor even the same 'short list' of 2 or 3, but the following list is likely to include the major factors in most cases:

1. Slump (misjudgment or deliberate variation).
2. Temperature.
3. Air content.
4. Fine aggregate silt content.
5. Fine aggregate organic impurity.
6. Fine aggregate grading.
7. Coarse aggregate dust content.
8. Coarse aggregate bonding characteristics.
9. Cement quality.
10. Admixture quality or dosage.
11. Fly ash quality (especially carbon content).
12. Time delays.
13. Coarse aggregate strength.
14. Fine aggregate grain quality.
15. Sampling and testing procedures (viz, segregation, compaction, curing, capping, centring in testing machine, lubrication of spherical seating, planeness of platens, stiffness of machine frame, alignment of ram and spherical seating, rate of loading, operator fear of explosive failure or desire to maintain specimen in one piece).

The mechanism of the effect on strength is via increased water requirement in many cases, specifically in items 1, 2, 4, 6, 7 and 12.

Finding the principal causes of variability

It may be fairly obvious in some cases which of these causes is likely to predominate, but often this is not the case. Rather than make a guess, or spread control either too thinly or too expensively over too many factors, it is better to follow the advice of the master of QC, J.M.Juran (1951) and 'ask the process'. This can be done in two distinct stages:

1. Compare actual and predicted strength at the trial mix stage and if there is any discrepancy, track it down. This may provide a firm lead on what is most likely to affect strength on the particular project.
2. Monitor strength and a selected number of 'related variables' using cusum analysis.

The selected variables will usually include slump, air content and concrete temperature. If a reasonably reliable water content is available from any source, this is certainly very important. The strength results will be particularly examined for pair differences and 7 to 28-day gain as a kind of internal consistency test. It is important to realize that low strengths do not 'just happen'; they are usually caused by either high water content, low cement content, incomplete compaction, defective curing and testing, or reduced cement quality. The art or science of QC is to establish which of these is the cause by a logical examination of the pattern of results, e.g. difference in cement quality from one delivery to another is not a reasonable explanation for isolated low results or for a period of low strength extending for a shorter period than that between the two deliveries. High water content will not explain low 28-day results if 7-day results from the same sample were normal. Certainly the possibility that the concrete is normal and the testing defective should be adequately considered as it is frequently encountered.

4.3.4 Related variables

Strength results alone are certainly inadequate for the operation of a control system. We cannot be totally **unconcerned** with whether particular isolated results or isolated groups of results satisfy specification requirements. Nevertheless an examination of the **pattern** of results and their correlation or otherwise with 'related variables' (e.g. slump, density, etc.) is far more rewarding. The primary aim should be firstly to establish the overall situation as exactly as possible, and only then to consider the significance of particular results.

Monitoring day to day performance variations

Experience has shown that it is not enough to set up an excellent laboratory and thoroughly train suitable staff, any more than it is enough to set up a good ready mix plant and supply it with reputable materials. In both cases it is necessary to monitor the actual performance. In the case of testing the criterion is the repeatability of the test (although the relevance of the result may also be in question, especially in workability testing but also for example in early age strength testing). The best measure of repeatability is the average pair difference when tests, e.g. cylinder compression tests, are carried out in pairs. Another useful indicator is the average gain from an earlier to a later age, this can be varied by other factors, such as cement composition and curing conditions, but such variation often arises from testing problems. For example when a strength drop is experienced on both 7 and 28-day results from the same **testing** date rather than the same **casting** date, the testing process would be highly suspect.

4.3.5 Examining correlation

The classic method of examining correlation is to use a computer statistics package to provide a regression analysis (slope and intersect values, and a coefficient of correlation in a linear equation between two possibly related variables). This approach ought to work but disappointing results from it have been reported by Shilstone (1987) (Chapter 5). The author has found that better results are obtained by cusum graphing (section 4.4.3).

4.4 CONTROL CHARTS

4.4.1 General

Plotting results on a graph provides an appreciation of a situation far more rapidly than a table of results. This is a little less true now that numerical data can be analysed by a computer which can be programmed to detect and advise on any problems. However, the computer can also produce the graph automatically. Even if not necessarily more foolproof, a graph, by providing an overall picture, does give a greater peace of mind that the computer has functioned as intended.

British Standard BS5700:1984 *('Guide to Process Control Using Quality Control Chart Methods and Cusum Techniques')* contains the comments:

> 'It has to be recognized, however, that in many cases the major impact and benefits of control charting do not arise only from the statistical interpretation of the plotted points, but more because someone is seen to be taking an interest in the process or product, and is extracting information that is being recorded and used…it is clear that the motivational effects of simply instituting any form of sampling and recording close to the process should not be underestimated'.
>
> 'It should also be recognized that the title "control chart" is itself a misnomer to the extent that the chart is an historical record and does not of itself "control" the process. The essential action of a chart is to sound an alarm when attention to the process is required, and to give as much indication as possible of the nature of the change that has occurred so that remedial action can be taken to restore the process to its acceptable state'.
>
> 'In control charting the assumption is made that there is a "natural (inherent) variability" that is typical of the process when it is "in control". This inherent variability is called the process capability. The process capability is usually taken to be an irreducible level that is simply typical of the process, and is said to be due to random effects or unassignable causes of variability. This term is to some extent misleading in that in some circumstances it would no doubt be possible to carry out investigations to determine some of these causes. A more realistic view is that these causes are unassignable not because it is impossible to assign them, but because it is uneconomic (because the process capability is acceptable) or pointless (because even if the cause of the variability could be identified, it cannot always be removed)'.
>
> The purpose of a control chart is to detect any departure of the process from the "in control" state; any such movement is said to be due to assignable causes of variation. The decision making aspect of control charts is centred on the problem of deciding whether a sample observation is due only to the

expected, inherent unassignable causes of variation (in which case no action is required) or whether it is due to the effect of some additional, assignable variation (in which case corrective action is usually required).'

4.4.2 Shewhart charts

There are two main types of Shewhart (1931) control charts. These are charts for **variables** and charts for **attributes.** The latter type, which is concerned with counting the **number of samples** which are defective, or have some other particular **attribute,** is not used in concrete quality control. Charts for **variables** are concerned with **variations in measured characteristics.** This is the kind of chart which is useful in concrete quality control.

The concept first proposed by Shewhart in 1924 (Shewhart, 1931) was to plot a chart (Fig. 4.2) with time or sample number as a horizontal axis and the value of a measured parameter (e.g. strength) as the vertical axis. Horizontal limit lines were drawn on the chart. It is important to understand that these limits were not specification limits. Their function was not to indicate whether the result plotted was **acceptable,** but to indicate whether it was **unusual.** The intention was not to decide whether to accept or reject the product represented by the result, but to detect whether there has been any **change in the process** producing the product. This concept has proved very difficult to promote but is still the basis needed to achieve good quality control.

The limits were calculated from a statistical analysis of previous results. Statistical tables provide factors by which the standard deviation is to be multiplied to calculate the limit outside which a selected proportion of the individual results can be expected to lie. A factor of 3.09 (effectively 3.00) times the standard deviation (σ) gives the 1 in 1000 limit, i.e. theoretically one result in every 1000 should be expected to differ from the mean of all results by more than three times the standard deviation of those results. Putting this another way, if a result outside the 3σ limit occurs, there is only one chance in 1000 that the mean strength and standard deviation of current production remains unchanged. It may be of interest to show the

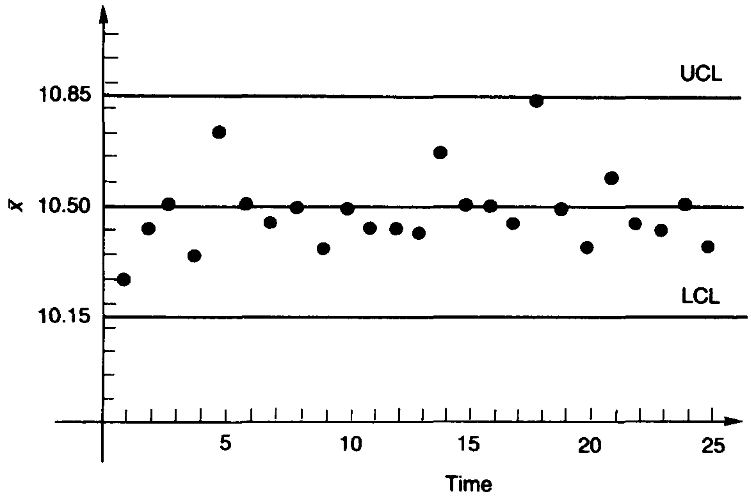


Fig. 4.2 Shewhart control chart.

(+ and −) 3σ limits on a control chart as an indica tion that **a change definitely has occurred.** However it would take a very large number of results (on average almost 500) before a small change in mean strength would cause a result to infringe such a limit. Closer limits are therefore selected to give a faster, if less certain, indication of a change, e.g. if the lines are drawn a distance 1.65×the standard deviation above and below the mean line, there will be one chance in 20 of a result lying outside each limit without a change having occurred. Such a result can be taken as a **warning** that a change **may** have occurred.

While charts can be used for individual results, detection of change is more rapid if the average of groups of three or four results are plotted. The ACI Committee 214 (ACI, 1977) uses groups of three results but some years ago Chung (1978) showed that better control would be obtained by using groups of four. Chung's paper earned him the Wason medal, but it did not result in adoption of the change by ACI 214.

It is important to be very clear on the objective of any QC operation. In using groups of three or four results, change is more efficiently detected but the explanation for individual variations may be prejudiced or clouded. For example if a high slump leads to a low strength, this will be at least partly obscured by grouping the result with others. It will be noted in the following section that cusum analysis is a much more efficient detector of change than groups of any size on a Shewhart chart. Nevertheless it is worthwhile to run direct plot charts in addition, in order to reveal such individual correlations/explanations. The current section should not be taken to suggest that the direct plots ought to be of groups of results. The function of the moving averages chart has been taken over by the cusum chart. The purpose of a direct plot in conjunction with a cusum chart is to reveal individual correlations and effects. Therefore individual results should be plotted in such a case.

4.4.3 Cusum charts

'Cusum' is a contraction of 'cumulative sum' (of the difference between each successive result and a target value, usually the previous mean). By definition the cumulative sum of differences from the mean is zero.

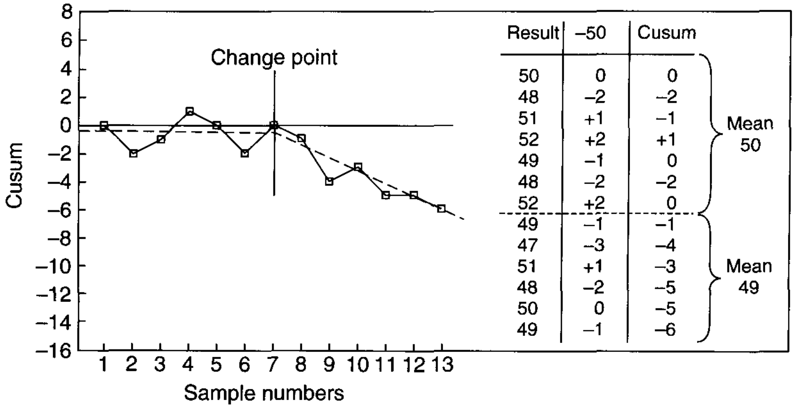


Fig. 4.3 Simple cusum control chart.

So if the previous mean continues to be the mean, a graph of the cusum will have temporary divergences (the extent depending upon the variability of the concrete) but will remain basically horizontal.

However, if the mean changes, even by a very small amount, each successive point on the graph will, on average, differ from the previous point by this amount. The graph will still show the same temporary divergences about a straight line, but the line will now make an angle with the horizontal and the **angle** will be an accurate measure of the change in the mean, and the point of intersection of the best straight line before and after the change will pinpoint the time of occurrence (Fig. 4.3).

The cusum technique was developed in the chemical industry (Woodward and Goldsmith, 1964) and was first used for concrete QC in the UK in the 1970s (Testing Services Ltd, 1970).

British Standard BS 5700 *(Guide to Process Control Using Quality Control Chart Methods and Cusum Techniques)* provides the following advice:

The advantages of cusum charts are explained in detail, and illustrated by examples in BS 5703: Part 1. Their advantages can be briefly summarized as follows.

(a) For the same sample size, a cusum chart will give a much more vivid illustration of any changes that are occurring (purely by visual examination of the plot, without recourse to decision rules).
(b) The cusum chart uses data more effectively, thereby giving cost savings.
(c) The cusum gives clear indication of the location and magnitude of change points in a process.

Possible disadvantages are as follows.

(d) The procedure is comparatively less well known, and there is a problem of re-education of those used to traditional methods. In particular it is sometimes difficult to establish the basic principle that the control parameter is the slope of a plot rather than its vertical ordinate.
(e) It is sometimes maintained that the calculations for the cusum plot are more complex than those for the traditional plot. This is arguable, particularly when associated with the use of microcomputers.
(f) Because average is indicated by slope, staff operating the charts require training to achieve effective interpretation.'

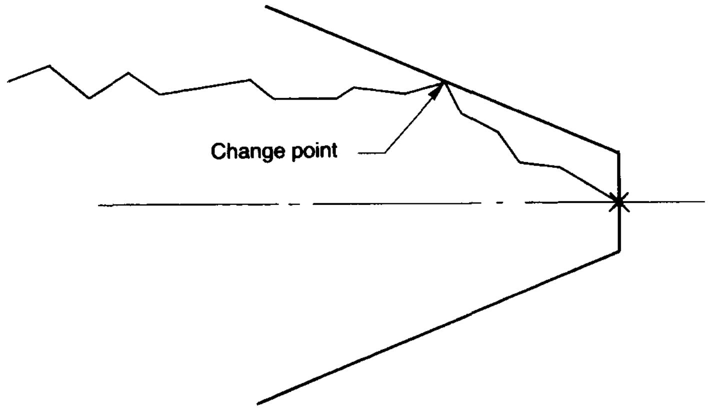


Fig. 4.4 Use of V-mask on cusum chart.

The significance of any particular change of slope can be accurately and simply assessed by the use of a 'V-mask' (Fig. 4.4).

The lead point of the V is placed over the last point on the graph and if the graph cuts the V, a significant change has occurred. The V-mask can be a sheet of transparent material carrying a whole family of Vs, each indicating a different degree of significance.

The system was originally adopted by the British Ready Mixed Concrete Association (BRMCA) and Quality Scheme for Ready Mixed Concrete (QSRMC) as the basis for control although an alternative system involving a countback of the actual number of results above and below the target value of strength is now also permitted (Barber and Sym, 1983). A run of nine consecutive results above or below the target value is taken to establish that a change has occurred. Dewar and Anderson (1992) state that the alternative is simpler to operate but is 'slightly less sensitive than the cusum method'.

Whether the cusum technique is effective or not depends on a number of factors:

1. The most basic factor is whether changes in mean tend to be isolated 'step' changes or to gradually increase in magnitude. The author's experience is that the more important changes do tend to be step changes, although not invariably and uniquely so. If you draw cusum graphs, you will soon see for yourself the extent to which this is true for your concrete.
2. The change points will be much more clearly visible if the general scatter of points is reduced. They will also become clearly visible from a much smaller number of results after the change, if the scatter is low. However, the efficiency of all kinds of control systems are greatly affected by the extent of scatter and in fact the cusum technique, although substantially affected, is better able to function under high scatter conditions than any other.
3. A significant change, as previously explained, results in a change of slope. An isolated error, or non-significant change, appears as an offset to the slope and can usually be readily discounted by eye examination (Fig. 4.5). Such offsets may invalidate the use of V-masks on an automatic basis (i.e. assuming an unintelligent operator) but do little to upset the judgement of a skilled interpreter.

Cusum charts are an important part of the Conad control system. The author has substantially increased their efficiency through his techniques of combining separate grades into a single graph ('multigrade'),

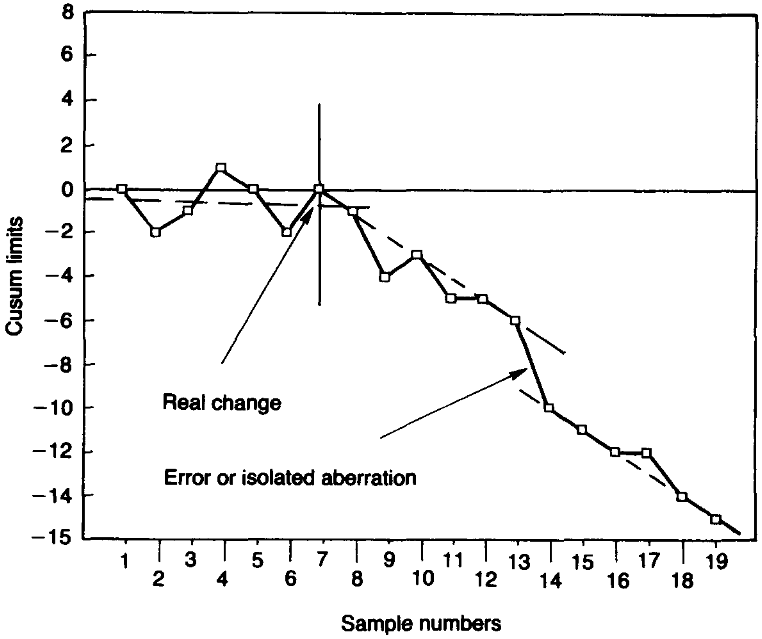


Fig. 4.5 Cusum graph exhibiting both real and non-significant changes.

simultaneously graphing related variables ('multivariable'), and forecasting 28-day strength from early age results.

Cusum graphing appears to overcome reported difficulties (Shilstone, 1987) in correlating related variables such as strength and slump. This is because coincident change points on cusum graphs display an instanta neous correlation unaffected by extraneous influences which may interfere with correlation over a period as in a regression analysis.

Different roles for cusum analysis

In 1996 the author was charged by ACI Committee 214 (Analysis of Test Data) to form a sub-committee to report on cusum analysis. The sub-committee consisted of K.Day (chairman), J.Shilstone Jr., and three UK experts in the technique, B.Brown, J.D.Dewar, and L.Sear. One of the more interesting aspects of the report was the realization that quite different philosophies could be applied to the use of cusum charts and would affect how they should be constructed. The different purposes listed by the committee were:

1. **Diagnostic,** i.e. to assist in determining whether variations were systematic changes and when the changes occurred (single variable) and the cause of variations (multivariable).
2. **Action,** i.e. to show when changes might be needed and what the magnitude of such changes should be.
3. **Obligatory (regulatory),** i.e. as the basis of a specification or certification scheme requiring a specified action to be taken in specified circumstances.

Shilstone incorporated a single variable cusum capability in his computer system as a diagnostic tool. Day used them as a multivariable diagnostic and action tool and the UK scheme uses cusum as an obligatory action tool.

When used as a regulatory tool, a cusum will be based on a specified target and may use a V-mask to determine when a specified limit has been exceeded. Day had objected that anyone could see that a downturn had occurred, long before the V-mask confirmed it. However, the UK members pointed out that there was no objection to taking earlier action, the requirement was that action must be taken if the graph cut the V, not that it should not be taken unless this occurred.

The UK members also pointed out that taking very much earlier action meant acting before there was any statistical certainty that action was necessary. Day pointed out that the certainty need not come from the single variable (strength) alone, as in the UK situation. If the strength downturn were accompanied by a density downturn, and either or both a temperature upturn and a slump upturn, it may be very clear from only two or three results after the downturn not only that it was genuine, but also what caused it (i.e. additional water).

So multivariable graphs are what the producer needs, and a V-mask is a feature which may be useful in a specification (including perhaps an internal code of practice specification for a large and widespread company) but will not alone give sufficiently rapid control action to enable the attainment of very low variability. An interesting possible use is in the situation (apparently common in the USA) where mix adjustment is currently not permitted. As discussed elsewhere this situation is very detrimental to the future progress of concrete technology. An alternative might be to permit a (downward) adjustment of cement content only if and when it is confirmed by the cutting of a V-mask.

4.4.4 Factors affecting rapidity of control action

The use of a V-mask has been described in the section on cusum analysis. It is a beautifully simple method of applying a rigorous mathematical test to establish the degree of certainty involved in a downturn on a cusum chart. Although the originators of the cusum technique, the UK concrete industry appears to have made little subsequent progress in its application. They still consider that as many as 10 to 14 results may be necessary after a change point before action can be taken. In the author's view this is too long to wait and he expects to take action only 3 or 4 results after a downturn has occurred (Day, 1983). Some of the factors involved in shortening the reaction time are:

1. **The degree of certainty considered necessary for action.** The consequences of acting unnecessarily (i.e. on a chance variation rather than a confirmed change) are minimal. They may involve the unnecessary use of 5 or 10 kg of cement per cubic metre of concrete produced for the one or two days during which the situation has been misjudged. This can be offset by normally working closer to the minimum strength level with the greater confidence that remedial action will be taken more rapidly. A potentially more serious objection which has been raised is that continual changes of this nature can invalidate the statistical analysis on which the control is based. Such an invalidation would be the artificial increase of standard deviation which is brought about by a change of mean strength during the analysis period. This objection is not sustainable because the changes will be relatively small (generally 2 or 3 MPa), apply for a short period, and are partly counterbalanced by the apparently low results which caused them to be applied.

2. **The degree of variability being experienced.** If the cusum graph prior to the change is extremely smooth and consistent in direction, the change will be much more clearly seen. This is a valuable general feature of QC, the better the control, the easier it is to detect problems and so further improve control. The ease of detecting a downturn is directly proportional to the standard deviation being experienced prior to the downturn.
3. **The effectiveness of the method used to combine results of many grades into a multigrade analysis.** The author's system expresses each result as a proportion of the average result to date for the particular grade of concrete, and this appears to be much more effective than the UK system of selecting a control grade and attempting to adjust results from other grades so that they can be analysed as part of the control grade. Furthermore, the author's system can equally well be applied to related variables such as slump, temperature and density. It is necessary (or at least desirable) to combine results from different grades into a single analysis in order to accumulate results more quickly on a cusum graph. If only analysed in separate grades, a large number of results could be obtained in total before any grade comprised sufficient results to confirm the downturn (or upturn). However, if the method of integration were inaccurate, a spurious higher variability would result and this may explain the high variabilities considered normal in the UK. (However, the UK practice of transporting freshly made specimens to the testing lab. may also be a substantial influence.)
4. **The use of multivariable cusums.** The automatic plotting of cusum graphs of slump, density, concrete temperature, average pair difference and 7 to 28-day gain in addition to predicted and actual 28-day strength is very helpful. Admittedly the gain and pair difference graphs are 'after the event' as regards early control action but they do monitor variations in cement properties and the quality of testing. Slump, and especially density and concrete temperature (of the fresh concrete, at the time the specimens are moulded) show good correlation with strength changes and are available even **before** the early strength results (especially if these are as late as 7 days). If a strength cusum downturn correlates with either a density downturn or a temperature increase, and those changes continue on past the latest available strength results, then there is a very substantial practical certainty that the downturn is real, even though the mathematical probability of this, judged from the strength cusum alone, may be quite limited.

This is discussed further in section 12.6.

4.5
COMPUTERIZATION OF QC PROCEDURES

4.5.1
Change point detection

Control charts have always been an excellent way of assessing test results. There is no way of improving upon the performance of an intelligent and interested person carefully examining multivariable, multigrade, cusum graphs. However, as further discussed below, result assessment is often left to persons of lesser commitment or intellect. It is therefore desirable to back up the graphical display with an automatic, computer based analysis which will advise the operator that a significant change has occurred.

Such an automatic detection can be applied to all variables monitored, not only strength. Further, the computer can automatically examine the extent to which a revealed change in strength is explained by changes in slump, temperature, aggregate grading, etc., and whether this explanation tallies with a density variation. The mechanism of this examination is to convert the changes in other variables into changes of water demand and then to evaluate the effect of the water change on strength and density. Any unexplained density reduction is then evaluated as either additional entrained air or a lack of compaction.

A system can be set up to automatically clear the screen and present a report of the above nature whenever inbuilt limits are infringed. Any departure from such rigidity carries the risk that the feature will be badly adjusted or switched off. However, it is usually assumed that someone of sufficient intelligence to make reasonable choices will be in effective charge. The system can then be supplied with a user-adjustable tuning factor, so that it can be set to react to every hint of a change, or only to massive, solidly established changes, or anywhere in between.

The author has examined the relative efficiency of several alternative automatic detection systems in a paper entitled *How Soon is Soon Enough*? The paper is reproduced in part as section 12.6. The economic importance of selecting an efficient analysis system becomes apparent when it is realized that it clearly justifies a reduced testing frequency for a given degree of security. The aspect of having a smaller amount of concrete 'at risk' (i.e. not yet proven to be acceptable) is probably much more important, but not so easy to present as a justification to non-technical administrators.

4.5.2
Development of computerization

Chapter 3 illustrated that computerization helped the mix design process by making it quick and simple to use, however complicated the calculation process used. The advantages are even greater in QC.

In essence quality control involves an unending stream of data, most of which simply says there is no problem. The tendency is either to hand the data to a junior employee or to leave it sitting in the basket while 'more important' things are attended to, and then flip quickly through the test certificates when they are already several days old. There is actually very little which is more urgent and important to attend to than a low concrete test result. The difficulty is to know whether the results are in this urgent and important category, or some of the 99% which do not require any attention at all.

The 'quick flip' is certainly too little, too late. Starting in 1952 (Day, 1959) the author has devised a succession of systems which enabled 'handing the data to a junior employee' to be a much more satisfactory system. Even the first such system was effective in promptly bringing any problem to the author's attention but there has certainly been subsequent improvement in the selectivity of the process and the amount of supporting data available. A major step forward was the introduction of cusum in the mid-1970s. Probably the most significant was the introduction of the computer (using spreadsheet programs) in the early and mid-1980s. Certainly another large step has been the conversion of the spreadsheets to compiled programs (during 1991). Philosophically a most important step has been linking the QC computer to the batching computer in the late 1980s. This was so important because it covered 100% of the concrete produced (Day, 1989) and so finally eliminated any doubt as to whether the concrete sampled truly represented the whole of the concrete produced. The data to accomplish this has been output by some computer batching systems at least since the late 1970s, but it had previously been impractical to use it effectively.

A batch plant may output a 4 m strip of paper daily showing the actual weight of every ingredient batched. Such a record gives extremely useful information if two hours a day is spent examining it, but usually no one intelligent enough to make effective use of the data has two hours a day to spend on it. This meant that such data was only used for reference **after** a problem was detected by other means. The new arrangements (described later) enable a graphical presentation showing every error in the batching of every

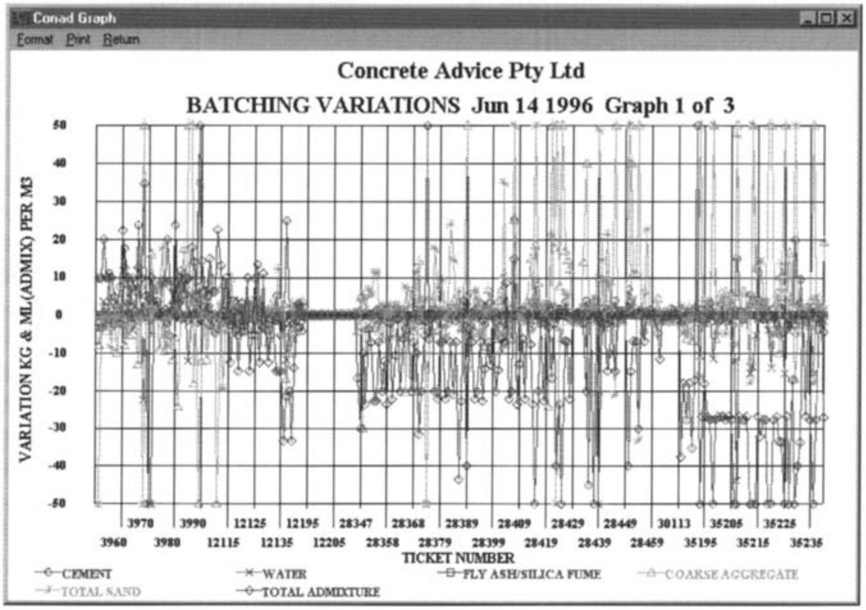


Fig. 4.6 Batching error graph.

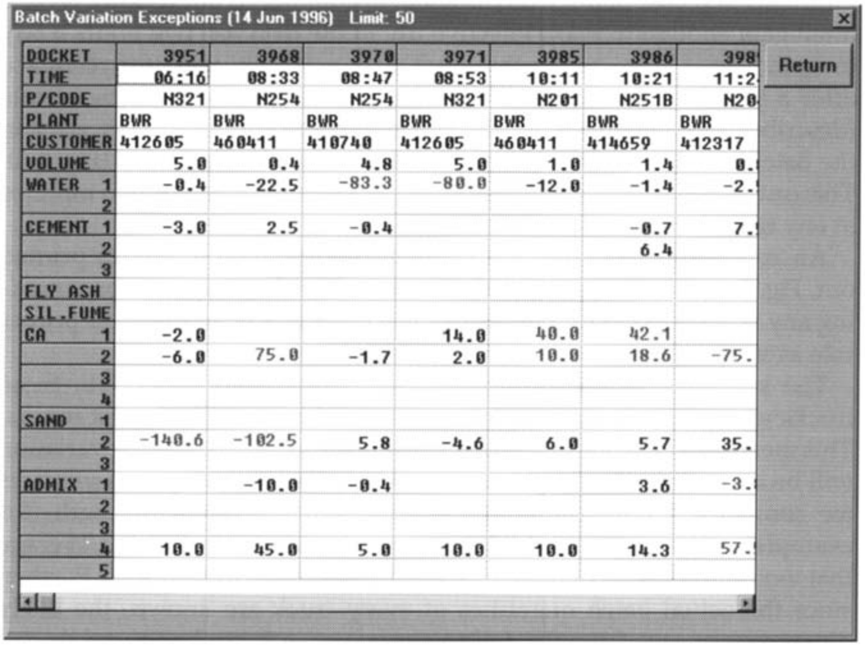
ingredient of each truck for a whole day (Fig. 4.6). This can be comprehensively inspected in a matter of one or two minutes at any time during the course of the day.

An 'exceptions report' (Fig. 4.7) can be displayed on screen (or printed out, Fig. 5.6) on every truck with any ingredient having an error exceeding any nominated limit. The 'as batched' data is now fulfilling its proper role as the primary source of problem detection.

The system goes on to automatically select from these data those trucks which have been sampled and to combine it with the test results. This means that any variation in the test results due to batching variation will be allowed for in the predictions which the control system prepares for comparison with the actual results. So the average strength (for example) can be not merely the average actual result, but the average that would have been obtained for the intended batch quantities. Further, since the actual batch quantities of every truck are known, the likely divergence of each from the daily average strength can be accurately calculated. So the lowest strength provided can be established, even if the truck in question was not tested. It is interesting to consider that a strength calculated in this way may be **more** accurate than an actual test on the truck in question.

4.5.3
Recent developments

At the time of writing the first edition (1994) it was already unthinkable that anyone should seriously undertake quality control of concrete without using a computer. At the current time (1998) the operating speed, data storage capacity, cost, and ease of use justify a further change of attitude. It is no longer necessary to be economical in demands made on the computer. This makes it possible to build a wide range of options and flexibility into programs and to have the computer perform a wide range of tasks automatically.



DOCKET	3951	3968	3970	3971	3985	3986	398
TIME	06:16	08:33	08:47	08:53	10:11	10:21	11:2
P/CODE	N321	N254	N254	N321	N201	N251B	N20
PLANT	BWR	BWR	BWR	BWR	BWR	BWR	BWR
CUSTOMER	412605	460411	410740	412605	460411	414659	412317
VOLUME	5.0	0.4	4.8	5.0	1.0	1.4	0.
WATER 1	-0.4	-22.5	-83.3	-80.0	-12.0	-1.4	-2.
2							
CEMENT 1	-3.0	2.5	-0.4			-0.7	7.
2						6.4	
3							
FLY ASH							
SIL.FUME							
CA 1	-2.0			14.0	40.0	42.1	
2	-6.0	75.0	-1.7	2.0	10.0	18.6	-75.
3							
4							
SAND 1							
2	-140.6	-102.5	5.8	-4.6	6.0	5.7	35.
3							
ADMIX 1		-10.0	-0.4			3.6	-3.
2							
3							
4	10.0	45.0	5.0	10.0	10.0	14.3	57.
5							

Fig. 4.7 Batching exceptions display.

One example of this is in relation to batch quantities. The computer can readily store huge quantities of batch data and can rapidly search through it to find particular records. The computer can automatically find the details of the truck from which each test sample was taken and calculate the effects of any batching error.

Another example is in respect of field test data entry. No two separate operating units, even if part of the same company, can ever agree about exactly what should be entered. Still less can they agree on the format of the field testing sheet. Yet it is highly desirable that the computer input screen should exactly match the field testing sheet (to avoid data entry errors) and that data should be able to be transmitted worldwide in the case of multinational companies. The solution is to provide a database format having a position for every imaginable item of data and then to allow each individual user to select which of these items will be displayed on their particular computer screen. Using the Windows system it is easily possible to move (drag and drop) entry items around the screen as desired. No longer is there any problem when it is decided to record additional data for future samples. The pigeonhole already exists for it and the previous data remains fully accessible. All users can access the data.

Previously it was necessary to use direct modem transmission to exchange data internationally. This called for a degree of operator expertise and was subject to substantial telephone costs and occasional difficulty. It is now ridiculously cheap and simple to transit vast quantities of data by attaching files to email messages. A major ready mix supplier can transmit several month's accumulation of test data to the other side of the world in a few minutes at a cost of less than a dollar.

Integration of different computers also used to be a problem but we now happily interface many different personal computers (PCs) with each other and with mainframe computers.

Computer systems have moved on from barely being able to cope with the numerical QC situation to integration with general record keeping and report preparation, accounting, production engineering (stock records, equipment performance monitoring, individual employee performance monitoring, method study).

The twin abilities to combine differing types of data into a single analysis and to split up data into the finest possible categories transforms many analysis situations. One example of this is the change it has been possible to make in multigrade analysis. The UK technique involves adjusting test results from different grades of concrete to the value they would have had if they had had the same cement content as a control grade (so that they can appear on the same cusum graph). The Conad system now calculates individual average values not only for the strength of each separate grade but also for its density, temperature, slump, average 7 to 28-day strength gain, average pair difference and a dozen other items. A cusum graph can then be obtained by cusumming divergences from these means as though they were all from the same mean. If it is not obvious that this is a more effective solution, it is only necessary to observe the relative efficiency of the two processes in early detection of change and the cause of change, and in attainment of low variability.

The use of truck mounted devices for controlling mixing, workability and water addition has become a practical proposition (section 12.1.2). This closes what was perhaps the last remaining gap in the QC chain. The devices store substantial data, which can later be downloaded and integrated with other QC data.

Early age strength monitoring has proved a very popular field. The Conad system for this is relatively simplistic but, as with most features of the Conad system, it derives adequate accuracy by a feedback technique (section 12.2). It is used both to obtain early prediction of 28-day strength and to measure early age *in situ* strength to establish readiness for stressing, stripping, lifting, etc.

5
The Conad quality control system

5.1
DEVELOPMENT OF THE SYSTEM

The current system had its origins in the UK in the early 1950s. By 1954 the author had originated a comprehensive system of multivariable Shewhart control charts with the guidance and encouragement of O.J. Masterman. System details were published by Masterman (1958) and formed the basis of a series of articles by the author after his relocation to Australia in 1955 (Day, 1958–9).

Little further progress in quality control (as opposed to mix design) was made until the late 1960s (Day, 1969). The author's company Concrete Advice Pty Ltd was formed in 1973 to provide QC services on many major buildings in Melbourne, Australia, commencing with Collins Place (a twin 50-storey project and the first use of 55 MPa *in situ* concrete in Australia) (Day, 1979) and the Victorian Arts Centre (the Melbourne equivalent of the Sydney Opera House). The system employed a newly available Hewlett-Packard hand calculator (HP41c) which was able to perform statistical analysis. This permitted use of a running mean and standard deviation of the last ten results without excessive labour cost.

Cusum analysis was introduced in 1975 (Day, 1979) and computerization commenced in 1980. Little headway was made on the latter until the introduction of Lotus 123 in 1982. The whole system was substantially complete on a spreadsheet basis by 1987 (Day, 1989) and was compiled into a C language version during 1991/92 (Day, 1992).

The Windows version was largely written during 1996 but extensive revisions continued through 1997 and into 1998 as the system was expanded and adapted to integrate with the very extensive activities and mainframe computer use of our major clients, CSR Readymix and Boral (Day, 1997, 1998a, b). Recent revisions have related to mix design and/or administration, including invoicing, report printing, and more efficient data flow rather than innovation in quality control techniques. However, an important new feature currently under development is the integration of data automatically recorded during delivery by equipment mounted on each truck.

There has been some equivocation about whether specific surface mix design is preferable to other systems but there is no question that multigrade, multivariable, cusum analysis is the most effective tool for QC.

5.2
ESSENTIAL FEATURES

The essence of good QC is to take action quickly, to take it in an accurately chosen direction, and to a measured extent. It is not good enough to start looking for a cause and considering a solution after a problem has been proved to exist. Rather the **potential** causes should be continuously monitored so as to **predict** a problem before it occurs and solutions to all likely problems should be **preplanned**.

This is not quite the same as QA, although quite similar. The fine distinction in philosophy is that QA aims to establish limits for all incoming materials and all production features and to inspect to ensure that all are certified by an authorized person as being within those limits. The Conad system, which the author describes as operating on 'feedback QC', aims to keep the final product between limits. Some definite limits on materials and processes may be set from time to time (as having been found to cause problems) but the emphasis is on adjustment rather than rejection and on early detection and rapid reaction (section 4.1.2). Conad does incorporate a QC Diary feature, which can be used to keep a comprehensive certified record of the quality situation on a daily basis. Using this feature it is practicable to go back at any future time to see exactly what records were available to the person in direct charge of QC, what decisions were made, and who was informed. There is provision for the report to be inspected and countersigned by a second person. This feature is an attempt to keep more comprehensive QC records than have ever been kept and yet to limit the time spent on such record keeping to only a few minutes per day (except when a major problem emerges). This is possible at the cost of providing for large data storage capacity. It is simply cheaper to store everything, everyday, than to spend time deciding what to store and when.

The Conad system is designed to satisfy these requirements. The most essential feature is **multivariable cusum** graphing. This means that variables such as slump, density and concrete temperature are graphed along with strength. This satisfies the requirement that the cause will normally be readily apparent. It also provides a degree of prediction since the other variables are known earlier than strength.

The system derives its reaction speed from computerization, a **multigrade** technique and the use of cusum (cumulative sum) analysis. More recently the system has extended into an instant analysis of computer batching records and further development of the new technique for predicting 28-day strength at 24 hours or less (Day, 1991). The technical features have to be matched by an appropriate attitude of mind to achieve maximum benefit.

Speed and accuracy in corrective action is greatly assisted by interaction with the system of computer mix control described in Chapter 3.

5.3
COMPREHENSIVE INTEGRATION

Until late 1991, the main use of the system was as a number of separate parts. The analysis of normally generated test results on behalf of the purchasers of concrete for major projects had been the mainstay of the author's company, Concrete Advice Pty Ltd, since its inception in 1973.

The mix design system, in a gradually developing form, has been used mainly by the author himself and a few colleagues over a period of almost 40 years. It has been used on behalf of a large number of employers and clients in many countries but until recently had not been sold for use by others to any great extent. Recent developments have been especially aimed at integrating mix revision with QC. This has the capacity both to produce more uniform concrete and to recover the entire cost of the QC system by reducing the necessary operating margin.

The analysis of computer batch plant output data is perhaps the simplest of all the techniques, yet of considerable importance in the overall concept of quality assurance. It is well justified if used solely to indicate the need for mechanical adjustments in the plant itself, and this was initially the main volume of use. However, it has great significance in justifying reduced rates of test sampling and in determining the cause of any aberrant test results. It can also be very useful in ways not directly connected with concrete technology such as stock reconciliation, invoicing for both concrete and testing services, control of delivery truck usage etc.

Early age testing (24 hours or less) combined with a maturity assessment enables a prediction of 28-day strength without accelerated curing. In 1990/91 the Conad Erliest (early estimation) System (Day, 1991) was successfully used for early age *in situ* strength control. However, its use as an integrated part of the control system only commenced in 1992.

The first edition showed a flow chart of a totally integrated system but we no longer regard a flow chart as relevant. Data input is continuously received from many sources and drawn upon by many others in any sequence. It is relevant only to list the input and output sources.

Data Input from:

- Data from materials testing and that provided by suppliers of aggregates, cement, and admixtures.
- Batching plant: actual and intended batch quantities (every truck), customer details.
- Delivery control history (eventually every truck?).
- Fresh concrete testing data: fresh concrete tests, *in situ* temperature records.
- Hardened concrete tests at laboratory.

A desirable goal is never to input any data twice, e.g. information about who is the client, at what address, and what concrete has been ordered need not be re-input by the field testing officer or laboratory staff, because it is already known at the batch plant and can automatically appear on records when delivery docket number is entered. It is important to always use delivery docket number as the primary identification because this is the only positive tie between batching, delivery, and testing records.

Data Output to:

- Test reports (although traditional test reports are of limited value except when in contention).
- Production analysis. The available data is of considerable use for business analysis, invoicing (both concrete and testing services), truck driver payment etc.
- Quality regulation, which is the main topic of this chapter.

As noted in the first edition, it is quite practicable to provide the software for a self-regulating batch plant, although this has still not actually been attempted. The current software certainly analyses the data and recommends mix changes but still leaves these to be manually implemented.

5.4
DATA STORAGE AND RETRIEVAL

A major advance introduced with the new Windows system has been improved data storage and retrieval. Full advantage has been taken of reduced costs in time and computer capacity of processing and storing very large amounts of data. We find that different clients wish to record different things and to analyse them in different ways, according to the regulations and criteria of different countries. The simple answer to this has been to provide for the storage and analysis of every conceivable thing, and then to provide a few extra ‘user nominated variables’ for anything we may have forgotten. However, the user is then permitted a choice of which data will actually be entered.

There are two potential costs or disadvantages in this approach. One is the increased processing time and storage capacity needed. This simply requires spending an extra $500 or so on your computer to fully offset.

The other is the potential for being swamped in detail. This is important. Many times (not only in concrete QC) the situation arises when more is less, when data are so extensive that they cannot be fully comprehended and less is learned. This has been countered in three ways. Firstly, the input system allows the user to nominate exactly what shall be input. Secondly, much of the data processing, especially including combining data from different sources, takes place automatically. Thirdly, the data to be used can be precisely specified and automatically extracted from the data pool. Clients are exhorted not to attempt too much initially and may start by using 10% of system capacity. They can make their own gradual transition to more sophisticated use because the capacity is there, no upgrade is necessary. It is like a 50 seat bus picking up two or three people on some routes but able to utilize full capacity whenever needed.

It is important to understand the principles that:

- The computer has the capacity to store literally everything that we know about the concrete being produced.
- There is very little disadvantage in storing huge amounts of data providing that it takes little time or effort to enter it (i.e. it enters at least semiautomatically), and that it is possible to select semi-automatically which data is actually required for a particular purpose and leave the rest behind.
- A system must contain the means of selecting and correlating data according to a very large range of criteria so that the user not so much selects data as specifies which data the computer is to select.
- A system must be able to carry out specified analyses of data and present the results of an analysis in a variety of easily selected formats so that users can see as much detail as they require for a particular purpose and no more.
- It is certainly beneficial if the assessment of the data, including any necessary calculations, can be done automatically and the user be presented with the results. However, when this is done, it is important to build in checks or display extreme values, or present a graphical output, so that the user does not lose touch with reality. The news that 95% of your troops have survived may not convey the same realization of the situation as seeing the 5% of dead bodies.

Implementation of these concepts requires:

- All constituent materials test data is entered (Fig. 5.1), preferably as it is produced. For example, Conad allows actual sieve masses to be entered and automatically calculates percentages passing and retained, specific surface (and fineness modulus, although Conad does not use it). Past entries can be viewed graphically (Fig. 5.2 or 5.3) and the computer can produce the latest grading on any nominated date, or the average over any nominated period, etc. Cement and pozzolan data is similarly treated but their format is currently under review.
- Batch plant data is automatically processed. The computer looks in the materials database, obtains data for the date in question and calculates yield, density, combined grading and MSF value for every truck of concrete produced.

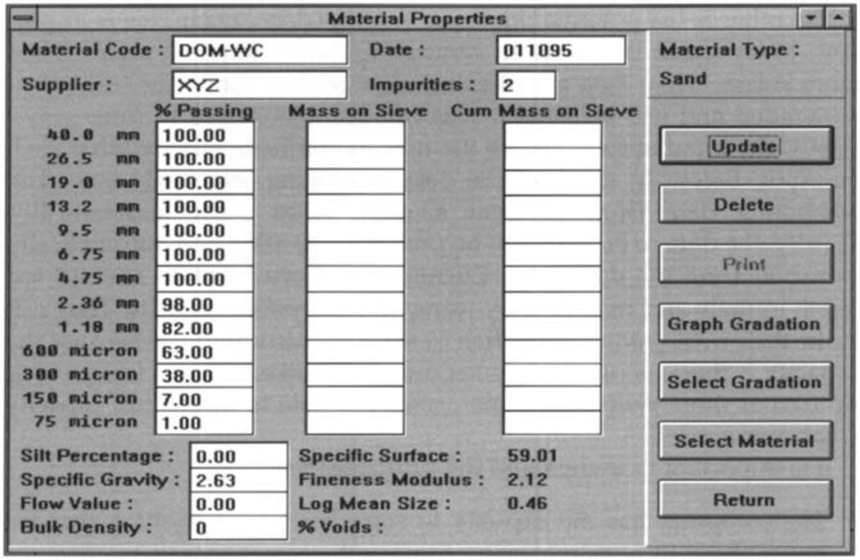
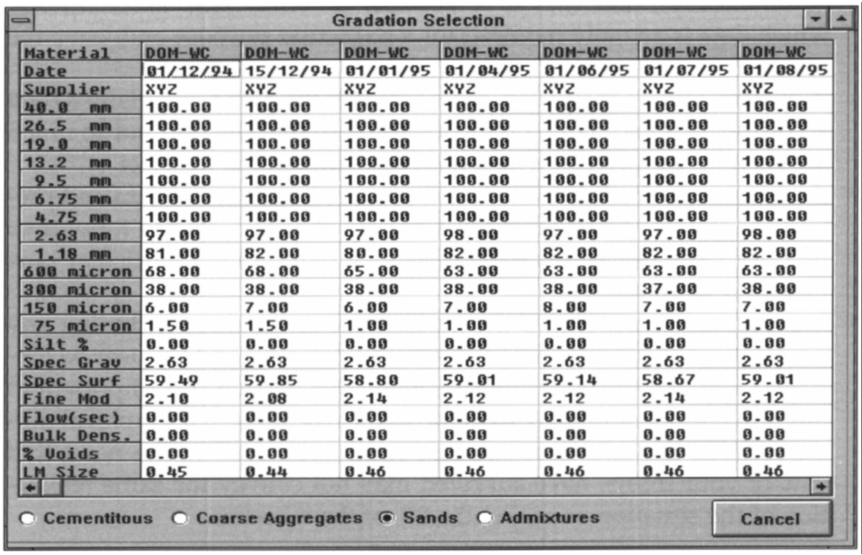


Fig. 5.1 Material gradings.



Material	DOM-WC	DOM-WC	DOM-WC	DOM-WC	DOM-WC	DOM-WC	DOM-WC
Date	01/12/94	15/12/94	01/01/95	01/04/95	01/06/95	01/07/95	01/08/95
Supplier	XYZ	XYZ	XYZ	XYZ	XYZ	XYZ	XYZ
40.0 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
26.5 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
19.0 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
13.2 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
9.5 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
6.75 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
4.75 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
2.63 mm	97.00	97.00	97.00	98.00	97.00	97.00	98.00
1.18 mm	81.00	82.00	80.00	82.00	82.00	82.00	82.00
600 micron	68.00	68.00	65.00	63.00	63.00	63.00	63.00
300 micron	38.00	38.00	38.00	38.00	38.00	37.00	38.00
150 micron	6.00	7.00	6.00	7.00	8.00	7.00	7.00
75 micron	1.50	1.50	1.00	1.00	1.00	1.00	1.00
Silt %	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Spec Grav	2.63	2.63	2.63	2.63	2.63	2.63	2.63
Spec Surf	59.49	59.85	58.80	59.01	59.14	58.67	59.01
Fine Mod	2.10	2.08	2.14	2.12	2.12	2.14	2.12
Flow(sec)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Bulk Dens.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
% Voids	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LM Size	0.45	0.44	0.46	0.46	0.46	0.46	0.46

Fig. 5.2 Material gradings listing.

- Field test data requires only truck delivery docket number for identification. The computer already has all the ordering and batching data. Some clients are looking into supplying some or all of this data by bar coding on the delivery docket so that it will be available to the field testing officer (who would have a notebook computer with a bar-code reader). The data supplied may include a maximum permissible water addition on site. At worst, arrival and testing time, slump, appearance of concrete (a single code letter), weather (a single code letter), number of specimens, etc., are written down and entered in the computer on return to the laboratory. At best every truck will carry measuring and recording equipment

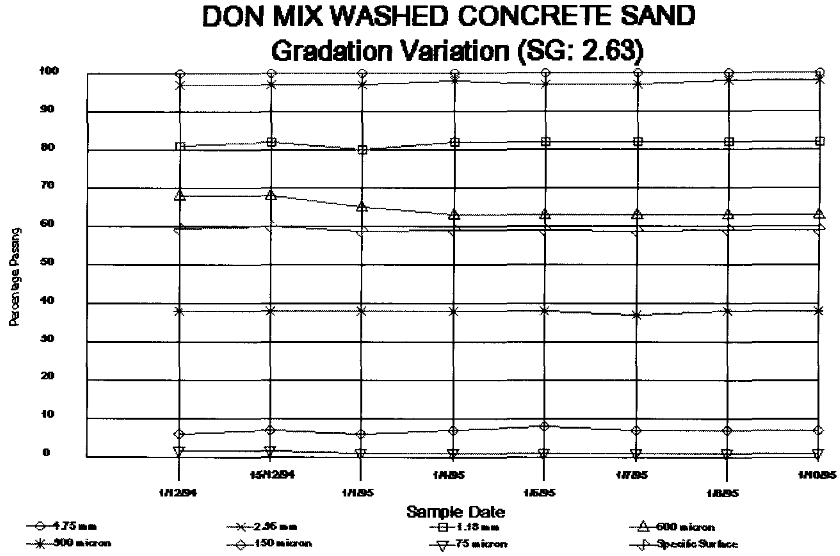


Fig. 5.3 Graph of grading variations.

which will be able to download age at discharge, slump, any water additions, etc., of every load delivered, at the end of the day.

- The computer will maintain batch data in two databases. One will cover every truck. The other will contain only the batch data for trucks that have been sampled for testing, stored with that test data. Obviously there will be far more data entered per day in the former than in the latter. The former may be archived on a bimonthly basis (so that they are still readily available at 28 days) but the latter may be archived on an annual or six monthly basis. These days 'archived' means stored as a zipped (condensed) file on a disk holding 100 Mb, or even 1000 Mb; such records are still quite readily available if required. However, there is not really any difficulty in holding all records semi-permanently in the working computer now that a 6000 Mb hard disk can be had in a low cost computer.
- Laboratory test data will also go direct into the database. In some cases this can be done directly from the testing machine and electronic measuring device via a computer hook-up.
- It is important that comments at any stage are standardized. A database of comments is established for any situation in which a comment is likely to have value. This has the dual advantage that the comment requires only one character to record it and also that the computer can readily recall (or specifically omit) all records having the same comment.

5.5 COMPUTER BATCHING ANALYSIS

During batching, the computer stores the actual batch quantities as well as the intended batch quantities and automatically integrates them with aggregate grading data. A combined grading (all materials, including cement, water, and even entrained air) of every truck of concrete produced is automatically put on file. On request, these are passed to an analysing computer. The latter is likely to be at one or more distant locations, such as the laboratory and the Technical Manager's desk. If any errors outside a preset limit occur during batching, most current systems will stop and sound an alarm so that a human operator can decide what action to take. The system described here is designed to make it easy for supervisory personnel to check what action the operator did take, and also to see the accuracy with which the system is operating.

Since at least the late 1970s systems have been available with the capacity to print out actual batch quantities. The difficulty has been the considerable volume of such data. This is such that no one with sufficient knowledge to make effective use of such data has had enough time available to analyse it as a routine. The effect has been that the data was referred to only **after** a problem was discovered in some other way, e.g. a low test result. Such use discards the crucial advantage that, for the first time, a 100% inspection facility is available.

There always has been, and probably always will be, a degree of error in the extent to which the test results truly represent the concrete batches tested (although the Conad system assists in revealing and reducing such error). There has also been a degree of uncertainty in the extent to which the batches tested represent the whole of the concrete produced. It is this latter uncertainty which it has recently become possible to eliminate almost entirely.

A further development has been the fitting of control and recording equipment to mixer delivery trucks (section 12.1.2). Such a system can detect and quantify the addition of water during delivery even if not from its own tank. However, there remains the problem of addition of water to pump hoppers after test samples have been taken.

Perhaps someday it will be required that a continuous record of pumping pressures be automatically recorded and made available to those in charge of QC. It would be reasonably easy to detect, and even approximately quantify, addition of water from such records.

One aspect of the uncertainty is that the concrete sampled may have had a higher or lower than intended cement content (or other significant difference such as excessive sand content). It is now possible, using the Conad system, to make a correction for such variations in the subsequent analysis, along with differences from intended slump, expected concrete temperature, air content and grading of input materials. **In effect the actual test result can be converted into the test result which would have been obtained had the sample been truly representative of the intended concrete, produced under the expected conditions, from the expected materials**.

It is then possible to establish which actual batch of concrete had the most unfavourable combination of characteristics and therefore the lowest expected strength for the grade in question for any particular day or week. Theoretically, this may mean that only one or two samples per day need be taken for the characteristics of every truck of concrete produced to be known. This would be going too far, but certainly substantial reductions in frequency of sampling are justified.

The comprehensive analysis facility is semi-automatic but still a little too elaborate for frequent routine use during a normal day. As already noted, a substantial degree of immediate protection is already provided by most systems in that they will stop and sound an alarm if a batching error outside a preset limit occurs. The Conad system adds to this a facility to screen a graphical display (Fig. 5.4) showing, on the one screen, every error in every ingredient batch weight of every individual batch for the whole day to the time of calling the display. It takes only a few seconds to call up the display and check whether there are any problem batches. Having done this, a 'plus or minus' limit value can be keyed in and the system will show a display (Fig. 5.5) truncated to the limits entered and expanded to fill the screen. It will also display and/or print out an 'exceptions list' of all non-conforming batches (Fig. 5.6). Such a daily list (and it should be kept short) could be handed to a nominated person for further investigation. It is our experience that if this is done, the errors become fewer and smaller as time goes by.

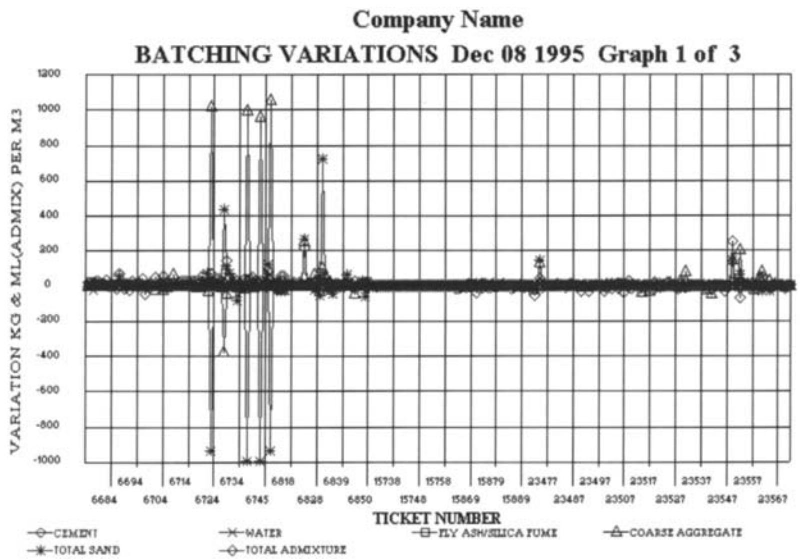


Fig. 5.4 Batching error display.

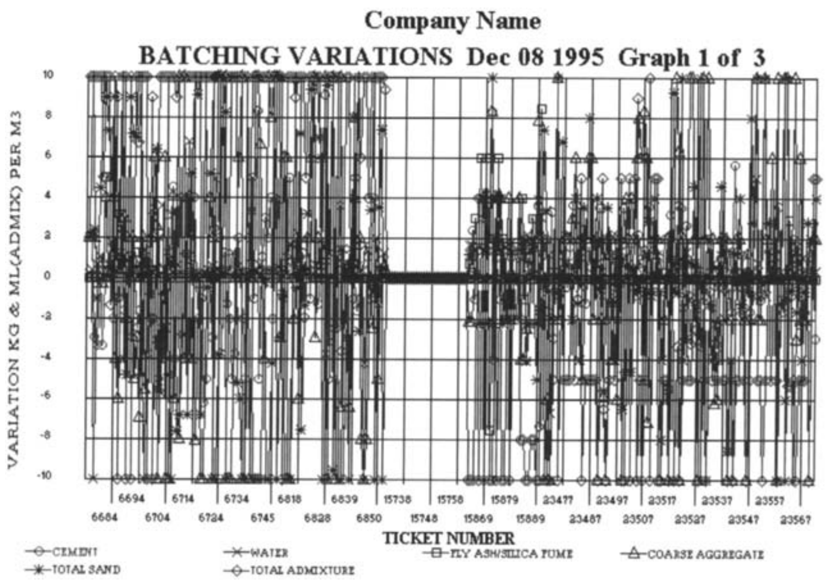


Fig. 5.5 Truncated batching error display.

A difficulty in analysing data is that, in spite of many technological advances, water content is often not fully reliable. To counter this, the system calculates a theoretical water content from slump, temperature and MSF value. The system can then display calculated or predicted strengths based on either or both of these water contents. For a single test result it may not be obvious where the truth lies. However, multivariable graphing over a period clearly shows the difference between defective testing, surreptitiously added water, and truly varied water demand (e.g. through grading variation, silt content and the like).

Conad Program
Batching Variation Data Exception Report With Limit ± 10 14/8/96

DOCKET NUMBER	PLANT	TIME	PRODUCT CODE	WATER 1	WATER 2	CEMENT 1	CEMENT 2	CEMENT 3	CEMENTITIOUS ASH	CEMENTITIOUS FUME	COARSE AGGREGATES 1	COARSE AGGREGATES 2	COARSE AGGREGATES 3	COARSE AGGREGATES 4	SANDS 1	SANDS 2	SANDS 3	ADMIXTURES 1	ADMIXTURES 2	ADMIXTURES 3	ADMIXTURES 4	ADMIXTURES 5
6675	BWR	05:25	KC280	0.2	0.0	0.0	2.0	0.0	0.0	0.0	0.0	2.0	0.0	0.0	0.2	1.8	0.0	20	-1	0	0	0
6676	BWR	05:47	N204B	0.4	0.0	0.0	2.0	0.0	0.0	0.0	0.0	2.0	0.0	0.0	5.8	7.0	0.0	20	0	0	0	0
6677	BWR	06:13	KC280	-24.8	0.0	0.0	-3.0	0.0	0.0	0.0	0.0	2.0	0.0	0.0	0.4	2.0	0.0	20	0	0	0	0
6678	BWR	06:20	N204	0.7	0.0	0.0	-3.3	0.0	0.0	0.0	0.0	-0.3	0.0	0.0	-5.3	4.3	0.0	23	2	0	0	0
6679	BWR	06:21	N254	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-2.0	6.5	0.0	25	0	0	0	0
6680	BWR	06:31	N204	0.7	0.0	0.0	-3.3	0.0	0.0	0.0	0.0	-0.3	0.0	0.0	8.0	4.3	0.0	7	-2	0	0	0
6682	BWR	06:41	N251B	0.7	0.0	0.0	0.0	0.0	0.0	0.0	-3.3	3.7	0.0	0.0	1.7	5.7	0.0	33	1	0	0	0
6684	BWR	06:47	S328SS	0.2	0.0	-2.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.0	0.0	9.0	0.0	10	1	0	0	0
6686	BWR	07:03	N251B	0.4	0.0	0.0	-1.0	0.0	0.0	0.0	4.0	-10.0	0.0	0.0	-1.2	4.4	0.0	20	1	-40	0	0
6687	BWR	07:17	N201B	2.5	0.0	0.0	1.3	0.0	0.0	0.0	88.8	-86.3	0.0	0.0	42.5	12.5	0.0	75	6	0	0	0
6688	BWR	07:21	N254B	0.7	0.0	0.0	-1.7	0.0	0.0	0.0	0.0	3.0	0.0	0.0	-2.3	-1.7	0.0	25	0	0	0	0
6689	BWR	07:29	N251B	0.4	0.0	0.0	0.0	0.0	0.0	0.0	-8.0	6.0	0.0	0.0	-1.2	-3.6	0.0	20	0	-40	0	0
6690	BWR	07:38	S328SS	0.4	0.0	1.0	0.0	0.0	0.0	1.0	0.0	0.0	0.0	0.0	0.0	9.0	0.0	10	1	0	0	0
6691	BWR	07:46	N251B	0.4	0.0	0.0	-3.0	0.0	0.0	0.0	0.0	2.0	0.0	0.0	6.8	0.4	0.0	10	0	-40	0	0
6692	BWR	07:59	S328SS	0.5	0.0	0.0	0.0	0.0	0.0	1.3	0.0	0.0	0.0	-5.0	0.0	7.0	0.0	25	5	0	0	0
6693	BWR	08:07	N234B	0.5	0.0	0.0	-2.5	0.0	0.0	0.0	0.0	-3.0	0.0	0.0	8.3	6.5	0.0	8	-1	0	0	0
6694	BWR	08:10	N204B	0.7	0.0	0.0	-1.7	0.0	0.0	0.0	0.0	-7.0	0.0	0.0	1.3	11.0	0.0	23	-2	0	0	0
6695	BWR	08:14	N251B	0.4	0.0	0.0	1.0	0.0	0.0	0.0	4.0	-2.0	0.0	0.0	-1.2	-3.6	0.0	10	1	0	0	0
6696	BWR	08:20	N204	1.1	0.0	0.0	1.7	0.0	0.0	0.0	0.0	-5.6	0.0	0.0	-16.1	0.6	0.0	38	-2	0	0	0
6697	BWR	08:33	N204B	0.5	0.0	0.0	-3.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.3	1.0	0.0	4	-1	50	0	0
6701	BWR	08:49	N201B	2.0	0.0	0.0	-5.0	0.0	0.0	0.0	-25.0	0.0	0.0	0.0	-2.0	-3.5	0.0	50	-5	0	0	0
6702	BWR	09:00	KC280	-5.8	0.0	0.0	-2.5	0.0	0.0	0.0	0.0	-3.0	0.0	0.0	-1.8	-1.3	0.0	13	0	0	0	0
6703	BWR	09:02	N251B	0.4	0.0	0.0	-4.0	0.0	0.0	0.0	4.0	2.0	0.0	0.0	-1.2	0.4	0.0	20	2	0	0	0
6704	BWR	09:13	S328SS	1.3	0.0	3.1	0.0	0.0	0.0	-2.5	0.0	0.0	0.0	-25.0	0.0	0.0	0.0	63	6	0	0	0
6705	BWR	09:20	N401P	0.4	0.0	0.7	0.0	0.0	0.0	0.0	-8.7	-3.7	0.0	0.0	-4.3	-0.4	0.0	20	0	0	0	0
6706	BWR	09:23	N204B	0.9	0.0	0.0	4.5	0.0	0.0	0.0	0.0	4.1	0.0	0.0	-4.1	-6.8	0.0	17	-4	0	0	0
6708	BWR	09:31	N204B	0.7	0.0	0.0	3.3	0.0	0.0	0.0	0.0	59.7	0.0	0.0	1.3	-9.0	0.0	23	-2	0	0	0
6811	BWR	09:48	KC280	0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-3.0	0.0	0.0	3.3	-6.3	0.0	25	-3	0	0	0
6812	BWR	09:48	N251B	0.4	0.0	0.0	-1.0	0.0	0.0	0.0	4.0	2.0	0.0	0.0	6.8	-7.6	0.0	10	-1	-30	0	0
6813	BWR	09:53	N251B	0.2	0.0	0.0	1.0	0.0	0.0	0.0	4.0	2.0	0.0	0.0	-5.2	-7.6	0.0	20	0	0	0	0
6814	BWR	09:59	N201B	1.7	0.0	0.0	5.0	0.0	0.0	0.0	-25.0	0.8	0.0	0.0	-5.0	-29.2	0.0	58	-5	0	0	0
6815	BWR	10:07	N201B	1.7	0.0	0.0	0.8	0.0	0.0	0.0	-25.0	0.8	0.0	0.0	11.7	-29.2	0.0	58	-5	0	0	0
6816	BWR	10:08	N251B	0.4	0.0	0.0	0.0	0.0	0.0	0.0	-4.0	2.0	0.0	0.0	10.8	0.4	0.0	10	-1	0	0	0

Fig. 5.6 Exceptions list for batching errors (see Fig. 4.7 for screen display).

5.6
FIELD AND LABORATORY TEST DATA INPUT

It is now time to look first at the formats available to input data and then at the mechanisms for their extraction. Two screens (Fig. 5.7(d) and (f)) have to be formatted for input data for each grade of concrete (or product code). This seems rather daunting when there may be even hundreds of such grades. However, the facility is provided to nominate any number of grades as being the same as a selected grade in most respects, and/or of inserting default entries to apply to all grades unless specifically altered. As discussed earlier, both screens are capable of recording as much or as little data as the user wishes. The desired format is elected via screens shown in Figs. 5.7(a)–(c).

Figure 5.7(a) shows the screen that records details of the grade itself. This data is optional but its careful nomination can save time in later use. The options available are:

1. A target strength which the system will use to evaluate results entered. (However, there is also a facility for automatic setting of targets according to standard deviation, etc.).
2. A grade pool which can be used to automatically group several grades for analysis.
3. A standard curing temperature and Q value, only required if using the early age analysis facility.

Numbers of specimens to be entered at nominated ages. This is neither essential nor binding if entered but, if provided, the system automatically pre-enters age and testing date as soon as casting date is entered and uses the information to produce a schedule of specimens due for test on any day. It also enables 28-day strength prediction from control and intermediate ages. Such entries can be overwritten at any time. Data at other ages may still be entered but will not give rise to a 28-day prediction.

The screen shown in Fig. 5.7(d) is far the most extensive one and records information about the sample of concrete. As noted above, some clients arrange for much of this data to be entered automatically as soon as a delivery docket number is entered on the first screen. However, if the data is to be entered manually from a field test report, it is essential that this screen has an identical order of entry to that report so that entries can be entered in sequence without jumping around the screen or the report. If this is not done, we find that frequent data entry errors are often encountered and it is necessary to spend valuable time checking data entry in this case. Fortunately, in the Windows system, having nominated which items are to appear, they can then be dragged and dropped into different positions on the screen. Note the facility for nominating default items to be automatically retained unless overwritten.

The specimen data entry screen (Fig. 5.7(f)) is relatively simple and records data for individual specimens. Again, only items selected via

Fig. 5.7 (a) Product set-up screen, (b) Docket information screen, (c) Specimen information screen.

set-up screen (Fig. 5.7(c)) appear on the specimen screen and the order can be re-arranged after selection. The system calculates strength and density if loads and dimensions are input or can accept direct entry of strength and density. Density on receipt is very important because it correlates very well with strength and is available at 24 hours or less.

On entry of earlier age tests (if of the nominated control or intermediate age) the system immediately predicts the 28-day strength, and for intermediate age, also the control age strength. These predictions are colour coded into five categories:

1. Predicted failures.
2. Predicted to pass but be below the target strength.
3. Predicted to pass and be above the target strength.
4. Predicted to exceed the target strength by a large enough margin to suggest a cement reduction.
5. Predicted to exceed the previous average strength for the grade in question by a margin which suggests that a mistake or specimen exchange may have occurred.

Results may alternatively be entered in a horizontal format if preferred (Fig. 5.7(g)). In this case the system automatically brings details of the specimens due for test to the screen ready for test results to be entered.

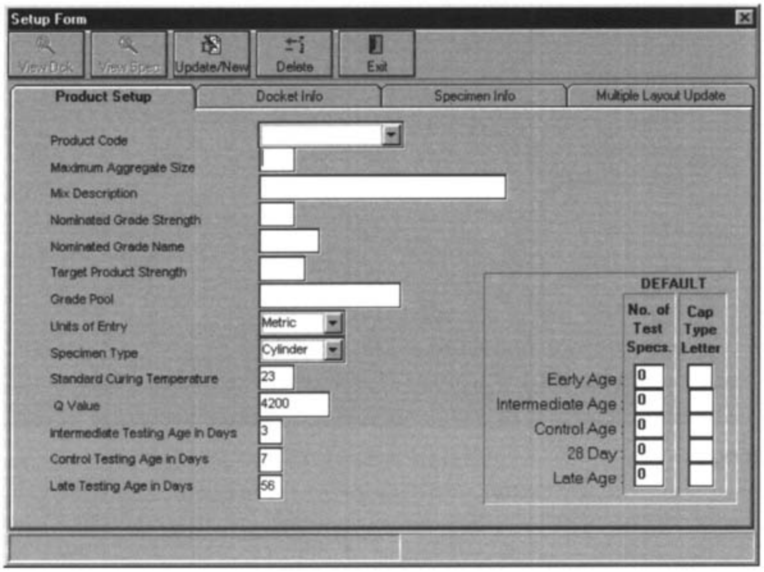


Fig. 5.7(a)

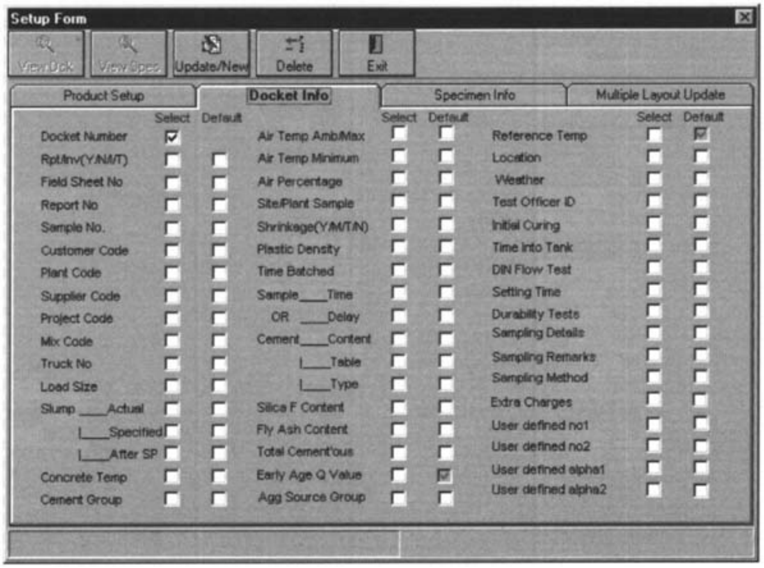


Fig. 5.7(b)

The data are stored in the same database whichever means of entry is chosen and a change in method used is possible at any time.

It will be noted that a button is provided on the docket entry screen to view 'Full Screen' (Fig. 5.7(d)). This displays all test data entered for a sample on a single line and is also colour coded. On clicking the mouse on any line of this screen the user is taken to the detailed view screen for that sample.

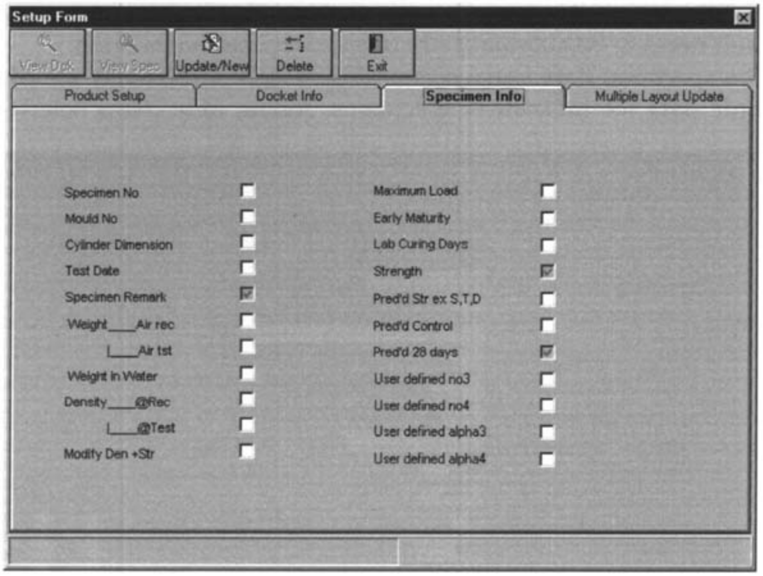


Fig. 5.7(c)

The system can equally predict control and 28-day results from tests at less than 24 hours or from heat accelerated tests but in both these cases temperature must be continuously monitored (section 12.2).

5.7
DATA RETRIEVAL AND ANALYSIS

Now to retrieval and analysis of the data entered. A first screen (Fig. 5.8) allows selection by date period, docket number range, or sample number range. Data can be restricted to that for a particular client, project, producing plant, or supplier (this last for use of the system by a major project purchasing concrete from more than one supplier). There are options to use batch plant data in the analysis or not, and to restrict batch data to only that from trucks which have been tested. Data can be restricted to a particular cement or aggregate source group. The right-hand side fields allow adaptation to suit different countries, running means of 3 or 4 and 10 or 20 (or anything else you enter), 'k' values of 1.65 for 5% below or 1.28 for 10% below.

At the top left is Product Code entry. Clicking on the arrow of this brings up a list of all product codes (Fig. 5.9) in use (there may be many hundreds in the largest organizations). The facility is provided to make these into multigrade groups. Even though this only has to be done once, it may still be an onerous task so the facility to use wildcards has been added. Depending on how carefully product codes have been chosen, this can make life much easier. Maybe product codes start with an N or an S (for normal or special, as they do in Australia). Maybe the second and third, or fourth and fifth give the grade strength. Maybe the sixth or seventh tell which are pump mixes, or which have 14 mm maximum sized aggregates. So N* will give all standard mixes, ??1 all pump mixes, ????F all mixes with fly ash, etc.

Then there are the two check boxes in the top right-hand corner. These refer to a second screen (Fig. 5.11) which offers an extensive choice.

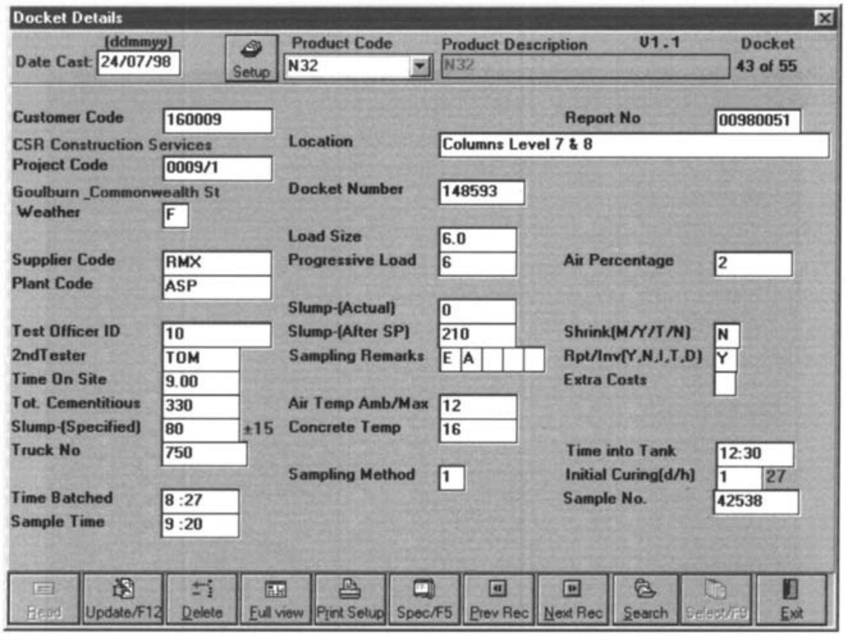


Fig. 5.7(d)

It is possible to segregate data that has any number of batch ingredient quantities above, below, or between any nominated limits; or which has given test results at any age above below or between any nominated limits.

The bottom section of this screen offers even more interesting possibilities. The average pair difference (in 28-day strength tests) of each testing officer in turn, over any selected period, can be examined, or the average difference between ordered and tested slump for each individual truck. Even the average difference between target and actual strength for each individual test specimen mould could be examined. Although this screen offers a very large range of possibilities, rarely would more than one of them be selected at a time. It is not suggested that extensive use should necessarily be made of this screen but, in the spirit of the rest of the program, if you have a use for the facility it is certainly available in a very comprehensive way.

The check box for 'Use Additional Search Criteria' on the main screen has been found to be very necessary as some clients may forget they have made entries and inadvertently use biased data in an analysis.

Similarly, when performing a restricted analysis, the running averages maintained by the system must not be updated.

Finally on the main selection screen, 'Gain Reset Date' requires explanation. The system automatically maintains an average gain for every grade of concrete for which results are entered. This enables the system to give correct predictions of 28-day strength whatever the characteristics of the particular cement or concrete mix in use. However, if a sharp change in cement characteristics (or admixture usage) takes place, it can take some time for the average to adjust to the correct value. Therefore if such a change is detected, its date should be entered in the box shown so that all results prior to that date will be excluded from the average.

PRODUCT CODE: N201

Date	Sample	Docket	Plant	No#.1	No#.2	No#.3	No#.4	Den#1	Den#2	Den#3	Den#4	Slmp
27/06/98	03621	73537	SFD	22.9	#35.6	#34.9		2461	2452	2446		85
27/06/98	03622	73539	SFD	20.6	28.3	29.0		2465	2448	2430		85
27/06/98	03623	73541	SFD	22.6	#35.5	#35.1		2474	2464	2459		80
30/06/98	01965	43170	SDN	15.9	23.5	22.7		2442	2452	2446		75
30/06/98	01966	43172	SDN	17.4	24.1	24.1		2464	2460	2471		65
30/06/98	01967	43175	SDN	18.3	24.8	25.6		2462	2467	2466		70
03/07/98	02283	34984	HAS	27.3	31.9	31.4		2467	2459	2475		65
22/07/98	50126	85249	ANG	23.1	29.2	29.4		2394	2392	2384		60
23/07/98	04878	14157	KLR	16.6	22.7	22.9		2351	2363	2358		80
23/07/98	04879	14158	KLR	16.9	23.6	23.8		2347	2355	2355		85
23/07/98	04880	14159	KLR	18.2	26.0	25.6		2363	2350	2359		80
31/07/98	04614	74882	SFD	17.7	23.1	24.5		2436	2452	2447		80
04/08/98	05193	72017	SFD	20.4	24.8	24.2		2470	2457	2474		60
04/08/98	05194	72021	SFD	18.6	23.2	23.2		2444	2461	2455		40
12/08/98	87407	15011	KLR	16.2				2312				85
12/08/98	87408	15012	KLR									90
12/08/98	87409	15013	KLR	16.9				2320				80
12/08/98	87410	15014	KLR	16.3				2326				90
12/08/98	15287	86773	COL	19.6				2421				80
12/08/98	15288	86774	COL	21.5				2411				80
14/08/98	50201	85345	ANG	20.1				2366				60
14/08/98	50202	85346	ANG	20.2				2393				65

Select Exit

Fig. 5.7(e)

5.8 MULTIGRADE, MULTIVARIABLE ANALYSIS

The first screen display after keying 'Multigrade' at the bottom left-hand corner of the selection screen is a table of all grades of concrete included in the selection (Fig. 5.10). This can also be printed in a slightly different format (Fig. 5.19).

Graphs, and especially multigrade, multivariable cusum graphs, are the most valuable feature of the system in detecting the occurrence and cause of change but the possibility exists that some grades represented by few results will not conform exactly to the general trend.

With new clients, if there has been a history of making adjustments to all mixes of plus or minus the same number of kilograms of cement, the departures may be substantially skewed, with all lower grade mixes under target (or vice versa). This will be readily shown by the colour-coded display.

There are three aspects to whether a grade is performing adequately, all of which are covered by both the screen display and the print-out: (a) whether its mean strength is on target—the current mean is displayed for 28-day results, for predicted 28-day results from control age (3 or 7 days), and for predicted 28-day results from control age specimens from samples which have not yet attained 28 days; (b) whether its characteristic strength provided (in most countries, mean minus 1.64×SD) is above the specified grade strength; (c) what actual number and percentage of failures have occurred in the grade.

Adherence to target strength is generally the most reliable criterion, especially for grades with few results. Calculated SD and number of failures both tend to be substantially affected by chance and testing error in all but the best regulated laboratories ('laboratories' in this context include field sampling personnel). It will be noticed that a 'basic SD' figure is given in the top left-hand corner of the screen (Fig. 5.10). This is derived from the average difference between successive results (as the average difference divided by 1.13). Of course, this includes a number of pairs from different grades. A second figure is therefore given which

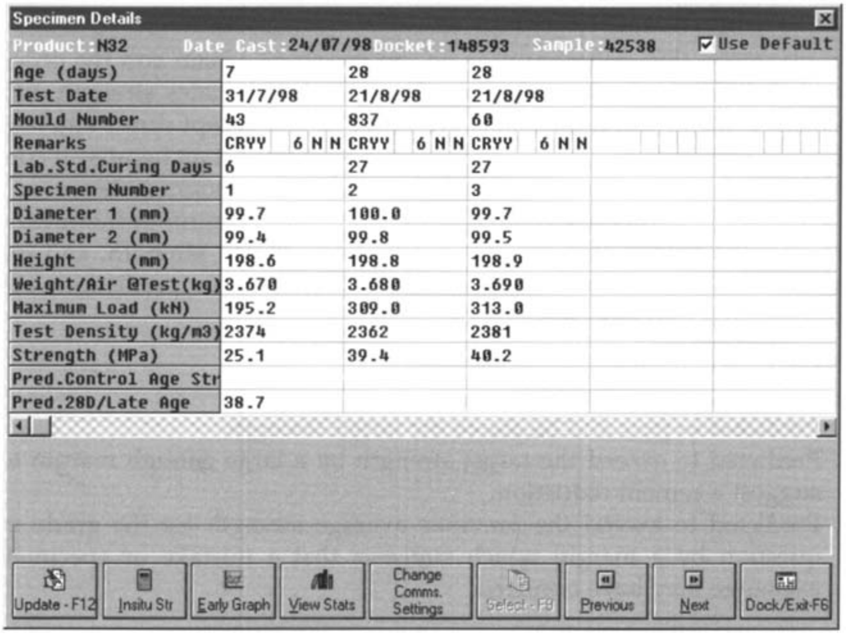
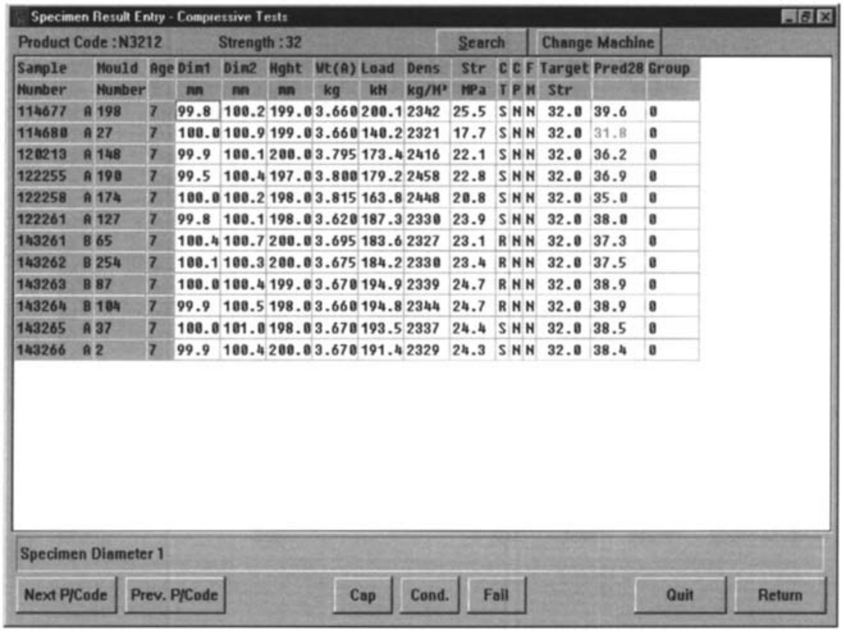


Fig. 5.7(f)

excludes differences greater than three times the initial basic SD. The SD calculated in this way is essentially that which would be experienced if there were no chages of mean strength during a period. It is conceivable that a particular grade being supplied in substantial quantities to a single project will have a lower variability than the basic SD but in general it is the lowest believable figure. On the other hand, unless there is a reason for persistent higher variability in a particular grade, SDs higher than the basic figure will generally be the result of either chance, testing error, or variations in mean strength (most frequently the latter). It is therefore likely that all concrete will actually be of acceptable strength (as opposed to necessarily meeting specification criteria) provided that the mean strength always exceeds the specified strength by at least 1.65× basic SD.

The grade/group selection screen (Fig. 5.9) can be used to set up groups of any mixes which are suspected of having higher than average variability. Examples may include those with a particular cement, pozzolan, coarse or fine aggregate, etc., or those of a particular mix type. In doing this, it should be borne in mind that if the effect causes changes in mean strength from time to time, then it will cause higher variability in a way which may not register at all on the basic variability figure.

So, multigrade cusum graphs will be the most sensitive detector of strength upturns and downturns but the displayed results table will show whether any particular grade does not follow the general trend. Mix adjustments may also be indicated by materials testing (cement and/or aggregates). Such adjustments, if accurate, may be more valuable than those based on concrete test result analysis because they may avoid rather than rectify changes in mean strength; however, they are likely to be less accurate. The aim should therefore be to make adjustments based on materials testing and then fine tune by adjustments based on concrete test data.



Sample Number	Mould Number	Age	Dim1 mm	Dim2 mm	Hght mm	Wt(A) kg	Load kN	Dens kg/M^3	Str MPa	C T	C P	F M	Target Str	Pred28	Group
114677	A 198	7	99.8	100.2	199.0	3.660	200.1	2342	25.5	S	N	N	32.0	39.6	0
114680	A 27	7	100.0	100.9	199.0	3.660	140.2	2321	17.7	S	N	N	32.0	31.8	0
120213	A 148	7	99.9	100.1	200.0	3.795	173.4	2416	22.1	S	N	N	32.0	36.2	0
122255	A 190	7	99.5	100.4	197.0	3.800	179.2	2458	22.8	S	N	N	32.0	36.9	0
122258	A 174	7	100.0	100.2	198.0	3.815	163.8	2448	20.8	S	N	N	32.0	35.0	0
122261	A 127	7	99.8	100.1	198.0	3.620	187.3	2330	23.9	S	N	N	32.0	38.0	0
143261	B 65	7	100.4	100.7	200.0	3.695	183.6	2327	23.1	R	N	N	32.0	37.3	0
143262	B 254	7	100.1	100.3	200.0	3.675	184.2	2330	23.4	R	N	N	32.0	37.5	0
143263	B 87	7	100.0	100.4	199.0	3.670	194.9	2339	24.7	R	N	N	32.0	38.9	0
143264	B 104	7	99.9	100.5	198.0	3.660	194.8	2344	24.7	R	N	N	32.0	38.9	0
143265	A 37	7	100.0	101.0	198.0	3.670	193.5	2337	24.4	S	N	N	32.0	38.5	0
143266	A 2	7	99.9	100.4	200.0	3.670	191.4	2329	24.3	S	N	N	32.0	38.4	0

Fig. 5.7(g)

Clicking on the tab entitled ‘Graphs’ at the top of the Record Selection screen (Fig. 5.8) takes the user to the Graph screen (Fig. 5.12). This is very similar to that in the first edition but now the variables shown on this screen can be selected from a list of 80+ calculated by the system.

A new feature is the capacity to move the direct plots vertically in addition to being able to expand or contract the scale.

On the right of the working (second stage) selection screen are six columns which enable up to six standard graphs to be set up. On the extreme right are the six buttons initiating actual display of the graphs. On the left, a list of the primary selection of variables appears. Between the two are scaling and location factors that enable the graph display to be adjusted so that no one variable dominates to the extent that variations in other variables are not discernable. For example, inserting a D in column 3 opposite ‘Predicted 28-day Strength’ will cause a direct plot of predicted 28-day strength to appear on the graph screen when graph 3 is selected. Inserting a C in column 4 opposite ‘Density on Receipt’ will cause a cusum plot of density to appear on clicking the graph 4 button. Entering a B will actually cause both a direct plot and a cusum of the selected variable to appear.

A maximum of eight variables can appear on a single graph. This is too many for many people to cope with but, as with several other aspects of the system, flexibility is desirable. We have found that some users like eight variables at once, enabling them to see correlations better. Others cannot cope with many variables at once (a surprising number of people are at least slightly colourblind) but have good memory retention when switching from one graph to another.

As noted, users are able to select their own combination of variables to be displayed on graphs. The facility is available to store a number of combinations as standard, but the option remains of producing a special combination of variables to examine a particular problem more closely. A typical use is to start with the maximum number of eight variables. This gives a screen which is too crowded to read fully but enables

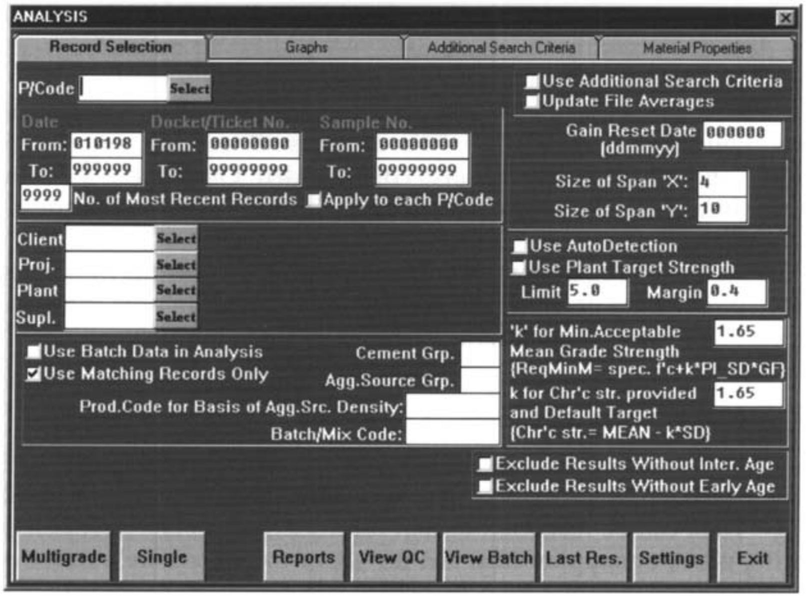


Fig. 5.8 Record selection screen.

a choice to be made of which variables appear to be most important. A smaller selection of variables can then be made to obtain a fully readable chart.

The graphing system automatically adjusts the overall scale so that the highest peaks and lowest troughs just fit on the screen, but if one variable displays ten times the variability of any other, all the others are lost in a featureless tangle along the *x* axis. On scaling the errant variable down by a factor of ten, the graph of this variable appears unchanged but all the other variables appear to have been scaled up by a factor of ten.

If the average strength, slump, density, or other results from each different mix of concrete in use is subtracted from the current results, the differences can be treated as though they were all from the same mean of the same mix.

A cusum analysis can be conducted of such differences on a multigrade, time sequence basis, i.e. no account is taken of which mix is involved, the results being entered in order of production of the concrete. If some factor such as cement quality experiences a change, all grades are likely to be affected and the multigrade cusum will show a change point. If one or more grades are not affected, this will normally be very useful in tracing the cause of the change.

Another kind of strength comparison is a direct plot of the difference between the strength and the specified strength (Fig. 5.14). This is a useful graph to plot along with the strength cusum (direct plots and cusums can appear on the same graph screen) since the latter only shows change points and does not establish whether or not the strength is still acceptable after a downturn.

The actual minus specified difference detects individual failures but is not a reliable guide to whether there is a problem with the concrete as a whole. To provide such a guide, the difference between the actual result and either the required average or the target strength is more effective (Fig. 5.15). The required average is the specified strength plus a constant times the current standard deviation. The constant depends on the permissible percentage defective. For 5% defective the figure is 1.645 (conventionally rounded to 1. 64 in the UK and 1.65 in Australia) and for 10%, as used in the USA, the figure is 1.28. The target strength

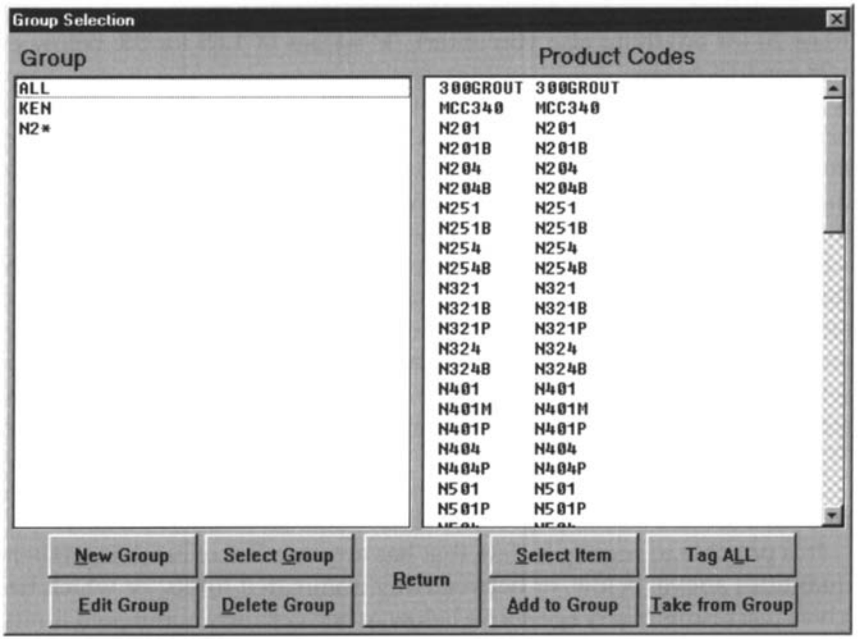


Fig. 5.9 Grade/group selection screen.

is essentially the same thing but has been preselected rather than calculated from current data. The usual requirement for satisfactory operation is that a running mean of points on this graph should stay above the zero line.

An interesting and useful aspect of the above is that the latter two graphs reveal different types of problem. If the average difference from the required mean dips below zero, there is a genuine problem with the concrete mix and a mix amendment or attention to the quality of one or more ingredient is required. However, if this curve remains acceptable but the ratio to the specified strength shows isolated dips below zero, the problem is one with individual trucks. Attention in this case is required to such items as waiting time and addition of water on site or to sticking and occasional irregular operation of cement or admixture dispensing equipment. Defective testing is also a likely cause in the latter case.

It has been continually emphasized that the control system must be directed to detecting the need for amendment of a concrete mix at the earliest possible moment. British Standard BS 5700:1984 states that a cusum chart **'will give a much more vivid illustration of any changes that are occurring…uses data more effectively, thereby giving cost savings…gives a clear indication of the location and magnitude of change points in a process.'** These properties are exactly what is required for good concrete QC.

Shilstone (1987) has reported very poor correlation of strength with slump in particular. His computerized control system is quite sophisticated in its statistical capabilities and is readily able to produce a regression analysis between any two variables. It would appear that there may be too many factors involved in concrete mixes to obtain a good correlation on simple regression over a period, although a better correlation might be anticipated from a multivariable regression. An example of the problem is that if a high slump results in better compaction of a test specimen, it may produce a strength increase, whereas if a low slump is a result of high temperature, it may produce a strength decrease. (Another explanation for the

Multigrade Analysis

Date From:010698-999999
Basic SD:2.5(2.4{-3 res.})
Print
Close
Most Recent 9999 records from each Product Code

Sort by P/Code
Sort by 28D-Targ

PRODUCT CODE	No. of Res	28D Fail No.	28D Fail %	PRED28D Fail No.	PRED28D Fail %	P28Dx0s7 Fail No.	P28Dx0s7 Fail %	CTL-AGE STR	28D STR	28D -Targ	PRED. 28D exCTL	PRED. 28D ex0s7	AVG. TARG STR	CTL-AGE SD	28 SD
2634664	3	0	0%	0	0%	0	0%	22.3	34.4	3.4	34.4			2.4	3.
2532763	12	0	0%	0	0%	0	0%	17.5	34.0	3.0	34.0			2.6	3.
2532663	2	0	0%	0	0%	0	0%	17.3	33.8	2.8	33.8			0.9	3.
3032663	1	0	0%	0	0%	0	0%	19.2	38.7	2.7	38.7			0.0	0.
3134664	2	0	0%	0	0%	0	0%	25.5	38.6	2.6	38.6			2.0	0.
3334663	29	0	0%	0	0%	0	0%	25.0	38.5	2.5	38.5			2.5	3.
2634665	8	0	0%	0	0%	0	0%	21.5	32.6	1.6	32.6		31.0	1.5	2.
3242663	1	0	0%	0	0%	0	0%	19.6	37.3	1.3	37.3		36.0	0.0	0.
2034663	1	0	0%	0	0%	0	0%	16.0	27.1	1.1	27.1		26.0	0.0	0.
3532762	5	1	20%	1	20%	0	0%	24.6	40.9	-0.1	40.9		41.0	4.8	4.
2534463	14	1	7%	1	7%	0	0%	19.2	30.6	-0.4	30.6		31.0	3.6	4.
3532962	38	2	5%	1	3%	0	0%	22.0	39.0	-2.0	39.0			2.6	2.
4042663	4	1	25%	1	25%	0	0%	25.8	43.8	-3.2	43.8			5.9	4.
3333763	2	0	0%	0	0%	0	0%	18.8	32.1	-3.9	32.1			2.1	2.
4032663	6	0	0%	0	0%	0	0%	26.6	42.4	-4.6	44.6	45.7		3.6	1.

Fig. 5.10 Multigrade analysis screen display (shown twice, scrolled left and right). The displayed table may be arranged in order of product code (=grade) or, as illustrated, in order of departure from target strength.

lack of correlation is given in section 12.1.1; this is that slump may need to be corrected for temperature, and for delay since batching, if good correlation with strength is to be obtained).

These problems are greatly minimized by the use of multivariable cusum analysis. Here correlation is not sought over a period but at an instant of change. If the strength cusum shows a change point and another variable also shows a change point at the same sample, then it is very likely that they are related. If there is a triple correlation between strength, slump and density, this will be visible on the graphical presentation.

This type of correlation has benefits beyond the early verification of strength changes. Its major use is in detecting the cause of any change, and therefore in the rectification of the change. Density provides a prime example; the heaviest ingredient of concrete is cement and the lightest are air and water. Therefore if a particular grade of concrete becomes lighter then it has either less cement, more water, more entrained air or the test specimen has been less fully compacted. Any of these changes would reduce strength. (There is also the possibility that the SG of the coarse aggregate has reduced and the author has detected an unauthorized change in the source of supply of coarse aggregate by this means.) If the problem is one of test specimen compaction, it is very likely to be accompanied by an increase in the range of test specimen densities, i.e. the difference between the highest and lowest density of specimens from the same sample of concrete. This illustrates how valuable density data can be and is the reason why the author wherever possible requires specimens to be weighed and measured on receipt at the laboratory rather than at test. If there has been no change of either slump or temperature then the gradings and silt contents of the aggregates (especially fine aggregate) would be urgently consulted for an explanation of any increase in water requirement. If available, actual batch quantity data would be consulted to check whether there has been any change in cement content (intentional or not).

When a clear change point, active over all grades of concrete, occurs without any other correlating factor, then the cement quality would be strongly suspect. The author has experienced a limited number of such

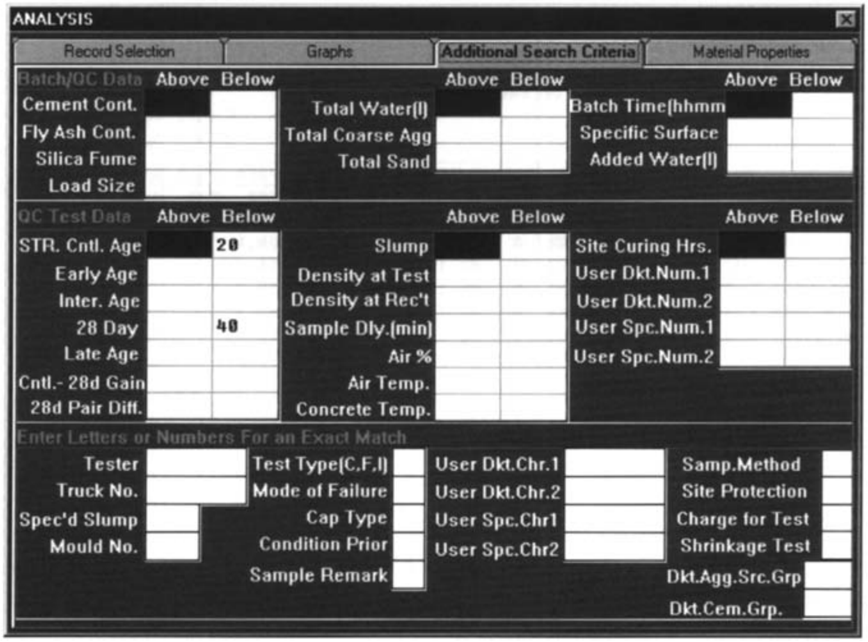


Fig. 5.11 Second screen criteria.

detections of cement quality variation, but they are a very small proportion of the number of changes due to other influences. This is in distinct contrast to the number of persons the author meets who are convinced that the main cause of variability in the concrete they produce is variation in cement quality. Such a claim tends to cause the author to be immediately suspicious that the control system in operation may not be very effective in detecting other causes of variation. However, there are some parts of the world (e.g. Singapore) where cement ground from a totally different (imported) clinker may be delivered without prior warning. The foregoing remarks obviously do not apply to such a situation.

Whether it is worthwhile for the purchaser to test cement depends on the circumstances. Important factors are the volume of usage and variability of cement from the source being used. However, if test data is made available by the cement producer, it is always worth including in the system. Figure 5.16(a)-(d) shows the graphs (cusum and direct plot) of such data kept by the author. As with other 'associated variables' a clear change point on either these graphs or the concrete strength graphs should lead to an examination of the other to see whether there is any correlation. The new Windows version of the system is able to graph such data along with strength and other QC data.

The author developed an early age check for a change in cement characteristics. The concept was that the rate of reaction of the cement can be obtained through the Conad early age analysis system by comparing the results of two test specimens at about 24 hours age, one of the specimens having been heated and the other normally cured. This would enable a correction to be made in the strength increase to be expected between an early age and 28 days. However, clients have not been interested in pursuing this, supporting the author's view that changes in cement quality, while they do occur, are not a major QC problem when an efficient analysis system is in use. A comforting finding is that if a cement reacts more slowly, it will show a larger drop at an early age and a larger subsequent gain. Therefore adverse changes will be initially over-estimated and conservatively dealt with.

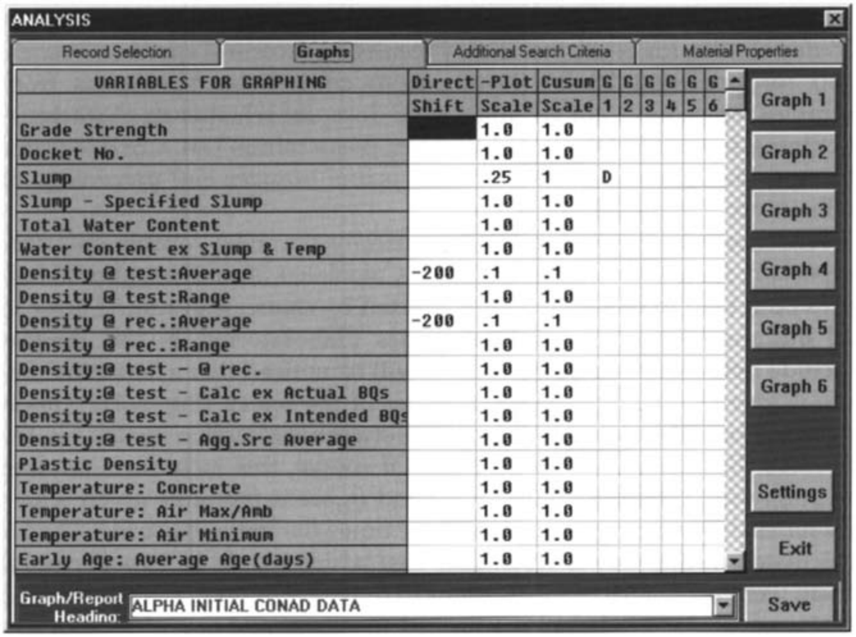


Fig. 5.12 Graph variables screen

Conversely, an upturn at an early age may not be sustained. In particular the possibility that the test specimens may have been heated should be considered. If this has happened, for example by leaving the specimens in the sun, it may have serious implications for the 28-day strength of companion specimens.

5.9 SINGLE GRADE ANALYSIS

It will be found that, in general, cusum graphs work better with a large number of points on them, say in the range of 50 to 250 or even more. On the other hand, direct plots are better with a smaller number of points, from less than 10 to no more than 100. Again, direct plots are more easily swamped by too many variables on the one graph. Many variables can initially be displayed on a cusum graph to get a quick idea of where the correlations are. Variables showing no correlation in a particular case can easily be 'weeded out' to permit a closer examination of those that do appear to correlate.

Another aspect of the cusum/direct plot comparison is that they tend to reveal different things. Essentially cusums are extremely good at picking exactly when sustained changes occur (however small) and in showing the cause of the changes through correlation. To some extent the sustained changes are shown so well **because** cusum graphs tend to cut through scatter and individual (non-sustained) variation without being greatly affected by it. This is not an unmixed advantage because there are times when individual variations merit consideration. Thus if a single truck has a very high slump, its strength becomes of particular interest. It is helpful to be able to look back along a direct plot to see what happened last time a very high slump occurred.

This can be done the day the concrete is placed so that even before any strength test result is obtained, it may be possible to estimate quite closely what the 28-day result will be. Such a use cannot be made of a cusum chart. At least in theory it may be possible to withhold permission to use a particular high-slump load

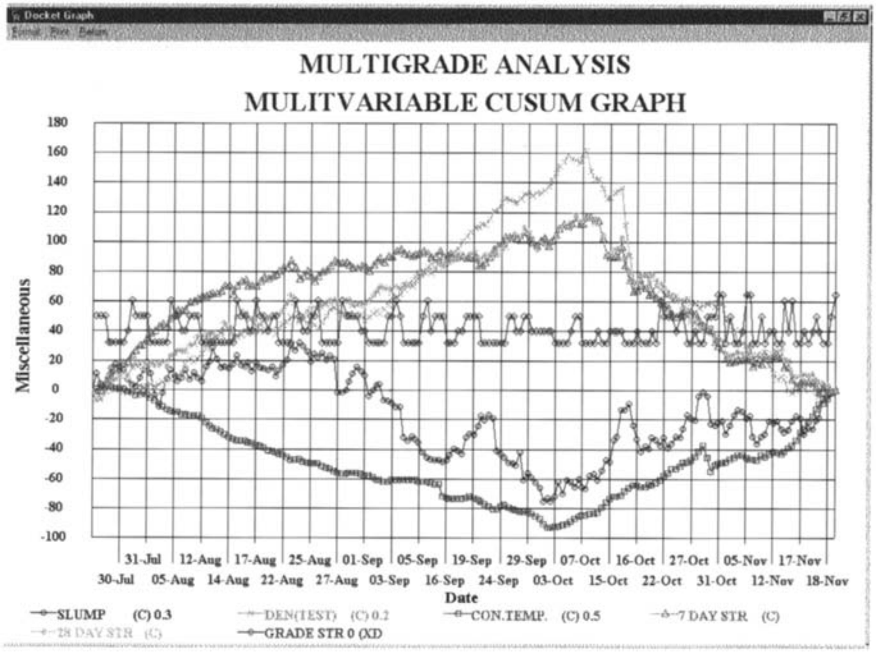


Fig. 5.13 Multigrade, multivariable cusum graph.

while such a check is done. Conversely, the control system can provide in advance a limit of slump at which strength will become marginal. Often it will be found that such a limit is much higher than that permitted under the Code or specification under which the concrete was ordered. The fact that a particular high slump is not likely to have a compressive strength below that permissible does not necessarily make such a slump acceptable. The higher slump may cause excessive shrinkage, segregation or bleeding, or delay readiness for trowelling. However, having knowledge of whether or not a strength deficiency will result is very useful when considering whether to reject a particular overslump truck of concrete. Rejection may be justified in terms of the specification but it may not be desirable to implement this. Rejecting a truck may result in a pump blockage or a cold joint or cause an undesirably late finish.

Some suggested combinations of variables to use as defaults or standard settings are:

1. Cusums of 7 and 28-day strength, slump, concrete temperature, density, 7 to 28-day strength gain, strength grade and plant. The last two items are not shown as cusums but just a direct plot of specified strength and docket number divided by a suitable constant; the latter simply produces a straight line across the screen unless a delivery is from a different plant, in which case a ‘blip’ appears drawing attention to the fact.

   Remember 28-day strength and 7 to 28-day gain are three to four weeks after the other data and so are generally too late to help directly in control action. However, they are needed to show the significance of the earlier results in the eventual strength attained. They also show the past effects of variations of slump, temperature and density and so help to judge the likely effect of current variations.
2. Cusums of 7 and 28-day strength, density, density range, pair difference and 7 to 28-day strength range. These are a slightly different set of variables concentrating more on testing quality. Density

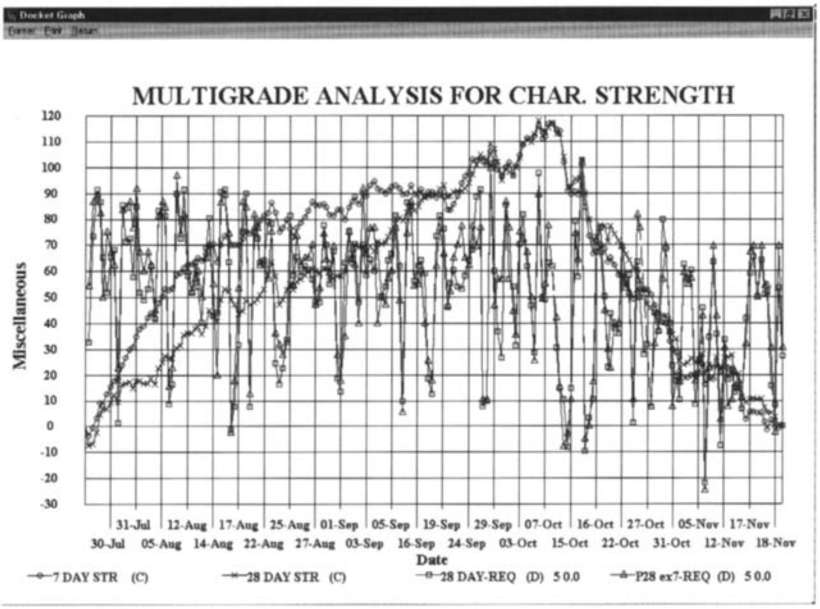


Fig. 5.14 Test strength minus specified strength

range (highest minus lowest from the one sample) gives the earliest possible warning of the specimen casting becoming slipshod.

3. Direct plots of 7-day strength (note that the multigrade technique is not useful for direct plots), predicted and actual 28-day strength, slump, temperature, density and plant. For a direct plot of density in metric units (where a typical figure is 2300 to 2400 kg/m^3) what is plotted is density minus 2000 which is then divided by 10. This apparently complex figure is actually very easy to read from a graph (i.e. '35' means 2350) and gives a vertical location and range of variation on the graph typically quite similar to strength.
4. Direct plots of 7-day strength, predicted and actual 28-day strength, 7 to 28-day gain, pair differences and density range. Again testing quality is more under scrutiny in this selection.

These combinations are for a typical sample of one early age and two 28-day specimens. They would be amended for other combinations.

5.10 MATERIAL PROPERTIES

The last tab on the Product Selection screen is entitled Material Properties (Fig. 5.17). This enables plotting of any selected data on material properties, such as sand specific surface or cement C_3S content on the same screen as the concrete test data. This can be very valuable when looking for causes but is a little more difficult to arrange than the other variables. This is because material properties are simply dated records and are not associated with a docket number (which is the *x* axis variable in all other graphing). The program therefore has to read the date attached to the docket number and sort the materials database for the property in question until it finds the last entry prior to the concrete production date. There are likely to be several

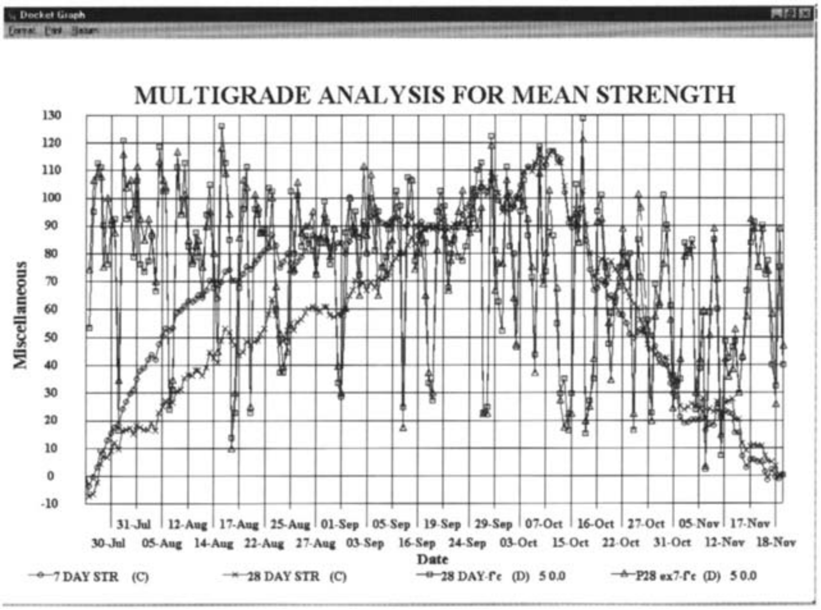


Fig. 5.15 Test strength minus required average strength

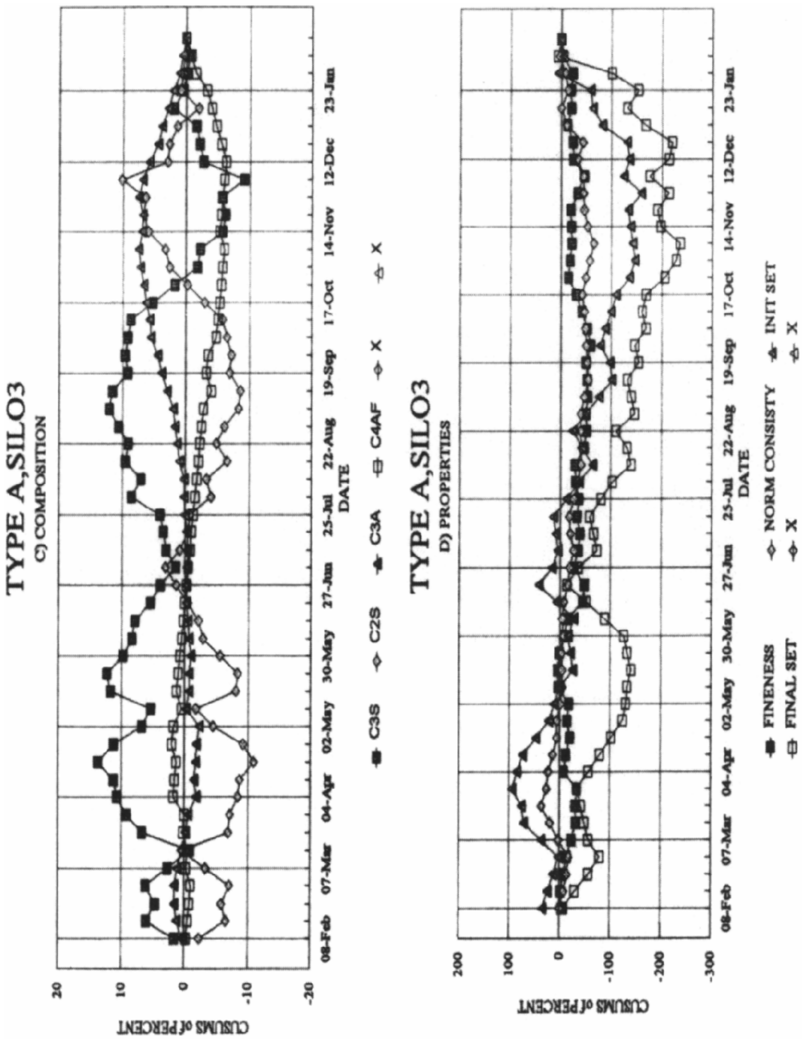
(probably many) concrete test results to each entry in the materials database so the graphs of material properties are likely to contain many horizontal straight line sections.

5.11
TEST DETAILS AND CERTIFICATES

It is desirable that the practice of providing individual test results to supervizing authorities should be discouraged as much as possible. Those receiving them tend to have an unjustified faith in their exact numerical accuracy, to allow them to accumulate without inspection from time to time, and to have little or no idea how to interpret early age results (which are the ones actually worth inspection). For many years the author's consulting engineer clients have preferred to get their results in the form of a condensed overall analysis with their attention specially drawn to any individual result requiring it. It is quite unrealistic to expect a busy structural engineer to make effective use of a large pile of individual test certificates unless necessitated by some significant problem. (This matter is further discussed in Chapter 11). Of course where required, it is quite simple to arrange to print out results in almost any desired format. It is the format of the internal database which must be common to all users if problems with upgrades are to be avoided.

5.12
STATISTICAL TABLE

The QC system should not be thought of as operating on printed tables and graphs. An elaborate monthly report may be worthwhile for record purposes and to keep all concerned in touch with what is happening. However, the actual task of control requires action to be taken when necessary on the



basis of graphs and tabular data viewed on screen within a few hours if not minutes of the test being done.

Control action, to be of real value, has to be taken immediately. Nevertheless a look back over performance in previous months has its own value in showing whether progress is being made or standards are slipping. Items such as the kind of change experienced from winter to summer and the relative variability of the different strength grades may have little influence on whether or not to increase cement content by 10 kg tomorrow but they assist those operating the system to gain a better understanding of the situation.

Having described such information as of substantial interest rather than being essential for immediate use, it may be questioned how much time should be spent on its compilation. Fortunately the question does

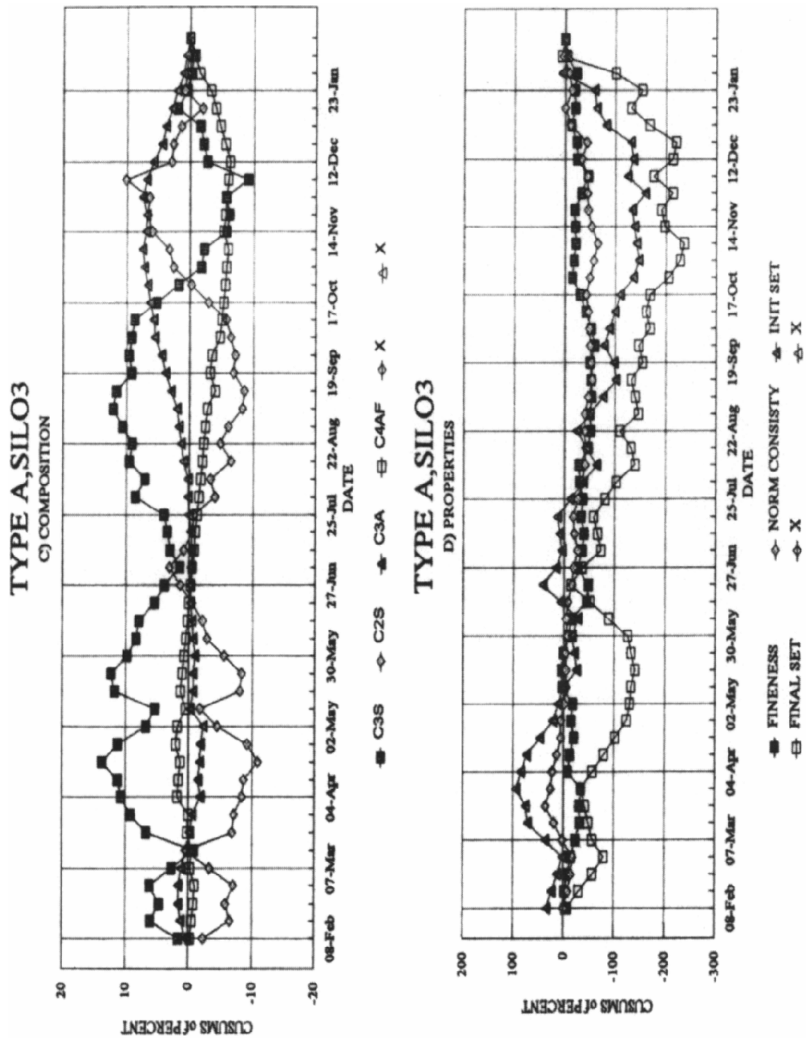


Fig. 5.16 Cement data control graphs.

not arise since the table in Fig. 5.18 is produced in about two minutes for the effort of pressing a very few keys. Typing the desired heading for the table is the most onerous task involved.

Such tables (Figs 5.18 and 5.19) enable informed discussion on such matters as:

1. Whether standard deviation or coefficient of variation is the better measure of variability.
2. The extent to which higher strength grades show a greater 7 to 28-day gain.
3. Whether statistical analysis of a small number of results (which will undoubtedly occur in some grades) is of much value.

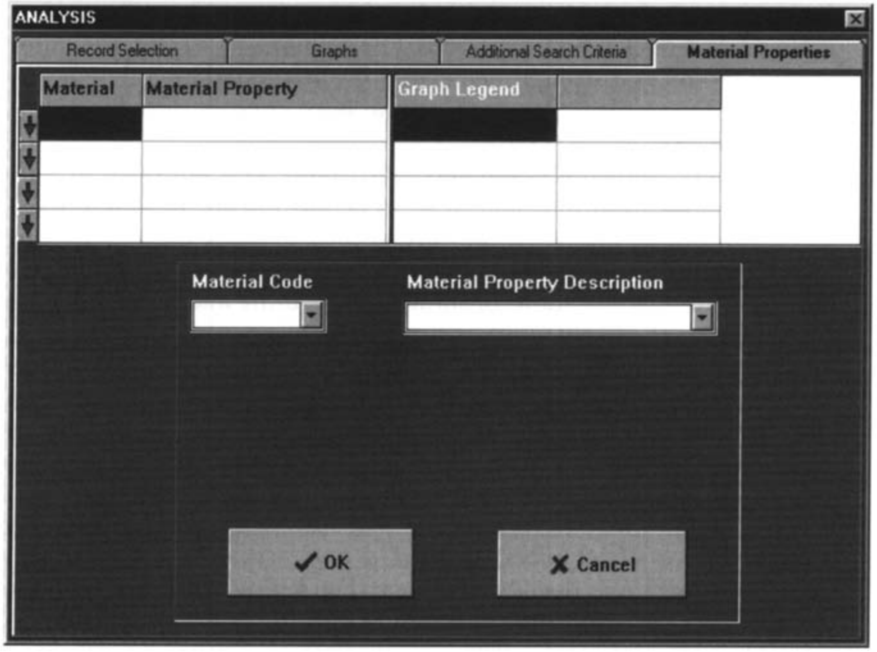


Fig. 5.17 Selection of materials properties for graphing.

5.13 INTEGRATION OF BATCHING DATA WITH CONCRETE TEST DATA

If the 'Use Batch Data in Analysis' box is checked on the record selection screen (Fig. 5.8), the program also includes actual batch quantities and aggregate gradings. If 'Use Matching Records Only' is checked, only batch data for which there is also test data is used. Since water and cementitious material quantities are available, assuming the as batched data retrieval system is in use and available to the system operator, a predicted strength is available by calculation for comparison with the actual test data. There is also sufficient data to calculate a theoretical water requirement to compare with the figure obtained from the batch plant.

If the actual strength line follows the calculated strength line, then the cause of every variation of strength is known. The points at which calculated and actual strength diverge are likely to be clearly defined and provide a good starting point for considering which factors in the calculation may be inaccurate or what property (such as cement quality) is not covered by the calculation formula. For example it is quite revealing to note when actual and calculated strengths diverge, whether actual and calculated water contents also diverge.

The **density** of concrete is precisely calculable given SG values and batch quantities. Measured density on a single specimen is not usually very reliable but the average density of a set of three or more specimens from the same sample of concrete is fairly reliable. If the latter values are cusum analysed, even quite small changes in average density are accurately detectable; e.g. in Australia, NATA requires individual density values to be rounded to the nearest 20 kg/m (see section 11.7 on rounding and when and why this should **not** be done) yet even so a change in average density of less than 10 kg/m^3 can be clearly seen on a cusum.

A change of 10 litres per cubic metre in water content (which is likely to cause a strength change of 3 to 5 MPa) will cause a density change of about 14 kg/m since each litre of water weighs 1 kg and displaces a

STATISTICS REPORT FORBOOK : ;000000 – 999999 Page 1 of 1

		SLUMP					CONCRETE TEMPERATURE					CONTROL AGE STRENGTH					28 Day STRENGTH					CONTROL – 28D GAIN					DENSITY					f'c		
P/CODE	MONTH	No	MEAN	SD	MAX	MIN	No	MEAN	SD	MAX	MIN	No	MEAN	SD	MAX	MIN	No	MEAN	SD	MAX	MIN	No	MEAN	SD	MAX	MIN	No	MEAN	SD	MAX	MIN	SD	exCntl	ACT
N201	Dec-95	23	87	6	95	75	23	17	1.9	21	14	23	16.1	1.9	21.0	13.0	23	22.9	2.1	28.3	20.0	23	6.8	2.3	12.3	3.5	23	2350	40	2433	2287	2.1	19.5	19.5
N201	Jan-96	17	79	8	90	65	17	22	2.6	24	16	14	16.0	2.7	20.5	11.5	11	25.6	1.8	29.0	22.5	8	7.6	3.4	12.5	4.0	17	2364	39	2420	2320	2.5	19.5	21.5
N201	Feb-96	21	83	9	95	70	21	18	2.7	23	15	14	18.8	2.3	23.5	15.0	0	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	14	2331	17	2360	2300	2.5	23.8	0.0
N201	pool	61	83	8	95	65	61	19	2.9	24	14	51	16.8	2.5	23.5	11.5	34	23.8	2.4	29.0	20.0	31	7.0	2.6	12.5	3.5	54	2350	37	2433	2287	2.4	19.9	19.6
N201B	Dec-95	128	83	9	110	50	128	18	1.6	22	14	120	15.5	2.0	22.5	9.9	128	25.4	2.1	30.5	20.3	120	9.9	2.3	15.8	3.8	128	2362	45	2447	2287	2.0	22.2	22.2
N201B	Jan-96	59	84	8	95	70	59	22	3.1	25	15	54	14.6	1.5	18.0	9.9	59	26.1	2.4	32.3	21.3	54	11.7	2.7	18.0	5.8	59	2352	45	2447	2287	1.5	23.8	23.6
N201B	Feb-96	98	82	9	100	60	98	21	3.2	27	14	62	14.7	1.5	18.0	11.5	0	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	62	2376	47	2460	2300	2.5	20.8	0.0
N201B	pool	285	83	9	110	0	285	20	3.1	27	0	236	15.1	1.8	22.5	0.0	187	25.6	2.2	32.3	0.0	174	10.4	2.6	18.0	0.0	249	2363	46	2460	0	2.2	21.9	22.6
N251	Dec-95	26	81	13	110	60	26	20	2.8	26	16	22	22.2	2.8	26.5	17.5	26	28.9	2.8	34.3	25.0	22	6.4	1.8	8.8	0.8	26	2379	49	2487	2300	2.8	24.0	24.3
N251	Jan-96	31	78	10	100	60	31	23	2.3	27	18	29	20.4	2.2	23.0	15.0	31	27.6	2.5	33.0	23.5	29	7.1	1.7	10.3	3.5	31	2326	29	2400	2273	2.2	23.8	23.9
N251	Feb-96	74	82	12	160	55	74	21	3.1	29	15	70	21.6	2.8	28.5	16.0	0	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	70	2349	38	2460	2280	2.3	23.9	0.0
N251	pool	131	81	12	160	0	131	21	3.1	29	0	121	21.4	2.7	28.5	0.0	57	28.2	2.7	34.3	0.0	51	6.8	1.7	10.3	0.0	127	2349	42	2487	0	2.7	23.8	23.7
N251B	Dec-95	43	83	9	95	65	43	19	2.2	24	15	43	18.5	1.5	22.5	16.0	43	30.6	3.1	37.0	25.3	43	12.1	2.6	17.5	7.8	43	2360	50	2460	2260	1.5	28.1	28.1
N251B	Jan-96	50	79	10	110	60	50	21	3.0	26	16	47	18.7	2.0	29.5	17.0	50	30.0	2.9	37.0	25.3	47	11.3	3.0	16.8	-1.8	50	2347	46	2447	2267	2.0	26.7	26.6
N251B	Feb-96	76	82	10	110	65	76	21	3.2	30	15	62	18.3	2.3	23.0	12.0	0	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	62	2350	49	2460	2280	3.1	27.4	0.0
N251B	pool	169	81	10	110	0	169	20	3.0	30	0	152	18.4	2.0	29.5	0.0	93	30.3	3.0	37.0	0.0	90	11.7	2.8	17.5	0.0	155	2352	48	2460	0	3.0	25.4	26.9

Fig. 5.18 Monthly statistical report.

litre of concrete weighing about 2.4 kg. Air content also affects both strength and density. One per cent of extra entrained air produces a strength loss of around 5% or say 1 to 4 MPa and a density reduction of 1%,

[redacted] : [redacted]
[redacted] PLANT

4316 010698-999999.Most Recent 9999 Records. Page 1

Basic S.D. = 2.5 (adjusted = 2.4) (* = used)

PRODUCT CODE	No of Res.	SLUMP MEAN	SLUMP TARG	CONC TEMP MEAN	28D Fail. No.	28D Fail. %	P28D ex7 Failures No.	P28D ex7 Failures %	P28D exOs7 Failures No.	P28D exOs7 Failures %	MEAN STRENGTHS ACTUAL 7-DAY	MEAN STRENGTHS ACTUAL 28D	PREDICTED 28D ex7D exOS7D	AVG TARG	STANDARD DEVIAT'N 7D	STANDARD DEVIAT'N 28D	ACT. 28D -TARG	28D PAIR DIFF	7-28 Day GAIN MEAN	7-28 Day GAIN MAX	7-28 Day GAIN MIN	28D DENSITY AT TEST MEAN	28D DENSITY AT TEST RNG	28D DENSITY AT TEST MAX	28D DENSITY AT TEST MIN	28 DAY CHAR. STR.	AVG. L. 4 28D
3032763	4	101	75	21	1	25%	0	0%	0	0%	19.5(4)	48.0(4)	48.0	36.0	3.0	27.3	12.0	3.9	28.4	66.4	13.4	2350	21	2362	2336	43.8*	48.0
2532263	1	70	75	22	0	0%	0	0%	0	0%	18.0(1)	35.9(1)	35.9	31.0	0.0	0.0	4.9	1.0	17.9	17.9	17.9	2305	13	2305	2305	31.7*	0.0
3034663	2	70	75	20	0	0%	0	0%	0	0%	28.4(2)	40.8(2)	40.8	36.0	2.0	2.1	4.8	2.1	12.4	12.4	12.4	2384	21	2403	2366	36.7*	0.0
3032463	6	63	75	22	0	0%	0	0%	0	0%	22.4(6)	40.6(6)	40.6	36.0	3.2	2.1	4.6	1.7	18.2	21.5	15.5	2376	20	2393	2346	36.5*	40.4
3034463	5	64	75	20	0	0%	0	0%	0	0%	25.1(5)	39.7(5)	39.7	36.0	4.4	3.4	3.7	1.8	14.6	20.8	11.5	2376	26	2393	2355	35.6*	39.3
2534663	5	75	75	20	0	0%	0	0%	0	0%	22.6(5)	34.5(5)	34.5	31.0	2.0	2.6	3.5	0.9	11.8	13.4	9.4	2344	24	2355	2332	30.3*	34.3
2634664	3	112	100	21	0	0%	0	0%	0	0%	22.3(3)	34.4(3)	34.4	31.0	2.4	3.6	3.4	1.4	12.1	13.4	10.5	2319	18	2328	2307	30.2*	0.0
2532763	12	78	75	19	0	0%	0	0%	0	0%	17.5(12)	34.0(12)	34.0	31.0	2.6	3.9	3.0	2.2	16.6	19.4	13.4	2385	23	2421	2356	28.3*	34.0
2532663	2	83	75	19	0	0%	0	0%	0	0%	17.3(2)	33.8(2)	33.8	31.0	0.9	3.0	2.8	0.8	16.4	17.9	14.9	2364	31	2368	2360	29.6*	0.0
3032663	1	70	75	21	0	0%	0	0%	0	0%	19.2(1)	38.7(1)	38.7	36.0	0.0	0.0	2.7	0.5	19.5	19.5	19.5	2375	20	2375	2375	34.5*	0.0
3134664	2	105	100	22	0	0%	0	0%	0	0%	25.5(2)	38.6(2)	38.6	36.0	2.0	0.6	2.6	1.8	13.1	14.9	11.2	2333	31	2337	2328	34.5*	0.0
3334663	29	87	75	19	0	0%	0	0%	0	0%	25.0(29)	38.5(29)	38.5	36.0	2.5	3.4	2.5	2.5	13.5	16.6	9.4	2351	22	2399	2319	32.9	38.2
2634665	8	114	125	21	0	0%	0	0%	0	0%	21.5(8)	32.6(8)	32.6	31.0	1.5	2.2	1.6	1.9	11.1	12.9	8.7	2338	24	2369	2320	28.4*	32.0
3242663	1	60	75	21	0	0%	0	0%	0	0%	19.6(1)	37.3(1)	37.3	36.0	0.0	0.0	1.3	0.7	17.6	17.6	17.6	2363	20	2363	2363	33.1*	0.0
2034663	1	75	75	22	0	0%	0	0%	0	0%	16.0(1)	27.1(1)	27.1	26.0	0.0	0.0	1.1	1.6	11.1	11.1	11.1	2356	26	2356	2356	23.0*	0.0
3532762	5	68	50	22	1	20%	1	20%	0	0%	24.6(5)	40.9(5)	40.9	41.0	4.8	4.5	-0.1	3.7	16.4	19.6	13.0	2394	15	2410	2379	36.8*	41.4
2534463	14	90	75	21	1	7%	1	7%	0	0%	19.2(14)	30.6(14)	30.6	31.0	3.6	4.1	-0.4	1.7	11.5	15.1	9.6	2345	24	2399	2315	24.8*	30.5
3532962	38	67	50	22	2	5%	1	3%	0	0%	22.0(36)	39.0(37)	39.0	41.0	2.6	2.8	-2.0	1.9	17.1	19.8	14.5	2385	27	2448	2313	34.5	39.1
4042663	4	81	75	22	1	25%	1	25%	0	0%	25.8(4)	43.8(4)	43.8	47.0	5.9	4.4	-3.2	1.5	18.1	19.7	15.5	2344	25	2374	2306	39.7*	43.8

Fig. 5.19 Multigrade statistical analysis table.

i.e. about 24 kg. A cement increase of 10 kg occupies 10/3.15=3.17 litres and so displaces 3.17×2.4=7.6 kg of concrete, increasing density by 10–7.6 = 2.4 kg/m^3, and increasing strength by 1 to 2 MPa. So, depending on which of the above events caused it, a density reduction of 10 kg/m^3 (0.4%) will correspond to a strength loss of between 0.5 and 8 MPa. The higher losses occur with higher strength concrete and are highest if the density change is caused by reduced cement content, average if caused by increased water and least if caused by increased air content.

In Australian practice, air content is rarely measured on a routine basis. Measurement of air content is naturally more common in those other parts of the world where air entrainment is vital to provide frost resistance and not merely of interest for its effects on water requirement, workability and strength. Fresh density measurement is also relatively rare in Australia, but cylinder density on receipt at the laboratory at about 24 hours age is very close to the same figure. Any difference is in any case of little consequence since it is a **change** in density which is important and any change in fresh density will certainly be reflected in a change in hardened specimen density. It is satisfactory to take a very occasional (say, once per month) air content reading by pressure meter, and to monitor test specimen density, taking further air meter readings only if a distinct change in specimen density is detected and is otherwise unexplained.

In the rare event that it becomes desirable to establish the air content of a hardened concrete specimen, this can be done with reasonable accuracy by cutting and polishing a cross-section of the concrete and measuring the air bubbles using a microscope.

It will be apparent from the foregoing discussion that the assumption that a substantial proportion of the variability of concrete is random, with no apparent cause, is no longer very useful. Certainly the strength of an individual test specimen is subject to such variability, and is also affected by testing procedures which are frequently less than perfect. However, by the time a number of specimens from each sample of concrete (albeit tested at different ages) have been considered together and combined with those of adjacent samples and with data other than strength in cusum or similar analysis, there is likely to be relatively little left to ascribe to pure chance. It is important not to over-react to single low results but it also important to seek a cause for them.

5.14 EVALUATION OF RELATIVE EFFICIENCY OF MIXES

This section of the first edition has been left unchanged because it is still true and will be very helpful to anyone setting up their own graphical analysis. However, in the Conad system it has now been replaced by the much more powerful ‘Benchmark’ system (Fig. 3.15).

A great deal of valuable information can be obtained by a comparison of a whole range of mixes produced largely from the same materials and in the same plant (Fig. 5.20).

At the most obvious level the following parameters can be graphed by simply arranging them in order of increasing strength grade along the *x* axis:

1. Strength in MPa (or N/mm^2) per 100 kg of cementitious material (or psi per Ib).
2. Water content.
3. Density.

If the sequence includes not only different strength grades but also pumped and non-pumped mixes, special high slump mixes, mixes with small coarse aggregate, spray mixes, mixes with different cementitious materials or admixtures, etc., the graph/lines are likely to sawtooth across the page showing the effects of differing water demand and other influences.

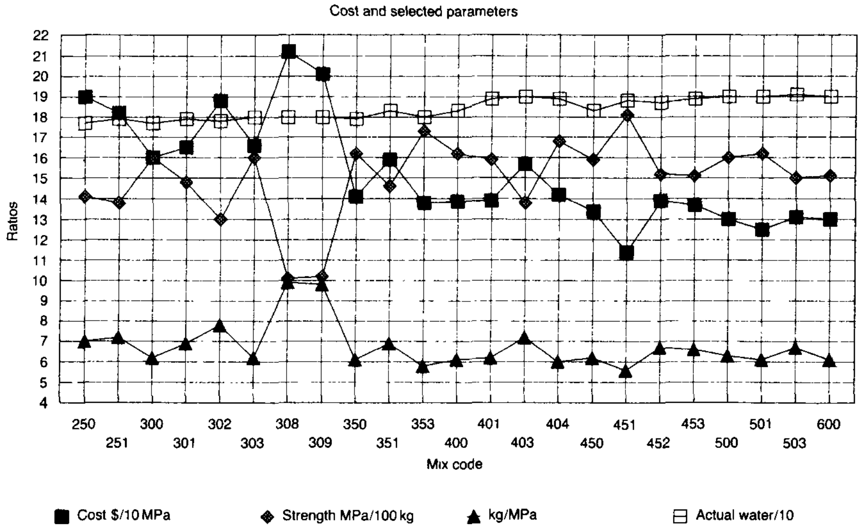


Fig. 5.20 Relative efficiency of range of mixes.

More sophisticated graphs of strength and water factors can be drawn, i.e. graphs showing the ratio of actual to calculated water content and strength. These will reveal any deficiencies or inadequacies in the calculation basis, e.g. if specific surface, as modified by the author, predicts that high sand contents will have a greater effect on water demand than actually occurs, this will stand out clearly when comparing a range of pump and non-pump mixes

Badly graded mixes will have an increased water requirement. For example, mixes containing an excessive proportion of middle size aggregate will suffer from particle interference, and this will result in extra water demand.

A graph of strength per 100 kg of cementitious material is of interest. It might be thought that this would show clearly when the point of diminishing returns from additional cement content was reached. This may actually be the case if the operator is unsophisticated, but it is more likely that different cementitious or pozzolanic materials, or more expensive admixtures, will have been used to avoid reaching this point. Best of all, a graph can be drawn of the actual strength of each grade divided by the cost of its ingredients (e.g. cost in dollars per cubic metre per 10 MPa of strength). Such a graph clearly shows the relative merits of different ways of providing the same properties if they are plotted as different grades. For example the costs with and without fly ash, blast furnace slag, silica fume, or water reducing admixtures, can be compared.

This type of comparison may be a preferable replacement for laboratory trial mixes in the evaluation of new (i.e. alternative) admixtures. One or two trucks per day of a particular grade would use the new admixture and these trucks would always be tested. After about five weeks (when the first week's 28-day results would be available and would yield the prediction factors required to evaluate the whole five weeks' mixes based on early age results) a far more accurate and reliable comparison would be available than any lab trial could provide.

It should be realized that the relativities between two mixes in a laboratory trial are not necessarily the same as in production. Differences in mixing can alter the amount of air entrained, the completeness and rapidity of admixture dispersion, etc. As one example, a laboratory trial will usually show a substantial advantage accruing by delaying the addition of a water reducing retarder for, say, 5 minutes after the start of mixing. In the first place it would be difficult to do this in a ready mix situation but in the second there would be a delay involved in the admixture (assuming it to be squirted in undiluted, not dissolved in the mixing water) dispersing through the truck. So the production situation would not be reproduced by **either** delaying or not delaying addition in the laboratory situation.

5.15
EARLY AGE TESTING

The use of Arrhenius' equation and the Conad 'Earliest' computer program to monitor early age strength development in the actual structure for such purposes as early stripping, prestressing, lifting of precast units is dealt with in detail in Chapter 12. What concerns us here is the use of these techniques for the purpose of earlier mix adjustment.

It is important to keep in mind that any increase in scatter of the results delays the point at which a change can be detected or confirmed. For example, it is possible that if 24-hour testing gave a very high scatter, then 10 or even 15 results might be obtained after a change point before it was considered sufficiently confirmed to act on it. If 7-day results were subject to a much lower scatter, it is possible that such a change point might be detected from only three or four results. Depending on the frequency of sampling, it is conceivable that the downturn could be confirmed at an earlier date by 7-day testing than by 24-hour testing. It is not suggested that this is likely, but it does emphasize the point that there is little or no advantage to be gained from early age testing if it yields results which are substantially less reliable than normal 7-day tests (or even more so with 3-day testing in tropical climates).

However, use of the system since publication of the first edition has shown that prediction of 28-day strength to an accuracy of better than ± 1 MPa is possible from early age results as low as 2 MPa. Even the author is surprised at this and it is not suggested that it is easy or can be taken for granted. Obviously it requires that the early age test itself is to an accuracy of the order of ± 0.1 MPa and many laboratories are not capable of this. It also requires that the temperature of the test specimen be accurately monitored to give a true value of its equivalent age.

5.16
PREDICTION OF 28-DAY STRENGTH

The previous section has considered how vario|us concretes gain strength under various conditions. While this obviously has a bearing on the prediction of 28-day strength from early age strength; it is by no means the only factor.

Firstly, the purpose of making the prediction must be clear. The answer is that the early age tests are required to detect the first sign of change in 28-day strength, to forewarn us of any potential failures, and to adjust the mix proportions (i.e. cementitious material content) to restore the previous 28-day strength.

Early Testing Age may be considered in three time periods after casting:

1. **7 Days**

   Specimens (cubes or cylinders) are normally cast and left on the construction site, covered but probably not insulated, until collection within 24 hours. They are then stored for 6 days in a thermostatically controlled fog room or water bath. The attractions of a 7-day test are:

(a) Specimens are tested on the same day of the week as they are cast so a 5 or 6-day working week causes fewer problems (although collection can still be a problem) and it is easy to keep track of the specimens (e.g. separate tanks or racks can be labelled with the days of the week).
(b) Assuming freezing and 'cooking' in hot sunshine are both avoided, the 7-day strength will not be greatly affected by the thermal history of the first 24 hours or the time of day on which casting and testing take place.
(c) The 7-day strength will be a large proportion (commonly 60 to 90%) of the 28-day strength. This means that inaccuracies in the predicted gain to 28 days will be much smaller in proportion to the 28-day strength than they are in proportion to the gain itself. It also means that the 'early' specimens will be on a similarly sensitive part of the testing machine range.

2. **2 or 3 Days**

Testing at 2 or 3 days may be quite satisfactory where testing laboratories operate 7 days a week and one of the following conditions applies:

(a) Conditions are tropical with little temperature variation from day to day and not very much from day to night, or
(b) Specimens are stored in well insulated containers for the first 24 hours (until collection), or
(c) Fresh specimens are taken to the laboratory so that even the first night is spent under controlled conditions, or
(d) Production is of precast concrete with specimens kept in stable factory conditions.

The author has used 3 days as the standard test age for all his work in SE Asia with no more problems than using 7 days in Australia.

3. **12 to 30 Hours**

At ages of less than 2 days, it is not enough to know the age of the specimens. It is also necessary to take into account their thermal history. The subject is separately dealt with in Chapter 12 but, providing the thermal history up to the point of a strength test is known, the testing can be at any age at which strength is sufficient to register a meaningful test strength (2 MPa is a suggested value).

Generally three factors are involved in the prediction of 28-day strength:

1. The normal average strength gain from the test age to 28 days for concrete of the particular constituents involved.
2. Factors which alter the relative strength at the early age and 28 days, e.g. variable curing conditions.
3. Testing error, which may cause an incorrect strength to be registered at the early age. Testing error in the 28-day strength itself will affect the **apparent** accuracy of the prediction and, by producing a false gain figure, may affect the accuracy of future predictions.

The normal average gain will vary in different grades of concrete using the same materials as well as varying widely with the different cements available in different parts of the world. Higher strength grades will show a higher absolute gain but a smaller percentage gain than the lower strength grades. A mean strength gain from 7 to 28 days for all grades up to 50 MPa might be 10 MPa, and this may vary by up to 20 or 25% between say 20 and 50 MPa grades, giving limits of, say, 7 to 13 MPa. These limits would not necessarily apply to special cements and pozzolanic materials.

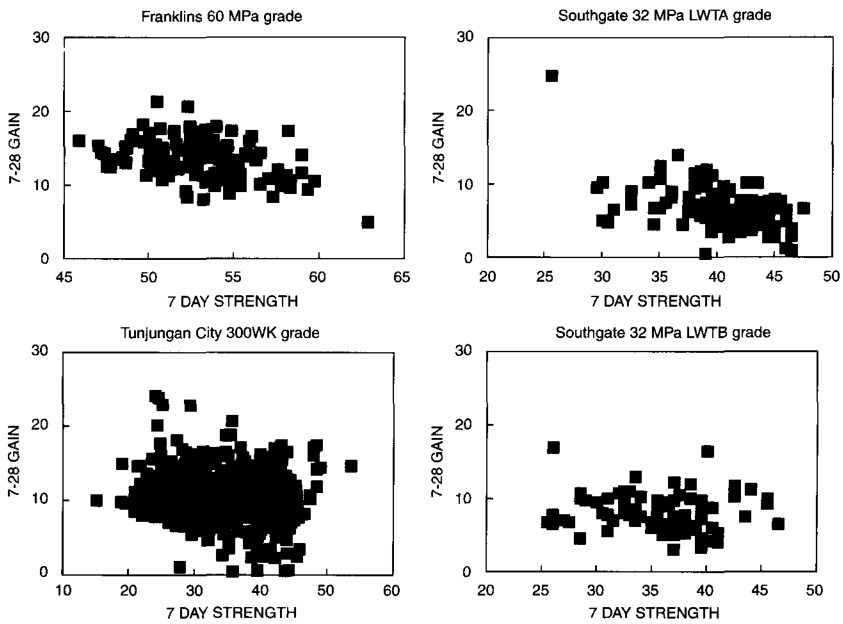


Fig. 5.21 Pattern of strength gain from 7 to 28 days.

The 7 to 28-day gain registered by individual samples in a single grade may show variations from under 5 to over 20 MPa. It is apparent that very little of this can be due to genuinely higher strength concrete showing a larger gain.

We can theorize about what gain should be anticipated from a given 7-day (or 3-day) result or we can do what the doyen of QC, J.M. Juran suggested and 'ask the process'. The author has plotted many graphs like Fig. 5.21 with 7-day strength as abscissa and 28-day minus 7-day strength (i.e. gain) as ordinate. Almost invariably they show an approximately horizontal line with substantial scatter and a tendency for the **lower** 7-day results to show a larger gain. The explanation for this is assumed to be that the very lowest 7-day results are so partly because they are lower than the true 7-day strength of that sample. So when the correct 28-day result is obtained, an apparent larger than normal strength gain is registered.

In practical terms this means that it is not possible to allow for the higher strength gain to be anticipated from concrete with a generally higher early strength (or the lower strength gain to be anticipated from concrete with a genuinely lower early strength) because this would result in magnifying any testing error in the early age specimen. Of course this applies only when the concrete is all of the same grade and under reasonably good control. Where mix composition may vary substantially and especially if duplicate early age specimens are used, so that the early strength is more accurately established, it may be more accurate to use a variable gain according to the particular early strength obtained. However, it should certainly not be a directly proportional or percentage gain, as quite often assumed.

As previously noted, the majority of the test results are likely to be close to average and to have only a cumulative, rather than an individual, significance in establishing the current level of that average. The results which really matter are the occasional low results since it is on the basis of these that strength may be adjusted. A frequently used scheme in Australia is to take three test specimens and test one at an early age and two at 28 days. This derives from the view that two 28-day specimens are required to establish the acceptability of the concrete.

On occasions this has been departed from when the first of the 28-day specimens fails to reach the specified strength and the other is retained for test at 56 days. This action displays a failure to understand the purpose of testing, which is to determine at the earliest possible time whether there is any need to revise the mix being supplied. This decision rests on the early age testing, the later age testing being required only to establish and continually reconfirm the prediction basis. If the early age specimen indicates a low result at 28 days, one of the specimens taken for 28-day test should be brought forward for immediate test to establish whether the low result was genuine or only a testing error. If the first 28-day specimen fails (following an acceptable early age result) it is certainly desirable to confirm or deny this result immediately. It may be legitimate to want to establish the 28 to 56-day strength gain but this should be done using the second specimen from one or more pairs which gave a normal (i.e. average) result on the first specimen at 28 days.

Note that those who remain dubious of the superiority of average gain over percentage gain as a prediction basis now have the opportunity to enter their own data in the sample program on the CD-ROM provided with this volume. The program is set up to provide an accurate assessment of the relative merits of the two assumptions on any inserted data.

5.17
NUMBER OF SPECIMENS PER SAMPLE

Three specimens seems to be the minimum desirable number. It is necessary to have an early test because control action must be taken earlier than 28-days. It is necessary to have at least one 28-day specimen to confirm the prediction basis and it is desirable to have a third specimen to cover the faulty test situation. It is also necessary that specimens from at least a proportion of samples are tested in pairs at one age or another. This is necessary in order to establish the extent of the testing variability and to continuously monitor whether there has been any change in it.

Strange practices are occasionally encountered in numbers of specimens per sample of concrete. At one time UK statisticians recommended a single (presumably 28-day) specimen per sample. It is true that this does give a more reliable figure for the mean strength and standard deviation **of test specimens** than the same number of specimens taken in pairs or other groups but it shows a total lack of understanding of the purpose of the testing. It would provide no indication of how much of the variability was due to the concrete itself and how much to the testing; there would be no way of ruling out a defective specimen on the basis of another result from the same sample; there would be no early warning of failures and finally the cost per specimen would be much higher because the cost of sampling, slump testing, recording, etc., would not be spread over several specimens from each sample.

In Singapore, on the other hand, cube specimens are often cast in sets of six, three being tested at 7 days and three at 28 days. On occasions they may be taken in sets of three with alternate sets of three all tested at 7 or at 28-days. The author was staggered to find an engineer who was concerned that ‘some of the concrete had lost strength between 7 and 28 days and was there a possibility that it would continue to lose strength?’ Obviously the two sets of three were not from the same delivery of concrete. The circumstances in which the actual concrete in a structure may fail to register a normal gain from 7 to 28 days are very limited (mainly a failure to wet cure) so an abnormal gain should be regarded as likely to be a failure of the testing process rather than the concrete itself.

Apart from the essential requirement that early and later specimens do come from the same sample, the question arises as to whether it is ever worthwhile to test three specimens at the same age. The author’s answer is only when testing is extremely bad and very cheap or for self-protection when faced with an old-fashioned person who is liable to reject concrete on the basis of a single low test specimen. It is well established that where a pair of test specimens from the same sample at the same age differ widely, the higher result is the best estimate of the true strength. This can be seen when the two results are compared with an early age prediction or, vice versa, when the two widely differing specimens are at an early age and predictions from them are compared to the later age result. However, this does not mean that the higher of the pair is never the one at fault, only that it is far less likely to be so.

5.18 DEPRESSION OF MEAN STRENGTH BY TESTING ERROR

If the assumption is made that the higher specimen is always correct, then if two laboratories conduct testing in parallel on a substantial number of samples, they will tend to obtain a different mean strength with the difference equal to one half the difference between the average difference between pairs of samples at the two laboratories. That is, any laboratory would find its average strength depressed by half its average pair difference. The author has found that laboratories with high pair differences not only do obtain a depressed mean in this way, but the depression is usually **more** than half the average pair difference. The significance of this is that if a laboratory has a high pair difference, then **even the higher** of the two results is likely, on average, to be less than the true strength.

A really excellent sampling and testing organization will have an average pair difference of little more than 0.5 MPa. So very little would be given away by using the higher of the pair as the best estimate of the true strength. A very poor laboratory with an average pair difference of, say, 2 MPa would, on average, be causing a strength depression of at least 1 MPa. It can be seen that the adjustment of mean strength is not what matters. What *does* matter is the treatment of the occasional sample with a pair difference of 3, or even more, and **one** of the results below the specified strength. In such circumstances, the Australian code permits the lower result to be disregarded.

The conclusion reached as a result of all the foregoing discussion is that, while concrete having a genuinely lower strength at an early age will probably also have a lower gain at a later age, it is necessary to forego taking account of this in predicting the later strength. This is because the additional accuracy obtained in this way would be more than counterbalanced by the additional error resulting from the magnified consequence of testing error in the early result. For example, suppose that a concrete has a specified strength of 30, a normal mean of 35 and an average gain of 10 from an early age. Thus an average early age result will be 25. It may be that a sample having a true early strength of only 20 will also have a true gain of only 8 to 9 and so will fail to reach the required 30. However, experience is that three times out of four when an early strength of 20 is obtained, the true early strength is at least 22, and a true gain of at least 9 may be experienced giving an apparent gain of 11 from 20 to 31.

It is again emphasized that the objective of early testing is to determine when a mix revision is necessary **and nothing else.** If a single **very** low result were obtained, sufficient to cause concern for structural safety or at least durability, subsequent action could not be confined to the sample of concrete giving the low result but would have to be very extensive indeed. The usual reaction of following up where the concrete went from the sample giving the low result and further testing it by coring, ultrasonic, etc., is usually an example of failing to think logically. There is one possible justification for such a course. This is where it is suspected that the test result was invalid, and the expectation is that the concrete in question will be shown to be normal.

It should also be recognized that the strength gain from an early to a later age will change from time to time. The ease with which such changes can be monitored and incorporated in the prediction system when using a simple addition basis is another point in its favour. A separate gain figure will be required for each grade of concrete and each of these gains will be monitored by cusum analysis.

5.19 A SELF-REGULATING BATCH PLANT?

Disappointingly, it can still not be reported that any client has tried a self-regulating batch plant as suggested in the first edition. Clients generally operate on tables of mixes set out in rows with cement content in 5 kg increments. Mix revision consists of moving one, or more rarely two, rows up or down based on current strength test data. Mix proportions may be determined initially by trial mixes but subsequently, and mainly, by eye adjustment of current mixes for fresh concrete properties. Linear interpolation is used between current mixes or between mixes determined at 100 kg intervals to give the 5 kg increments. This has been very useful to the author in confirming his techniques, since the tables of mixes can readily be converted to graphs of the author's MSF (mix suitability factor) against cement content. An initial exercise of this type was reported in 1996 (Day, 1996a, b) but many such have been carried out since with similar results.

The objection to the hard copy tabular system is that it potentially (and often in practice) denies the advantage of adjusting the mix for changes in aggregate grading, slump and temperature (although the latter two can be accommodated by a change of row in the table). However, having objected to the hard copy tabulation, it has been found to be useful to develop a system to automatically generate and print such a table (section 3.18.3). This enables easier examination of the results of specifying gradually changing relationships as cement content changes. A good example is the relationship between two sands, where it may be desirable to reduce the proportion of the finer of the two sands as cement content increases. This might be done crudely by replacing fine sand by cement volume for volume, but this would only maintain constant cohesiveness in very fortuitous circumstances. It has been interesting to note that in almost all such tables so far examined, the change in relationships has been almost exactly such as to keep MSF constant—and these tables were originated by others with no prior knowledge of the MSF concept.

The following is the section of the first edition under this heading. It has been thought better to leave this exactly as it was, even though minor changes in what might be saved as a mix type obviously follow from the above discussion about tapering ratios. This enables the reader to see how little has changed.

The Conad system automatically updates grading information and concrete test results from previous usage of the particular mix. Such results may show that adjustment is required of the water prediction factor or of the strength prediction factor. The new water factors determined after taking into account changes in MSF (surface area), silt content, slump, air content and concrete temperature on the concrete from which the data was obtained, and the calculated water content, will use the factor derived from the previous data but will be adjusted for the **now anticipated values** of all these factors. This should give close prediction, not only of average water requirement for a particular mix, but also of its variation from day to day and even truck to truck. Should some other factor have a significant influence in particular circumstances (e.g. varying carbon content in fly ash) the control system will show that this is happening and be of assistance in locating its cause.

Similarly for the strength factor. The calculated strength will have already allowed for most of the factors normally causing a difference between anticipated and obtained strength and it may be that the strength factor is largely a cement quality indicator.

The point is that having made all these adjustments automatically, the computer can easily be further programmed to reassert the original MSF and strength requirements. If programmed in this way, we would have a batching system which adjusts itself to take full account of both its own past performance and its currently available materials and anticipated conditions. This is considerably more than could be anticipated of the average plant operator. However, a human operator would no doubt still be considered essential to cope with emergencies and breakdowns. Of course, most if not all plants currently have a person to take orders for concrete and schedule production and delivery but this function also is tending to become centralized and heavily computer assisted where several plants jointly serve a particular area.

Yet one further step in automation would be the elimination of standard mixes. At least theoretically, a 'mix' could consist of a required minimum strength, a slump, an MSF value and perhaps a maximum aggregate size or ratio of say 20 mm to 10 mm material. Of course, cement type, fly ash substitution, etc. (if alternatives existed) would also have to be specified. The plant would effectively be operating as an expert system for designing concrete mixes, and it could do this quickly enough to individually design each truck of concrete.

One difficulty in the elimination of standard mixes would be in the QC system. However, the Conad QC system already analyses results on a multigrade basis and the principle could be extended. Results in the existing multigrade system are expressed as a proportion of the average results for the grade in question. In a 'no grade' situation each result would be expressed as a ratio between predicted and actual results for the batch in question. In other words, the graphs would be of the water and strength factors and also of ratios between expected and actual slumps and densities. Change always involves a learning process but it may be that such a change would present an opportunity for progress in QC rather than a difficulty.

It would still be possible to conduct 'single grade' analysis by taking all results for a particular specified strength. The analyser would have the choice of whether or not to restrict the range of MSF values, slumps, cement contents, temperatures, etc., to be included in the analysis.

The above possibilities have yet to be tried in practice. The author has been suggesting them since the early 1980s but admittedly they only became really practicable with the development of an interface between the batching and QC computer systems in 1989. It remains to be seen how long it will take to find a client brave enough to implement them!

6
Specification of concrete quality

The only change in the author's attitude to specification since the first edition is an intensification of the views expressed in it and a new realization of how strongly they apply to the USA. Many discussions have been held with US concrete producers and the general conclusion is that it is not profitable to invest in high tech quality control in the current American environment. There are three main impediments to cost-effective implementation of the kind of measures discussed in this volume:

1. A large proportion of concrete is of specified cement content so that no financial incentive whatever exists to undertake more than the specified minimum of control.
2. Many specifications actually do not permit mix changes without preliminary trial mixes. So the author's techniques for rapid reaction to change cannot be used to reduce variability.
3. Where the above points do not apply, the financial incentive to achieve low variability is reduced under US specifications owing to the use of a 10% defective criterion rather than the more usual 5% in most of the world. At issue is not the desirable margin between strength and permitted stress but rather the relative value placed on mean strength and variability. With a 10% criterion, one MPa of standard deviation is worth 1.28 MPa or say 6 to 7 kg of cement per cubic metre. With a 5% criterion, the figure is 1.65 MPa or say 8 to 9 kg of cement—a 29% greater incentive.

6.1
DEFICIENCIES OF EXISTING SPECIFICATION TECHNIQUES

The 1950s, 1960s and 1970s saw a great increase in our knowledge of concrete, in our ability to produce it in large quantities and in our ability to achieve high strengths and high quality. To our shame the same period also saw the production of vast quantities of concrete which were of inadequate durability and has already created such an economic problem, and such an eyesore, as to call into question the use of concrete for many purposes. Substantial amounts of this concrete have already been demolished, while concrete from earlier times remains serviceable.

The specification of concrete is clearly not solely a matter of technical knowledge but also of commercial pressures and human nature. As in many other fields, a little knowledge can prove to be a dangerous thing and our concrete specifications were generally not written by concrete technologists but by architects, structural designers and administrators through the medium of 'specification writers'. The initial reaction of such persons was to blame the strength specification technique and/or to blame the ready mix, cement, or admixture suppliers for subverting this by discovering how to produce concrete which satisfied a strength specification without having sufficient cement to provide durability. For a time there were ridiculous specifications which called for 20 MPa with a minimum cement content of 350 kg/m^3 or similar. What was wrong was that the specifier had insufficient knowledge to realize that higher strengths must be specified in order to ensure the lower w/c ratios needed for durability. Fortunately (as we shall see later) the pressure to specify minimum cement contents was resisted and most countries now recognize that **the higher of the two strengths required to provide for structural stresses and to ensure adequate durability must be specified.** However, it is now becoming apparent that it may also be necessary to specify the nature of the cementitious material to be used.

Historically, concrete was specified by its materials, their proportions, and the methods to be employed in its production and use. The supervising engineer was the person who best knew the relative qualities of materials and what was important in the satisfactory production and use of concrete. The contractor merely provided labour and simple tools and followed the directions of the engineer. The specification was generally simple in the extreme and the quality of the concrete basically depended on the expertise and diligence (and perhaps even the personality) of the supervisor.

Today concrete is usually purchased ready mixed from a specialist producer by the main contractor. Contracts are let on the basis of the lowest tender and a supplier prejudices his continued existence if he provides a higher quality (assuming this means a higher cost) than is strictly necessary to satisfy the specification. The supervising engineer cannot demand a higher quality than is provided for in the specification, although he may, and frequently does, fail to obtain this quality in full. The message is clear that, at best, only the quality specified will be obtained and that only if an effective enforcement system has also been specified.

A basic fault of most concrete specifications is that they regard concrete as either black or white, i.e. they provide only for the acceptance or rejection of concrete. Most concrete is in fact grey, i.e. is on the borderline between acceptance and rejection at the lower end of its quality spectrum. If it is not so, then the producer is operating uneconomically and will therefore be less likely to obtain the next contract. Of course most of the concrete produced (except under very poor specifications or supervision) is of higher quality than the minimum specified, but what we must be concerned with is the quality of the worst 10% or so. We are therefore concerned with the average quality and the extent of variation about that average.

We shall also find that it will be desirable, if we are not to be ruthlessly myopic, to consider the effect of our choice of specification basis on the future development of concrete technology.

6.2
SPECIFICATION BASIS

6.2.1
Aspects of quality

This question has been addressed in more detail in Chapter 1 and is only briefly reviewed here.

Quality is a matter of fitness for purpose. Just as we do not all need to drive a Rolls Royce motor car, very little concrete needs to be of the highest possible quality. The question is what constitutes an appropriate quality in particular circumstances and what is the best criterion of that quality.

The aspects of quality are:

- Adequate durability (usually of reinforced concrete).
- Adequate strength.
- Acceptable appearance.
- Low permeability.

- Dimensional stability.
- Surface texture.
- Low variability.

These qualities are not independent of each other but each, in some circumstances, may impose requirements not necessarily satisfied by compliance with all the other requirements. For most purposes the list may be regarded as to some extent in order of importance. Certainly there can be no question that adequate durability (not always particularly high durability) is the most important criterion since without it the concrete by definition cannot continue to display any of the other qualities for the desired period of time.

To some extent **low variability** is the oddball on the list. It has been placed last since, if all the other criteria are satisfied without it, then it would be non-essential. However this could only arise if the average quality is much higher than necessary. In fact for most purposes, and especially from the economic viewpoint, low variability will be seen to be one of the most important characteristics of concrete and its encouragement will strongly influence how we should choose to specify and control concrete. It is also noted that in the real world it is unlikely that concrete without a low degree of variability will satisfy any of the other characteristics in full, with the possible exceptions of strength and dimensional stability.

There has already been adequate discussion (section 1.3) of **strength** and its dependence on w/c ratio.

Having agreed on the primacy of **durability**, the next list must be of the factors involved in its achievement:

- Cover to reinforcement.
- Low permeability.
- Resistance to aggressive chemicals.
- Avoidance of internal disruption.
- Resistance to cracking.
- Resistance to abrasion.

Cover to reinforcement is given first place in the durability list since it has been clearly established to be the factor most frequently causing deterioration, however it is not part of the subject matter of this volume.

Low permeability which appears on both previous lists, is influenced by (again in approximate order of importance):

- Compaction and uniformity in place.
- Water/cement ratio.
- Curing.
- Freedom from cracks.
- Absence of bleeding voids.
- Presence of pozzolanic materials (e.g. pfa or silica fume).
- Use of air entrainment.
- Suitable aggregate grading.

Again first place is taken by a factor not covered by this volume, since it is a fact that most leaks in concrete structures occur at defects rather than through general permeability. Curing is also an extremely important factor outside our present scope. While curing is important to strength development, it is distinctly more important for low permeability—especially when it is the permeability of the surface layer (i.e. cover to reinforcement) which is in question. It may well be economical, in some circumstances, to provide a higher potential strength in lieu of good curing to develop the full potential of a barely adequate mix. However, the same philosophy applied to permeability would necessitate a substantially more expensive mix.

This leaves w/c ratio as the most important factor in our present scope, particularly since water content is also of direct importance in reducing shrinkage (and therefore cracking tendency) and in reducing bleeding.

Looking now at resistance to aggressive chemicals, the key factors are:

- Low permeability.
- Cement chemistry.
- Use of pozzolanic materials.

The most readily attacked compound in cement is tricalcium aluminate and the most readily attacked hydration product of cement is calcium hydroxide (which is liberated in considerable quantities during the reaction between cement and water). Tricalcium aluminate is limited by specification in low heat Portland cement and in sulphate resisting Portland cement. Tricalcium aluminate (C_3A) reduces the rate of penetration of chlorides into concrete so that unfortunately there is a conflict between providing maximum resistance to sulphate attack and maximum resistance to steel corrosion by chlorides. Pozzolans combine with free calcium hydroxide to form more durable compounds but again it is this material which is principally responsible for the alkaline environment which protects steel against corrosion. However, there is still ample calcium hydroxide to provide the required alkalinity. Providing there is adequate curing, the additional impermeability provided by the pozzolan more than compensates for any reduction in calcium hydroxide in resisting carbonation.

While the topic is currently the subject of vigorous debate, in the author's opinion the best compromise for resistance to combined sulphates and chlorides is an ordinary Portland cement plus a pozzolan-about 20 to 40% if a pfa, 60% or more if a blast furnace slag, 5 to 15% if silica fume. In the absence of a pozzolan and where only sulphate resistance is required, low heat or sulphate resisting cements are fully satisfactory but for both sulphates and chlorides (e.g. sea water) a modified OPC with 4 to 8% of tricalcium aluminate should be used.

For resistance to cracking the factors are:

- Avoidance of restraint to shrinkage (in design).
- Reduced shrinkage.
- Reduced bleeding settlement.
- Reduced heat generation.

These amount to a need for reduced water content and the use of either pozzolanic substitution or a low heat cement.

Resistance to internal disruption (other than by sulphate attack) means essentially avoiding alkali-aggregate reaction either through selection of non-reactive aggregates (Chapter 7), or a low alkali cement, or by using a pozzolan to consume alkalis.

Resistance to abrasion is largely a matter of adequate strength, except that surface finishing techniques are even more important (section 12.5).

Dimensional stability is essentially a matter of selecting a suitable coarse aggregate and limiting water content so as to reduce drying shrinkage.

Surface texture of formed surfaces is largely a matter of the form itself and the release agent used. As such it is outside the scope of this book. The main concrete properties involved are resistance to bleeding and segregation.

It should be fairly clear from the above that the most important basic criterion of quality is the w/c ratio. Other important factors are uniformity and type of cement, with a slow acting cement being preferable from almost every viewpoint except early strength.

Unfortunately it is difficult and expensive to accurately measure w/c ratio directly. Even if this could be done, it would still leave some matters unresolved (such as bond to coarse aggregate and indeed general aggregate quality, also shrinkage, moisture movement, air content, etc.). It is well established that compressive strength is closely related to w/c ratio. Furthermore, in so far as this is not the case, the cause is likely to be inferior aggregates or an excessive cement content or inadequate compaction, in all of which the strength better reflects the quality than does the w/c ratio. The one exception is air entrainment, which will reduce strength at a given w/c ratio while improving durability and other desirable properties. Strength also has the advantage of being very suitable for the examination and control of variability.

6.2.2 Possible deleterious effects of specifications

The practice of specifying a **minimum cement content** still persists in some countries. **It cannot be too strongly emphasized that this has probably been a major factor in impeding progress in the technology of concrete production.**

It must be realized that concrete is usually produced by commercial organizations charged with the responsibility of making a profit for their shareholders. Certainly there are likely to be a small proportion of altruistic individuals in the industry dedicated to, or at least receptive to, the concept of improving technology for its own sake. Even such individuals will not be permitted to expend much time and resources on activities which do not yield financial benefits. Even if the individual in question has full power to act, he will not have funds available to purchase equipment, engage extra technical personnel, etc., if his organization is not profitable. It is true that the building of a good reputation may improve sales and that the avoidance of problems may save considerable expenditure. Nevertheless, it behoves us to make it profitable to achieve technical progress if we are not to seriously inhibit such progress.

It is surely obvious that the effects to be desired are those of low variability and reduced permeability and shrinkage in addition to the specified strength. No knowledgeable person would prefer concrete with a mean strength of 50 MPa and a variability (standard deviation) of 6 MPa to concrete with a mean strength of 45 MPa and a standard deviation of 3 MPa. Both provide a characteristic strength (on the basis of 5% below) of 40 MPa, but the former is likely to contain 30 to 40 kg/m^3 of additional cement. An over-sanded concrete with a sand of high clay content may have a water requirement 20% higher than normal. It would therefore require about 20% more cement for equal strength. It would also be more permeable and have much greater shrinkage. Since the unwashed sand would presumably be cheaper than a good sand, and cheaper than coarse aggregate, such a mix would be likely to be the economical solution under a minimum cement content specification, but not otherwise.

Be perfectly clear about this. If **you** specify a minimum cement content you are expressing a preference for high variability, over-sanded concrete using the cheapest available aggregates by a producer who has made the minimum possible investment in plant, technical personnel, R&D, etc. **This type of specification has been a major factor in delaying for thirty years the advent of the techniques advocated in this**

book. Such specifications are **still** being advanced to the author as a major impediment to the profitable introduction of the techniques to the USA and some parts of SE Asia.

There are certainly many situations in which cement content should not be less than some particular value. This is quite readily obtained by specifying a strength which cannot be attained with less than the required cement content. Such a way of ensuring the desired cement content also provides an incentive to use good aggregates and technology. It is also much easier to check compliance with a strength specification.

Another less serious, but not inconsiderable, deleterious effect is that of the use of 28-day strength for specification purposes. The effect of this is that any strength developed later than 28 days is of no value to the concrete producer. The concrete producer will therefore prefer the cement which gives the greatest strength at 28 days per dollar of cost. The pressure to sell their cement causes cement producers all over the world to concentrate on 28-day strength to the detriment of other quality considerations.

Concrete should not be **controlled** on the basis of 28-day strength even when so specified. Rather it should be controlled on the basis of strength at 7 days or earlier, but using an early age strength **predicted to provide the specified 28-day strength.** Once this is realized, it can be seen that there would be no difficulty in using a 56-day strength criterion. Such a strength would similarly be translated into a required early strength and the pressure to provide maximum strength at 28 days would be avoided.

On the whole, it is concluded that compressive strength is the least unsatisfactory criterion of concrete quality, since it permits the greatest precision of assessment, including assessment of variability. However, there are a few provisos to this conclusion:

1. Entrained air, if desirable, must be separately specified.
2. The 'quality' of concrete is only measured by strength for a particular cement. It does not follow that if one cement gives a higher strength than another with identical mix proportions, it provides higher durability and it particularly does not follow if the higher strength is only at an early age. It may be necessary to specify the type of cement to be used and to adjust the strength specified so as to provide the required quality. Thus, if cement development continues to provide higher and higher strengths for a given w/c ratio, it may be necessary to continually review (i.e. increase) the strength requirement to provide a given degree of durability.
3. If practicable, it may be desirable to specify strength at an age later than 28 days (say, 60 or even 90 days) so as to judge quality with a greater degree of independence from cement type. Hopefully it may be possible in this way to reverse the pressure on the cement industry to produce higher early strengths where this is not essential, or even not desirable, for particular applications.

6.2.3 Encouragement of good control

In most countries, a percentage defective figure of 5% is used as a specification basis. This means that the mean strength is required to be 1.645 standard deviations (the use of three decimal places here is unrealistic but is used to explain why some countries use 1.65 and others 1.64) above the specified strength so that not more than 5% of test results will fall below the specified strength. A major exception is the USA where 10% (i.e. 1.28 standard deviations) is used for most concrete.

This is not something which is likely to be changed overnight. However, in the author's view, the decision has not been made on the best logical basis. The basis used has been a consideration of what proportion of concrete of less than the desired strength it would be reasonable to permit. A more suitable criterion is the relative value of mean strength and low variability. The use of a larger multiplier for the standard deviation (say, 2, or even 3) would give a greater financial advantage to those able to achieve a lower figure, i.e. it would provide a greater incentive to achieve good control.

There is not necessarily any correlation between the margin of mean over design strength to be provided by an average producer and the choice of the standard deviation multiplier. The required mean could be taken as specified strength, minus 4 MPa, plus 3×standard deviation. For a producer with a standard deviation of 3 MPa, this would give the same mean strength as specified strength plus 1.65×standard deviation. However the new basis would give him an advantage over his cruder competitor with an SD of 5 MPa and a disadvantage compared to a high tech competitor achieving an SD of 2 MPa. (section 10.2).

6.2.4
The dilemma of imprecise assessment

Having selected strength as the criterion of quality, it might then appear reasonable to divide the total amount of concrete into discrete lots, to test each lot, and to accept or reject the lots on the basis of the tests. This is in fact what most specifications attempt to do. Unfortunately it is economically impracticable to obtain sufficient test results to accurately assess the quality of a lot which is small enough to reasonably accept or reject as a whole. Most specifications **both** assess quality inaccurately **and** apply the decision to too large a quantity of concrete to accept or reject as a unit.

The results are, not surprisingly, that much unsatisfactory concrete escapes detection and that when concrete is shown to be defective according to the specification criteria, the usual outcome is that it is eventually accepted, and paid for in full, after a long sequence of meetings, drilling of cores, etc. **Very** defective concrete is of course removed, if detected, but the real problem is concrete which is not fully satisfactory but is capable of fulfilling its intended purpose. A frequent problem in floor slabs is concrete which is under-specified, e.g. 20 MPa concrete is specified when at least 30 MPa would be needed for satisfactory performance. When 19 MPa concrete is actually provided, it is obviously unsatisfactory, but who is most at fault?

A particularly unfortunate aspect of attempting to assess quality on an inadequate number of results is that variability is less accurately assessed than mean strength and so tends to receive inadequate consideration.

The fact that strength is usually not so much an intrinsic requirement (as regards the last few per cent) as a selected criterion of general quality is completely forgotten, as is the substantial degree of error in typical testing.

The point is again laboured that most concrete is not black or white but grey, i.e. market forces ensure that it will be close to the borderline of acceptability. To have rejection as the only penalty is equivalent to prescribing a $10 000 fine for breaking the road speed limit by a smaller mar gin than the accuracy of a police radar—and the same penalty for going at three times the legal limit. It can be imagined that there would be bitterly contested cases with expert witnesses testing and retesting the radar devices, etc.

6.2.5
The solution of separating requirements

The solution to the current unsatisfactory situation requires firstly that it be fully understood and secondly that logic rather than prejudice be applied. The requirements can be defined as follows:

1. The quality of concrete must be accurately assessed, including its variability, and the producer must be motivated to attempt to provide the desired quality in full.
2. If the quality of production becomes inadequate to satisfy the specification, this must be detected and rectified at the earliest possible moment.
3. The possibility that some part of the structure will have inadequate strength to be structurally safe (at a particular age) must be guarded against as far as possible.

It is very important to realize that there is no particular reason why any two of these requirements have to be accumulated into a single requirement. This applies particularly to the first two, which will be seen to be very different in character viz:

1. The first requirement includes no reference to time or to a limited amount of concrete, it refers solely to accuracy, to variability, and to motivation.
2. The second requirement stresses only urgency and action, there is not necessarily any emphasis on accuracy or on never being wrong.

The reader is urged to re-read this section and to consider it carefully. The point is so simple, and has been expressed in so few words, that it may easily be overlooked. Yet the message is that the fundamental basis of 99% of the world's specifications for concrete is essentially unworkable and that a very simple and satisfactory alternative is available.

6.3
CASH PENALTY SPECIFICATIONS

A statistical analysis of 30 or more test results will provide a very accurate assessment of the mean value and standard deviation of those results. Since it is possible to predict very closely how much additional cement would be required to raise the mean strength of concrete by a given (small) amount, it is possible to calculate almost exactly how much it would have cost (in terms of additional cement) to raise the strength of marginally defective concrete to an acceptable level. This cost will vary in different parts of the world but is not likely to exceed 0.5% of the extruck cost of the concrete per 1 % of strength deficiency. The ex-truck cost of the concrete is in most cases not likely to reach even half of the total cost of the placed concrete, including labour, formwork and reinforcement and even this cost is probably small in relation to the cost, including program delays, overheads, etc., of actually removing concrete from a structure.

Clearly there is a very large margin between the saving to be had in making concrete a few per cent understrength and the cost of having it rejected. However, before concluding that any concrete producer must be mad to work anywhere near the rejection point, it is necessary to consider how indefinite that point is, how rarely concrete is actually ever rejected, and what the profit margin is. The latter point may strike academic or public service readers as irrelevant but if profit margins are so low that a producer can obtain a very substantial percentage increase in profit with only a very small risk of actually suffering any substantial consequences, many will take the risk.

Consider now the situation when a cash penalty of say 1% of the ex-truck price of the concrete per 1 % strength deficiency is to be levied on the basis of the statistical analysis of any 30 consecutive test results. It can be shown (section 12.3) that there is very little chance of escaping a penalty well in excess of any savings on cement (double on average is the intention) so there is absolutely no point in the supplier deliberately supplying under-strength concrete. Another crucial advantage is that it is not necessary to know

where defective concrete is in order to penalize it, and no particular sampling rate is necessary. Thus it would be possible, at least theoretically, to take only one sample a week for a year on a huge project using 1000 m^3 of a particular grade of concrete per day and still levy an appropriate penalty (provided of course that the samples were unbiased).

Another advantage of this type of specification is that any desired emphasis can be put on low variability by selecting an appropriate permissible percentage defective. Also the specified test age could just as easily be 60 or 90 days as 28 days.

It would appear that all aspects of the first requirement of a suitable specification are fully satisfied by this type of specification.

It is possible, if the specifier is resolutely opposed to cash penalties, to levy the penalty in terms of an increase in required strength for a defined subsequent period, or a defined investigation procedure which the supplier can see will cost more than he can hope to save on cement. This is a situation in which the author often finds himself. It can work, it **does** work, but it is an infuriating waste of time and lack of precision. It is like handing a surgeon a kitchen knife for an operation and insisting he wears dark glasses. Perhaps a better analogy is handing a dummy firearm to a bank robber—it will work if he is a good enough actor.

However, there is one quite satisfactory alternative to a cash penalty. This is a cash bonus for providing over-strength concrete within defined limits. Elsewhere (Chapter 12.4) it is argued that it is often quite desirable to deliberately over-specify strength by a small margin. A cash bonus provision can provide an identical incentive to a cash penalty plus over specification. If it does so in a way more acceptable to some people, so be it.

6.4
RAPID REACTION TO UNSATISFACTORY QUALITY

The second requirement of section 6.2.5 is early rectification of unsatisfactory trends. If a cash penalty provision is in force based on strength at 28 days or later, one of its best features is that it produces an identity of interest between the concrete supplier and the project supervisor in the rectification of unsatisfactory trends at the earliest possible stage. It is not necessary to prove beyond reasonable doubt that a downturn has occurred because the supplier will find it in his financial interests to react to any substantial likelihood of this. Conversely the supervisor can take a relaxed attitude to any but major downturns (which will in any case be clearly exposed) secure in the knowledge that the supplier will be keen to rectify the situation.

Emphasis has been placed above on the need for a high degree of accuracy in judgement and the absence of any pressure of time in applying cash penalties. The situation is exactly reversed in the case of early spotting of downturns. Here the need is to act early and the risk of occasional, or even fairly frequent, errors of judgement is readily acceptable. Experience is that downturns tend to occur as isolated sharp drops of between 2 and 5 MPa in mean strength. When such a drop appears to have occurred, an immediate appropriate increase in cement content should be made. The next day's test results will tend to confirm or deny the suspected drop and allow the cement increase to be continued, discontinued or further increased.

There is a philosophy, often sound in past practice, that frequent adjustments should not be made, as the result will be that there is no continuous period permitting a valid statistical assessment of the situation. Such adjustments can indeed often add to variability when based on 28-day results, since the cement increase can well be made at the end of a period of lower strength and merely intensify the next upturn. However, if based on 7-day or earlier results, this is unlikely to be the case. Such adjustments, as well as increasing the mean strength, will usually tend to reduce the overall variability over a period (Chapter 10).

The same argument applies to strength upturns and cement reductions, although in this case a little more caution and a slightly higher degree of certainty may be wise.

As discussed in the section on quality control, the most efficient mechanism of 'change point' detection is the use of cusum graphs. The second requirement of a suitable specification will therefore be met by requiring that initial tests be made at a selected early age (see section 5.16 for discussion of preferred age) and that they, together with all other related data, be analysed by cusum on a computer and appropriate action taken in the event of a strength downturn.

6.5 SPECIFICATION OF METHODS, EQUIPMENT AND FACILITIES

6.5.1 Security against isolated defective loads

The occurrence of isolated trucks of very low strength concrete (e.g. through a gross cement deficiency or a gross excess of a chemical admixture) is normally extremely low, perhaps 1 in 1000 trucks, but it is not zero. Statistical quality control is clearly not applicable to such events and the risk of them cannot be used to justify additional traditional compressive testing (since even if alternate loads were tested, at enormous expense, there would still be only a 50% chance of detecting such an error). Their detection generally depends on the diligence and powers of observation of the truck driver or job personnel, rather than on planned action. However, recent developments, including computer batching and its convenient rapid analysis (see below) do provide a much greater security against such occurrences. Also it is possible, if considered worthwhile, to apply some form of early age non destructive testing on a 100% basis. For example, it would cost little to informally test every critical column with a Schmidt Hammer as it was demoulded.

6.5.2 Batch plant equipment

The availability of computer operated batching equipment, able to positively record the actual as-batched quantities for each batch of concrete, is an important factor in the control process. It provides the following advantages:

1. It gives a considerable degree of assurance that the batches sampled are in fact typical of the whole output. This greatly strengthens the argument in favour of a reduced rate of testing, allowing emphasis on quality of testing and a thorough analysis of the results rather than sheer volume of testing.
2. It provides a ready means of adjusting mixes and of keeping accurate records of what adjustments were made and when.

It is therefore fully justified to specify that such equipment should be used on any important work and that the resulting data bank should be made available to the supervising team. Should such equipment not be made mandatory, it would be reasonable to halve the otherwise envisaged sampling rate if it were provided.

6.6 SPECIFICATION OF EARLY AGE STRENGTH

It is not desirable to specify **times** for stripping, stressing and de-propping as these would have to be very conservative (as now) in order to cope with climatic variations and variations in concrete properties. A strength requirement should be specified but it is preferable that this should be assessed by test of the *in situ* concrete, since this is often quite different to that of even job-cured test specimens. Techniques available include pullout and break-off tests (see Chapter 11) and also the Windsor Probe or Schmidt Hammer. Note that, although the Schmidt Hammer in particular is not very accurate, it may well be better to have an inaccurate measurement on the particular *in situ* concrete involved than to have a very accurate measurement on a specimen the strength of which may have little relationship to that of the *in situ* concrete.

Another development on early strength control is to record the temperature history of the *in situ* concrete and use this to calculate the strength relative to that of a test specimen. While it is a significant development, it is inappropriate to discuss it here because it does not affect the conclusion that the necessary strength should be directly specified. Details are given in Chapter 12.2.

6.7 PROPOSAL-APPROVAL SPECIFICATIONS

Without increasing cost excessively, it is virtually impossible to so specify a concrete mix that it will necessarily be satisfactory. Strength, slump and surface area (as measured by the author's MSF) can be specified but problems can still result from details of the combined grading. Mix design should be a matter of combining available materials so as to minimize any disadvantages they may have individually. It is possible to specify conformance of each individual material to ideal requirements so that they can be combined in standardized proportions, but this is usually only practicable on large contracts for which aggregates are being specially produced. Even so, some variation is inevitable, and it is difficult both to require rigid compliance with specified proportions and to provide for variation. This path leads to full acceptance of total responsibility for concrete quality by the supervising authority, which is undesirable for many reasons (from needing to take over control of incoming materials quality to facing claims by the Contractor that any defects in the finished product are due to matters beyond his control). The Australian Government airfield construction branch used such techniques in the 1980s. Excellent concrete resulted, and it was considered by those in charge that the high cost sometimes caused was justified by the importance of the work.

The preferable course is to specify as closely as possible the properties required of the concrete and require the Contractor to set out in full detail exactly how he proposes to provide them, including **his** specification limits on incoming materials and within what limits and to what accuracy he proposes to adjust the mix. This clearly gives the Contractor absolute freedom to propose the most economical and practicable way of providing concrete of the required properties. It is very much easier to detect any unsatisfactory features of such a proposal than it is to so specify a mix that it could not possibly have any unsatisfactory features.

Once the Contractor's proposals have been accepted by the Supervisor, they become the specification. Insistence on conformance to this specification is easier since the Contractor, having proposed it himself, cannot claim it to be unrealistic in any way and there can be no surprise loopholes' in the original specification.

6.7.1
Early detection of changes

Many specification writers take the view that only the 28-day strength of concrete should concern them (except where early age stripping or prestressing is involved). However, enormous costs can be incurred by the late detection of inadequate concrete in the structure. The owner can theoretically recover losses incurred by delay in such items as financing difficulties or, for example, failing to open for business in time for the Christmas trading rush, etc., but only if it does not push the contractor into insolvency, and only if the losses can be established to the satisfaction of a court of law. It is therefore very important not to allow contractors to get themselves into an untenable situation in regard to concrete quality. Also, if concrete is only marginally inadequate, the owner may be pressured by his own need for early completion to accept a compromise he would have preferred to avoid. Therefore the specification must require early age testing to be carried out.

It is important to be clear what is likely to happen if early age testing does reveal substandard concrete. If even one test in 1000 normally led to concrete being removed from a structure, accelerated testing would be essential. Rapid Analysis Machines (Cement and Concrete Association) would abound and overnight steaming of specimens would be routine (which is not to say that these things would not be an advantage in any case). However, the normal situation is that a number of consecutive results lead to the revision of the concrete mix. The time at which the need for the revision is considered to be established is dependent not only on the test age, but also on the frequency of testing, the variability of both the concrete and the testing process, and the effectiveness of the result analysis system. If accelerated testing resulted in a less precise prediction of 28-day results or if fewer results were obtained (as would be the case at a given expenditure) this could actually lead to a later establishment of the occurrence of a change than the use of 7-day results.

The choice of the particular early age test to use will vary with the circumstances. The important factors are:

1. If testing is to be at a laboratory which normally operates a 5 or 6-day week, the cost of testing at weekends may be exorbitant (and the **quality** of weekend testing is sometimes less certain). A mixture of 3, 4 and 5-day results is difficult to analyse, although a recent development by the author may change this perception. So a 7-day test may be the only reasonable choice in some locations.
2. The maturity of concrete of a given age is dependent upon its initial temperature and subsequent curing history. The maturity/strength curve is very steep initially. This means that the precise time of test may be a significant factor at an early maturity.
3. Where temperatures vary substantially and erratically from day to day or during a day, even a precise testing age in hours may not be sufficient to accurately quantify maturity.
4. Accelerated testing may overcome the problems of **inadequate** maturity but must be very carefully done if it is not to add to the **variability** of maturity at test.

The author has generally used 7-day testing in Australia and 3-day testing on major projects operating 7 days per week with an on-site laboratory in tropical countries. He has also tried 1-day non-accelerated testing in the latter circumstances and found 28-day predictions to be fairly reasonable, but not quite accurate enough. Two-day testing (although not tried by the author) would probably be ideal in these circumstances. Accelerated testing is probably useful in cold countries under 7-day, onsite lab conditions but the author is a little dubious of its value in general. If it represents **either** substantial extra expense or additional inaccuracy in prediction, it may be counter-productive.

The latest development is the measurement of temperature history by an embedded probe in the early age specimen (or a duplicate kept under identical conditions). With such a history a prediction as accurate as that from a 7-day test can be made at an age less than 24 hours (section 12.2).

If a test age earlier than 7 days is used, it should initially be backed up by 7-day testing. It is easy (using computer based QC) to establish the relative average difference between predicted and actual 28-day (or later) results for the two alternatives and especially to see (using cusum analysis) which most accurately predicts changes in mean strength at the later age. A selection can then be made with confidence as to which alternative should be dropped.

6.8 THE RECOMMENDED SPECIFICATION

Putting together the foregoing elements, it can be seen that the ideal specification should include:

1. A minimum strength at 28 days or later, based on the analysis of any 30 consecutive results. The requirement should be in the form:

   $$\text{Required mean strength} = \text{specified min strength} + k\,\sigma$$

   where σ is the standard deviation of the 30 results and k is an arbitrarily selected number. The coefficient k should preferably be 2 to 3 rather than 1.28 or 1.65 as is currently common. This would give greater emphasis to the attainment of low variability and, as explained in section 10.2, does not necessarily imply an increased mean strength since the higher margin could be offset to any desired extent by adjusting the 'specified minimum'.
2. A cash penalty rather than rejection for a failure to satisfy the first requirement, that penalty, at least for the first 2 MPa (300 psi) of deficiency, being only approximately twice the cost of the additional cement which would have been required to satisfy the strength requirement. For larger deficiencies an increased rate of penalty may be appropriate, with provisional rejection of the whole of the concrete represented by the 30 results for a deficiency exceeding 5 MPa (750 psi). 'Provisional rejection' means no payment and extensive *in situ* testing to determine whether physical removal of some or all is necessary. It should be noted that it is inconceivable that such a deficiency would actually occur with the early age control system in operation, i.e. this provision is only there to satisfy those who would otherwise say 'but what if…?'
3. A requirement that the concrete be produced in a computerized batch plant able to record full details of each batch.
4. A requirement that the results at a selected early age be analysed by cusum analysis in order to detect change points and that the mix be adjusted to maintain the forecast later age strength at the specified level.
5. A requirement that the contractor submit a very detailed mix proposal supported, if likely to be contentious, by test results on trial mixes or previous production. It may be important to permit this submission to be made and approved in principle (perhaps subject to post tender trials) prior to the tender date, as the acceptability or otherwise of these proposals could have major economic implications (e.g. if non-acceptability meant importing distant aggregates or cement).

If required, specify:

6. A particular type of cement or pozzolanic material.

7. A shrinkage limit, compliance with which is to be established by a trial mix or test results on a similar current production mix using the same materials. Note that the accuracy and repeatability of shrinkage testing is often questionable. Also, results are obtained at 56 or 90 days and it is impractical to reject otherwise satisfactory concrete at this age for this reason. The solution may be to have the concrete supplier nominate the range in which he can work with his quotation and to impose a cash penalty if he fails to stay within his nominated range.
8. An air content.
9. An (early) strength required for stripping, pre-stressing, de-propping, etc.
10. A maximum Los Angeles abrasion value where very high wear resistance is required. Note that, unless the wear will be such as to necessarily expose aggregate (e.g. tracked vehicles or scrap iron handling), the wear resistance of the coarse aggregate will be immaterial if that of the surface layer of mortar is good enough. Also note that the surface finishing process may be even more important than the concrete mix in achieving this.

6.8.1 Development of standard mixes

Specifications tend to assume that the concrete supplier will design a special mix to comply with the specification. This may be necessary, but it does have some disadvantages:

1. No history of previous satisfactory performance on actual projects.
2. No common pool of test results with same mix on other projects.
3. Truck drivers less familiar with mix, and less able to judge workability and detect abnormality.
4. Variability may be increased if every now and then the standard mix is supplied in error.

It might be reasonable to provide a financial advantage to suppliers who have satisfactory standard mixes in use, under routine control and with a range of properties established. The form of encouragement could be to allow a reduced testing frequency for such mixes and to require pretesting of new mixes.

The above points apply even for major projects, but their importance is far greater for the many ‘ordinary’ projects which probably account for most of concrete produced. Small projects cannot economically generate sufficient test data to maintain good control. This means that they are essentially dependent upon the producer’s quality assurance system. In such circumstances it is counterproductive to specify non-standard mixes unless absolutely essential. It is possible that a very small project could nevertheless derive great advantage from the use of 100 MPa concrete in a particular column, or involve a wall of exposed aggregate concrete of super critical appearance. In such circumstances special mixes are obviously involved and control costs are of little importance. However, a refusal to accept a standard 25 MPa mix for an internal floor slab would be justified only if the standard mix were distinctly unsatisfactory.

The specifier should generally concentrate on obtaining full information, both past and current, about standard mixes. The aim should be to check that the supplier’s control system is working well rather than to supplant it. These remarks are relevant when only compressive strength is regarded as important. The following section deals with requirements other than strength and the importance of using standard mixes of established performance is much greater in respect of such requirements.

A time is coming when it may be less essential to use standard mixes. The control system being pioneered by the author enables results from many grades to be combined onto a single control graph. The performance of mixes may be seen in terms of factors in mix design equations rather than a stand-alone assessment. The same situation has been encountered in many different industries (Toffler, 1981). Initially, mass production requires acceptance of a reduced range of products. However, as the technology of both production and quality control advance, the standardization necessary tends to be that of small parts of the whole. In this way products of very wide variety can be produced from components which are rigidly standardized. It is emphasized that this stage has not yet been reached in concrete technology and specifiers should currently concentrate on the second phase of reduced variety.

6.8.2
Requirements other than strength

Other properties than strength can be of considerable importance: Shrinkage.

- Durability.
- Permeability.
- Bleeding.
- Segregation resistance.
- Setting time.
- Wear resistance.

Looking into the future the author believes that the specification of limits on many of these properties will become more common with considerable benefit to the quality of concrete in use. The key to this development is a testing program to establish what are economically attainable values followed by the direct specification of the desired limit.

A case in point is shrinkage. In 1976, when the author first gave a lecture in Australia making these points, no one knew anything about concrete shrinkage except that it was often too high, especially in the case of pump mixes. Now several major projects have specifications setting shrinkage limits and every technically competent premix supplier knows which of his aggregates will meet such specifications and which will not. This situation resulted from the measurement of shrinkage of a range of mixes by the Australian Commonwealth Scientific and Industrial Research Organization (CSIRO) under a programme arranged by the Concrete Institute of Australia which showed what factors were involved and what were reasonable limits. To everyone's surprise, the choice between two basalt aggregates turned out to be as significant as whether the mix had the higher fine aggregate content considered necessary to achieve pumpability.

Each of the requirements listed above is now considered briefly.

6.8.3
Shrinkage

Essentially shrinkage (section 1.4.4) is affected by the following (in approximate order of importance but this order varies in different localities):

1. The total water content of the mix.
2. The volume change of the coarse aggregate on wetting and drying (to which absorption is often, but not invariably, a good guide).

3. The balance of the C_3A and gypsum contents of the cement (a difficulty occurs here because higher C_3A increases shrinkage unless offset by increased gypsum but gypsum content is limited by specification in order to avoid unsoundness, i.e. a disruptive expansion tendency).
4. The elastic modulus of the coarse aggregate (see below).
5. The clay content of the fine aggregate (mainly a problem via increased water requirement but there is a suggestion that shrinkage can be further increased if bond is reduced).
6. Maximum aggregate size. Larger aggregates may be of benefit through reduced water requirement but may also tend to substitute internal stresses for external shrinkage by resisting the shrinkage of the mortar surrounding them (hence the influence of bond and elastic modulus) to a greater extent than a smaller aggregate. This could result in increased permeability and may be an important reason for very high strength being easier to obtain with small aggregates.

6.8.4 Durability

Durability is mainly dependent upon w/c ratio and the selection of cementitious materials and to this extent has already been covered in section 1.4 and Chapter 8. It would be possible to attempt to directly specify durability in terms of some standard accelerated test (e.g. freezing and thawing, attack by sulphate solution, or rate of carbonation) but this does not appear to have been tried, or to be likely to be worthwhile.

Durability in general, and especially resistance to deterioration by freezing and thawing, is improved by air entrainment. Since, at least in rich mixes, air entrainment will increase the necessary cement content for a given strength, it will increase cost and therefore will not be provided unless directly specified. Note that a higher air percentage is required for freeze/thaw resistance than is generally desirable for grading improvement (for which 3 to 5% is normally adequate). The critical factor is the spacing of the air bubbles (there must be a bubble within 0.2 mm of any point in the cement paste for good frost resistance), hence the interest in admixtures providing the smallest possible bubble size because the strength reduction caused by air entrainment is proportional to the air percentage while the benefits are proportional to the spacing of the bubbles and therefore to their number. Also the desirable air content for freeze/thaw resistance should be considered in terms of the proportion of mortar in the mix (since obviously there will be no bubbles in the coarse aggregate) and should be around 10% of the mortar volume, which usually means around 6% of concrete volume. Caution should be exercised when air entrainment is combined with the use of a superplasticizer. Air bubbles may coalesce into larger voids, increasing the spacing. Also larger bubbles can be more readily vibrated out.

Durability is also influenced by susceptibility to alkali-aggregate reaction and sulphate attack but detail beyond that already given in Chapter 1 is outside the scope of the current volume. Essentially these considerations will influence any restriction on cement type and may necessitate insistence on the use of a pozzolan.

In so far as a minimum cement content is necessary for durability, it is important **not** to specify it directly. Rather a strength should be specified which cannot be obtained with less than the minimum cement content considered necessary for the particular situation as otherwise the intent may be subverted by an increased water content. The minimum characteristic strength for adequate durability may range from 30MPa (4350 psi) to 50 MPa (7250 psi) depending on the severity of the conditions. It may also be necessary to specify the use of a particular cement or a pozzolanic content.

In this respect it is noted that the lowest permeability and highest degree of durability is probably provided by the use of silica fume at 10 to 15% of cement content, together with a superplasticizing admixture to both reduce water content and help to ensure full compaction. Depending on relative quality and availability, a 50 MPa concrete with 25% of a good pfa is probably more economical and almost equally as good as the silica fume solution. Also 60 to 80% of a ggbfs would be an adequate substitute for most purposes (except possibly for wear resistance).

6.8.5 Permeability

Again w/c ratio is the main factor but adequate curing is extremely important and the use of pfa, silica fume or other finely divided materials is beneficial, providing that water requirement is not thereby significantly increased. Oddly enough, air entrainment will also reduce permeability by reducing internal bleeding voids and channels which permit the passage of water.

It is understood that the specification for the Singapore underground railway (following problems experienced with the Hong Kong underground) required the use of 10% of silica fume in the tunnel concrete solely on durability grounds. The use of silica fume to provide watertightness in basement walls in Singapore is now commonplace.

6.8.6 Bleeding

An in-depth discussion of bleeding is beyond the scope of this volume and should be sought elsewhere (but see section 1.4.2). Bleeding is the upward movement of water through the mass of the concrete, caused by settlement, internal voids, vertical pores and cracks. Bleeding is best inhibited by air entrainment and by the use of low slump concrete and/or by the increased use of fine material. However, bleeding can be promoted by gaps in the grading (i.e. by missing sieve fractions) especially in the sand grading. Harsh angular fine aggregates, especially if deficient in fines, can also promote bleeding. This partly arises because a larger proportion of fine aggregate will be required when it is of coarser grading and therefore will have a lower cement content per unit volume of mortar (cement is also a good inhibitor of bleeding). Excessive retardation also extends the period of bleeding and can result in serious bleeding problems in types of concrete which are otherwise low-bleeding. A bleeding limit may be directly specified as a percentage of water released per unit volume of concrete in a standard test. It should be noted that bleeding is quite temperature sensitive, being more severe in cooler weather (the same effect as chemical retardation).

The foregoing begs the question of whether bleeding is undesirable. Excessive bleeding is certainly undesirable, but the effect of eliminating bleeding is experienced when silica fume is incorporated in a mix. When the rate of surface water evaporation exceeds that of bleeding, the surface dries and a crack is almost automatic. This does not mean that bleeding should be deliberately encouraged, but suitable precautions are necessary in its absence. The precautions may include covering, shading, or mist spraying the surface, erecting wind breaks or spraying on a film of aliphatic alcohol. Such precautions may be particularly necessary when the concrete contains silica fume. Bleeding followed by revibration has the effect of producing better concrete quality, i.e. concrete of lower w/c.

6.8.7
Segregation resistance

Segregation is the separation of a mix into coarse aggregate and mortar. It is the opposite of cohesion, the property of sticking together. Cohesion is improved by more cement, by more or finer fine aggregate (i.e. by more surface area of aggregates) and by air entrainment. Segregation is promoted by high slump and by gaps in the aggregate grading. A mix may be cohesive at very high slump (if it has a high surface area) and it may be cohesive with a large gap in the grading (if it is of low slump) but a mix will be segregation prone if high slump is combined with **either** low surface area **or** a gap grading.

Segregation resistance costs money because the measures providing it all increase water demand and thereby increase cement requirement for a given strength (except air entrainment, which reduces water requirement but still increases cement requirement). The basis of economical mix design is therefore to keep segregation resistance to an acceptable minimum for the use intended.

The author's MSF (mix suitability factor) parameter is intended as effectively a direct measure of segregation resistance (Chapter 3).

6.8.8
Wear resistance

See section 12.5 on high quality industrial floors.

6.8.9
Setting time

Setting time is affected by cement composition, temperature, admixtures and water content. A detailed discussion is outside the scope of this volume. The setting time is most frequently questioned in the case of floor slab finishing operations. Another and even more critical situation is in the operation of slipforming vertical surfaces. Problems are generally caused by a failure to realize the very strong effect of low temperatures in delaying set. Generally any difficulties are soluble through the use of appropriate admixtures at appropriate dose rates (Chapter 9). It is not usual for specifications to contain any reference to setting time as this is negotiated between the contractor and the concrete supplier.

The one aspect which must concern a specifier or supervisor is the possible use of calcium chloride as a setting accelerator. Calcium chloride is without doubt the most effective and economical accelerator and therefore it is likely to be used if permitted. Its use was at one time almost universal in cold weather, and it is clear that a large proportion of floor slabs in which it was used have survived. Nevertheless it is well established that this material very strongly promotes the corrosion of reinforcing steel and its use is therefore no longer permitted in most countries.

6.8.10
Heat generation

Heat generation is a useful and desirable feature of concrete in cold weather but high cement contents lead to high heat generation and high temperature differentials which tend to cause early age thermal contraction cracking. While the author is opposed to the direct specification of **minimum** cement contents, it may well be desirable to specify a **maximum** cement content in some circumstances. Such a specification combined with a high strength requirement is a way of ensuring that a low water content is used and therefore that the mix is not over-sanded and does not contain excessive silt.

For mass or heavy section concrete, the direct specification of a maximum amount of heat generation may be appropriate. This can be directly measured in joules per kilogram in a suitable calorimeter, but the equipment is expensive. While such equipment is justified for organizations with a continuity of projects, a simple alternative specified by the author for a project with heavy raft foundations may be of interest.

The test is to line a tea chest or specially made crude box with 6 inches (150 mm) of polystyrene foam so as to produce a 1 ft (300 mm) cube container insulated on all six faces. The mix to be tested must be placed at the highest temperature at which concrete is to be accepted for use on the job (because the higher the initial temperature, the greater the **rise** in temperature will be) and a maximum rise in temperature is specified, normally in the range of 30 to 40°C. Temperature is preferably measured by thermocouples but a maximum registering mercury-in-glass thermometer in a metal tube filled with water at the centre of the cube can be used if electronic equipment is not available (without a tube the thermometer will be lost, the water is needed to give good thermal contact).

The job requirement will be in terms of the maximum differential temperature between the centre of the concrete mass and any external surface and may be 20 to 25°C. The necessary limit in the test will be affected by the size of the section and by whether any insulation is to be provided (both affecting the extent to which heat will be able to escape). It is normally substantially cheaper to limit heat generation and/or to provide insulation than it is to limit initial temperature by such means as flake ice or liquid nitrogen.

Maximum temperature may be limited by:

1. Using a slower acting cement.
2. Using pfa or substitution (ggbfs **may** also be effective, see Chapter 8).
3. Using a water-reducing admixture to reduce the necessary cement content.
4. Limiting the supply temperature.

Note that maximum temperature in very large masses is **not** in general reduced by using a retarding admixture. The temperature peak occurs **later** but is no lower.

Generally great expense may be involved in limiting supply temperature and is rarely essential. What really matters is **either** the maximum temperature reached (where there are unavoidable external restraints to contraction on cooling) or the maximum **differential** temperature within the concrete mass (when the core concrete will restrain contraction of the outer layers). The former case is often more economically achieved by the above measures and the latter by a combination of these measures and **insulation** of the concrete surfaces so that the concrete increases in temperature throughout its mass, reducing differentials.

6.9
SMALL VOLUME PROJECTS OR PLANTS

A frequent reaction to the author's proposals is that they could only be applied to large, long running projects. The proposal in question is that quality be specified in terms of a 'characteristic strength provided' (CSP) which is to exceed the specified strength (f'_c).

Thus we have:

$$\mathrm{CSP} \geqslant f'_c$$

$$\mathrm{CSP} = X - 1.65\sigma$$

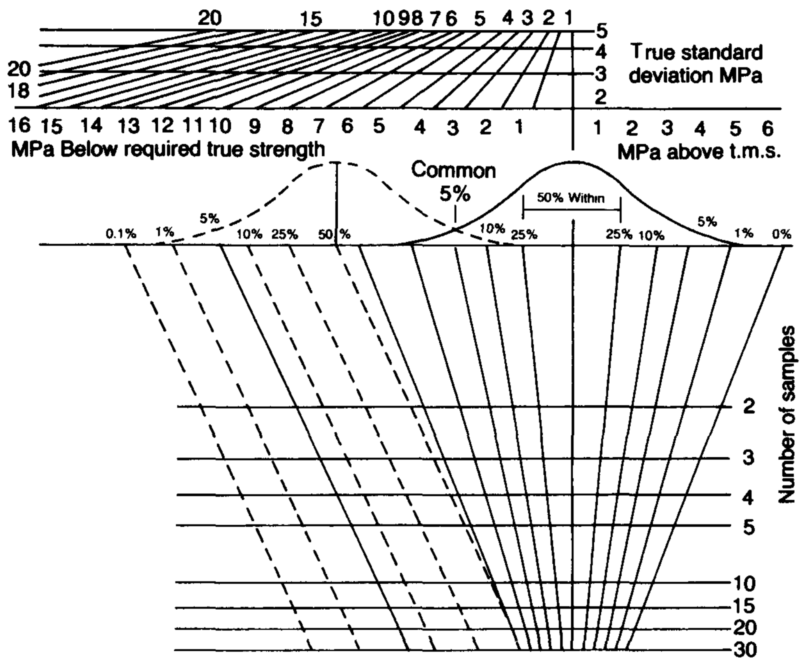


Fig. 6.1 Effect of sample size on accuracy of mean strength estimation.

where: X=mean strength of all samples

σ=standard deviation.

And it is proposed that cash penalties be levied on any shortfall.

The problem is that the values of X and σ derived from a limited number of samples are subject to substantial error. The solution is that mar gins must be allowed to cover the possible error and still ensure satisfactory concrete. It is necessary to examine the extent of error possible, and therefore the margins necessary. This applies to both the specifier and the concrete producer. The former will increase the specified strength until compliance with the specification will ensure the strength he requires. The latter will increase his target strength above that officially required so that the errors of assessment will still not cause him to be subject to a penalty.

Both these strength increases cost money in terms of additional cement content and the relative economics should be examined of taking additional samples in order to reduce the margin of uncertainty and therefore the additional cement cost. The reader who is appalled by statistics and tables of figures can skip the details and just hold on to the nice simple rules which follow, but for those who wish to be convinced, the details must be provided.

Although direct calculation of any particular situation is easier and more exact than reading from a chart, Fig. 6.1 is helpful in providing an overall view of the situation. The full lined flattened bell-shaped curve on the right represents the normal distribution of concrete strength, showing the percentage of a large number of samples which would lie outside various distances from the centre.

The horizontal scale is in units of standard deviation (× 0.5) and is converted into MPa by the nomogram along the top of the page for standard deviations between 2 and 5 MPa (the possible range in practice).

The lower half of Fig. 6.1 shows the distribution of the means of a number of samples taken from the distribution shown (the SD of the mean of n samples is $\sigma/\sqrt{n}$ where σ is the SD of the concrete, i.e. of single samples). The range shown is from 1 to 30 samples.

There are two points to note at this stage:

1. 50% of all groups of samples will show quite close to the true mean but a small percentage is widely distributed.
2. Referring to the nomogram at the top of Fig. 6.1, it is clear that high standard deviations are a worse problem than small numbers of samples in terms of accuracy of assessment.

In fact, the big problem with small samples is not that the mean strength is insufficiently accurate but that an accurate SD cannot be obtained.

For example, in the lower half of Fig. 6.1, project upwards from the intersection of the horizontal line indicating three samples and the almost vertical line indicating 5%. In the upper section of the figure, this vertical projection reads approximately 3.0 MPa on the horizontal line representing 3 MPa standard deviation. Thus if the concrete has a variability of 3 MPa, then the average of three test samples will be within ± 3.0 MPa of the true mean strength on 90% of occasions (5% being lower and 5% higher than this). However, referring to the accompanying distribution in dashed lines, there will be a 5% probability that a set of three results giving a mean 3 MPa below the true mean could have come from a different concrete, having a mean strength 9.5 MPa lower than the true mean (projecting upwards from the mean of the dashed distribution to meet the horizontal 3 MPa SD line at approximately 9.5 MPa).

Figure 6.2 shows the distribution of SDs derived from small samples in the same way as Fig. 6.1 shows the distribution of their means (in this case the SD of the SDs is given by $\sigma/\sqrt{n}$). It can be seen from Fig. 6.1 that the error in the mean (at the 5% level) in taking only two samples is less than 2.5 MPa when the SD is 2 MPa but almost 6 MPa when the SD is 5 MPa. An error of over 2 MPa is still obtained with even 15 samples when the SD is 5 MPa.

Looking now at Fig. 6.2 and taking only the 10% probability level, it is seen that the error in SD is ± 1.05 MPa from a sample of three when the **true** SD is 2 MPa (i.e. a true SD of 2 could be estimated as anywhere from 0.95 to 3.05 MPa). With a true SD of 5 MPa, the error from a sample of three is ± 2.61 MPa (i.e. it could appear as anything from 2.39 MPa to 7.61 MPa). Even with 20 samples, the true SD of 5 MPa is subject to an error of ± 1.01 MPa.

However, it should be borne in mind that these are the limits of the dis tributions, most of the time even a sample of three will give an SD estimate within 1.5 MPa of the true value.

A given mean of several samples may just as easily be at the upper or lower extremity of a distribution. Thus a mean of samples may be at the lower 5% limit in the distribution shown in full lines or at the upper 5% limit of the second distribution shown at the left of Fig. 6.1. The figure is constructed so that the 5% upper limit of the dashed distribution is coincident with the lower 5% limit of the full line distribution over the whole range of 1 to 30 samples. However, the percentage limit lines from the left-hand side of the dashed distribution do not taper towards the mean over the range of sample numbers because their function is to show the range of **individual sample** results to be expected from the dashed distribution with its mean located as described above. It is emphasized that the 0.1% limit is very theoretical and it is most unlikely that any results will actually fall outside the 1% limit. For example, the mean of three samples will just be acceptable (at the 5% probability level) if it falls 0.95σ below the target mean. On the 2 MPa SD scale this reads= 1.905 MPa, on the 5 MPa SD scale it reads as 4.76 MPa. Moving now left along the same horizontal line, it is seen that this mean of samples could equally well have come from the dashed distribution with a

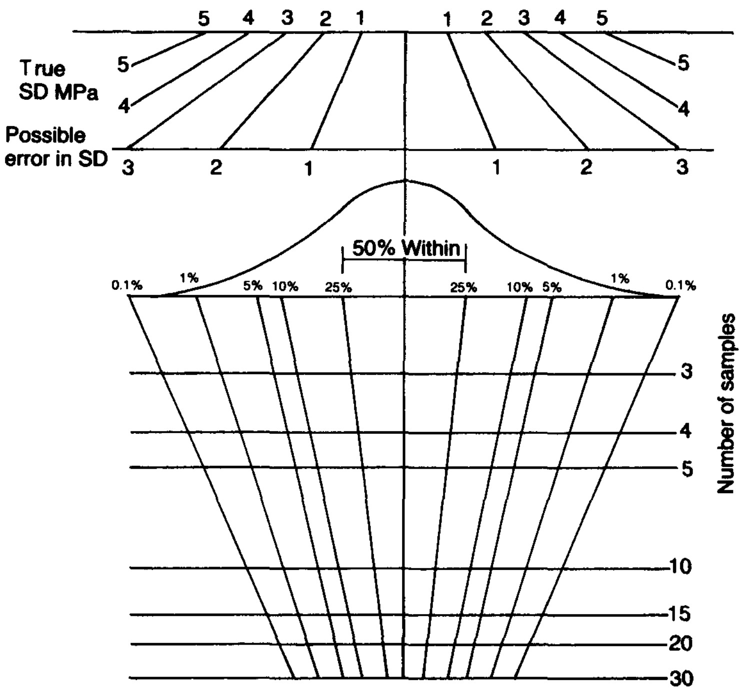


Fig. 6.2 Effect of sample size on accuracy of estimation of standard deviation.

mean 0.95σ **below** the mean of samples (3.81 MPa on 2 MPa SD scale) and that this distribution would be expected to include 1% of results 2.33 σ below that mean (3.81+(2.33×2)=8.47 on 2 MPa scale, 21.2 MPa on 5 MPa scale).

Thus with an SD of 2 MPa, the target strength for 30 MPa concrete may be 30+(1.65×2)=33.3 MPa and concrete may be accepted with a mean strength of 33.3–1.9=31.4 MPa and with 1% of results as low as 33.3–8.5=24.8 MPa. When the SD figure is 5 MPa, the target is 38.3 MPa, the possible mean 33.5 and the 1% limit 17.1 MPa. However, even this depressing possibility does not take account of the fact that the actual standard deviation cannot be determined with any accuracy from as few as three sample results.

Table 6.1 shows a selection of the data in Fig. 6.1 for those who find it easier to absorb in this form.

Similarly, Table 6.2 shows some of the same data as Fig. 6.2.

Two decisions can be taken which will ease the problem:

1. σmust be in the range of 2 to 5 MPa, higher estimates are simply taken as 5, lower estimates as 2.
2. While it is necessary to allow for the possibility of high variability as it affects possible low strengths on the project, the specifier does **not** need to allow for possible high variability causing an assessment which is unfair to the producer (this is for the producer to worry about, see later).

An initial assumption is therefore made that the SD will be 3 MPa unless proved otherwise (as regards setting acceptance criteria but **not** as regards setting a target mean strength). The possible error (at the 10% level will be sufficient) in determining SD must be added to the 3 MPa. Thus if the possible error is ± 1.2 MPa, 4.2 MPa (=3+1.2) has to be used in the assessment. If the actual obtained estimate exceeds 4.2 MPa, the true

SD must exceed 3 MPa. The consequences of this can be attributed to the concrete supplier rather than the sampling system.

Table 6.3 has been constructed on this basis. The SD column is the estimate of σ as 3+(1.28×3) √n (or as 5 MPa if this is lower).

The 'strength margin' is then calculated as $1.65\times\sigma/\sqrt{n}$. This is also the amount the true mean might be below the mean of the *'n'* samples and therefore the amount of additional error from small sample numbers for which an allowance must be made, i.e. the specified strength must be increased by this amount above the value which would be specified in the case of long run projects.

The cost of this additional strength is reckoned as $1 per cubic metre per 1 MPa on the basis that $1 will buy 10 kg of cement which will

Table 6.1 Effect of sample size on accuracy of mean strength estimation

SD		*2*		*3*		*4*		*5*		
1/100 or 1/1000		*2.33*	*3.09*	*2.33*	*3.09*	*2.33*	*3.09*	*2.33*	*3.09*	*P value*
Single sample		*4.66*	*6.18*	*6.99*	*9.27*	*9.32*	*12.36*	*11.65*	*15.45*	*P×SD*
Sample					Extreme limits of sample mean					
Size *n*	√*n*									
2	1.41	3.30	4.37	4.94	6.55	6.59	8.74	8.24	10.92	P×SD
3	1.73	2.69	3.57	4.04	5.35	5.38	7.14	6.73	8.92	√*n*
4	2.00	2.33	3.09	3.50	4.64	4.66	6.18	5.83	7.73	
5	2.24	2.08	2.76	3.13	4.15	4.17	5.53	5.21	6.91	
10	3.16	1.47	1.95	2.21	2.93	2.95	3.91	3.68	4.89	
15	3.87	1.20	1.60	1.80	2.39	2.41	3.19	3.01	3.99	
20	4.47	1.04	1.38	1.56	2.07	2.08	2.76	2.61	3.45	
30	5.48	0.85	1.13	1.28	1.69	1.70	2.26	2.13	2.82	

Table 6.2 Effect of sample size on accuracy of standard deviation estimation

SD		*2*		*3*		*4*		*5*		
4 out of 5 or 50/50 chance		*1.28*	*0.675*	*1.28*	*0.675*	*1.28*	*0.675*	*1.28*	*0.675*	*P value*
Sample				Extreme limits of sample standard deviation						
Size *n*	√2*n*									
2	2.00	1.28	0.68	1.92	1.01	2.56	1.35	3.20	1.69	P×SD
3	2.45	1.05	0.55	1.57	0.83	2.09	1.10	2.61	1.38	√2*n*
4	2.83	0.91	0.48	1.36	0.72	1.81	0.95	2.26	1.19	
5	3.16	0.81	0.43	1.21	0.64	1.62	0.85	2.02	1.07	
10	4.47	0.57	0.30	0.86	0.45	1.14	0.60	1.43	0.75	
15	5.48	0.47	0.25	0.70	0.37	0.93	0.49	1.17	0.62	
20	6.32	0.40	0.21	0.61	0.32	0.81	0.43	1.01	0.53	
30	7.75	0.33	0.17	0.50	0.26	0.66	0.35	0.83	0.44	

increase strength by 1 MPa (the reader may substitute a different figure to suit his own area costs).

The cost of taking an additional sample of concrete (three cylinders and associated slump test, etc.) is taken as \$50 (again the reader may substitute a different figure).

It is possible to reduce the error of assessment by taking an additional sample and thereby saving a certain cost per cubic metre. The second last column of Table 6.3 shows the number of cubic metres needed to enable this saving to exactly equal the cost of the additional sample. The last column of Table 6.3 shows the average number of cubic metres per sample which this would involve.

The numbers in Table 6.3 appear quite practicable. If only a single sample were taken, it would be necessary to specify a strength 8.25 MPa (say, 10 MPa) higher than that really required. If the project involved 20 m^3, it would pay to take a second sample and only increase the strength by 5.74 MPa (say, 5 MPa if strength not very critical). Three samples would definitely justify reducing the margin to 5 MPa and would be taken from, say, 40 m^3 and over. At this level the worst of the problem is over. Adjustments to specified strength of less than 5 MPa are probably impractical and either 5 MPa or no adjustment would be made, according to how critical the concrete was considered to be, when four or more samples were taken. The rate of sampling would also be arbitrary rather than taken from the table, providing that at least four samples in all were taken; a rate of between one sample per 10 m and 1 per 50 m up to 10 samples may be satisfactory and anything up to 1 per 100 m^3 beyond this.

Of course, on projects of the size considered, it is likely that no 28-day results would be obtained until after the concreting was completed. Perhaps on very small projects not even any 7-day results would be obtained prior to completion. There are two different ways of approaching this problem. One, assuming that the concrete is not very critical and that one has reasonable faith in the technical competence of the supplier, is simply to leave it to him having ensured that he is adequately motivated. The motivation would of course be in terms of a cash penalty as discussed elsewhere. This approach may well be combined with a **further** increase of 5 MPa in the strength specified so that concrete accepted with a small penalty would certainly be satisfactory

Where the concrete is too critical for this approach, or where the supplier's competence is in doubt, it will be necessary to have his mix proposals submitted and professionally assessed or to conduct a mix trial (or both).

Certainly it is possible to ensure that small projects are supplied with satisfactory concrete. There is an additional cost involved but at worst this need not exceed \$10 per cubic metre and would rarely involve more

Table 6.3 Cost justification of increased testing frequency

Assumptions:

Sample cost > \$50.00 <<<<< (Different figures can
Cost/MPa >\$1.00 <<<<< be entered here)

SD=Min of 5 or [3+(1.28×3/sqrt (2*n*))] Strength margin=[1.65×(SD/sqrt *(n)*)]

Sample Size *n*	Sqrt *(n)*	Sqrt (2*n*)	SD	Strength margin	Saving over previous	Cumulative to justify	Cumulative per sample
1	1.00	1.41	5.00	8.25			
2	1.41	2.00	4.92	5.74	2.51	19.92	9.96
3	1.73	2.45	4.57	4.35	1.39	36.00	12.00
4	2.00	2.83	4.36	3.60	0.76	66.12	16.53
5	2.24	3.16	4.21	3.11	0.49	103.03	20.61

Assumptions:

Sample cost > $50.00 <<<<< (Different figures can
Cost/MPa >$1.00 <<<<< be entered here)

SD=Min of 5 or [3+(1.28×3/sqrt (2*n*))] Strength margin=[1.65×(SD/sqrt *(n)*)]

Sample Size *n*	Sqrt *(n)*	Sqrt (2*n*)	SD	Strength margin	Saving over previous	Cumulative to justify	Cumulative per sample
6	2.45	3.46	4.11	2.77	0.34	146.11	24.35
7	2.65	3.74	4.03	2.51	0.26	194.87	27.84
8	2.83	4.00	3.96	2.31	0.20	248.96	31.12
9	3.00	4.24	3.91	2.15	0.16	308.04	34.23
10	3.16	4.47	3.86	2.01	0.13	371.88	37.19
11	3.32	4.69	3.82	1.90	0.11	440.24	40.02
12	3.46	4.90	3.78	1.80	0.10	512.92	42.74
13	3.61	5.10	3.75	1.72	0.08	589.77	45.37

Table 6.3 *(Contd)*

SD=Min of 5 or [3+(1.28×3/sqrt (2n))] Strength margin=[1.65×(SD/sqrt (n))]

Sample Size *n*	Sqrt *(n)*	Sqrt (2*n*)	SD	Strength margin	Saving over previous	Cumulative to justify	Cumulative per sample
14	3.74	5.29	3.73	1.64	0.07	670.63	47.90
15	3.87	5.48	3.70	1.58	0.07	755.37	50.36
16	4.00	5.66	3.68	1.52	0.06	843.85	52.74
17	4.12	5.83	3.66	1.46	0.05	935.98	55.06
18	4.24	6.00	3.64	1.42	0.05	1031.64	57.31
19	4.36	6.16	3.62	1.37	0.04	1130.75	59.51
20	4.47	6.32	3.61	1.33	0.04	1233.22	61.66
21	4.58	6.48	3.59	1.29	0.04	1338.96	63.76
22	4.69	6.63	3.58	1.26	0.03	1447.91	65.81
23	4.80	6.78	3.57	1.23	0.03	1559.99	67.83
24	4.90	6.93	3.55	1.20	0.03	1675.14	69.80
25	5.00	7.07	3.54	1.17	0.03	1793.30	71.73
26	5.10	7.21	3.53	1.14	0.03	1914.40	73.63
27	5.20	7.35	3.52	1.12	0.02	2038.40	75.50
28	5.29	7.48	3.51	1.10	0.02	2165.24	77.33
29	5.39	7.62	3.50	1.07	0.02	2294.88	79.13
30	5.48	7.75	3.50	1.05	0.02	2427.26	80.91

than $5 per cubic metre (possibly plus another $200 for assessment of mixes for critical projects).

Before leaving the question of small projects, two other matters require consideration. One is the concrete producer's risk and the other that of the quality of testing.

The producer is in a stronger position to assess his own likely mean strength and variability and should be concerned only with the unjust penalization of satisfactory concrete. He will make his own judgement of the SD he can achieve and set his target strength accordingly. If not confident of his mean strength, he can either conduct trials or make a suitable allowance. However, beyond this, he would be wise to consider that on a small project his SD could be misjudged by up to 2 MPa and his mean strength could also be misjudged by up to 2 MPa. He might therefore allow up to an additional 2+ (2×1.65)=5.3 or, say, 5 MPa above the mean strength he would use for a long running project. Where more than four samples are likely to result, a margin of 2 to 3 MPa may be sufficient to allow if he is reasonably confident of achieving a standard deviation below 4 MPa. The cost of the concrete to the consumer would of course thereby be increased by a further $2 to $5 per cubic metre.

The final problem with small projects supplied without plant control testing is that of testing quality. There is not the opportunity to monitor pair differences and protest at poor performance by a testing laboratory and the author has found that good reputations are not necessarily justified in practice. If the testing is good, the average pair difference will not exceed 1 MPa and no pairs will differ by more than 2 MPa. It is therefore not a large concession to use the higher of each pair as the best estimate of true strength. Where average pair difference exceeds 2 MPa or individual pairs differ by as much as 4 MPa, it may not be sufficient to merely take the highest of each pair (although this is the **minimum** response needed). The supplier (unless of course he also did the testing!) would be within his rights to claim that even the highest of each pair could be as much as the average pair difference below the true strength in such circumstances. The consumer should therefore take care in selecting the persons or organization engaged to undertake the testing, possibly requiring three 28-day cylinders per sample if in a remote location or for other reasons not having confidence that reliable testing services are available.

6.10
SUMMARY

The message of this chapter has been that current methods of specifying concrete tend to be cumbersome, ineffective and deleterious to the current harmony and future progress of the concrete industry. This is in spite of the fact that it is quite easy to write a simple, effective, and readily enforceable specification for the concrete to be supplied (as opposed to the structure as a whole) which will promote a co-operative approach between suppliers and users, as well as encouraging future progress.

The main mental blocks inhibiting such a desirable outcome are:

1. A tendency to specify unnecessary or only partly relevant detail (e.g. aggregate gradings, slump, delivery temperature, pouring delay).
2. A tendency to specify a minimum cement content in the mistaken belief that this necessarily equates to quality.
3. A reluctance to use a cash penalty basis.
4. A lack of knowledge of what really matters (e.g. in such matters as durability, shrinkage, wet properties, batching plants, testing laboratories and control systems).
5. An unjustified faith in the veracity, accuracy and relevance of every single test result.

The most critical requirement, once a mix design has been accepted, is rapidity in the detection and rectification of adverse changes in the quality of concrete currently being supplied. It has been emphasized (section 6.2.5) that this requirement can best be satisfied if it is entirely divorced from considerations of the acceptability of concrete already in the structure. The basis of the specification can be a significant factor in the extent of co-operation established between controller and controlled in the consideration of such early action.

A cash penalty specification example is given below.

Should the value of 'F'_c Provided' fall below the specified F'_c value, then the whole of the concrete represented by the 30 results covered by the analysis shall be initially rejected as not in compliance with the specification. The Contractor shall then select one of the following three courses and shall within 7 days advise the Engineer of his choice:

(a) To remove the concrete from site at his own expense and meet all expenses involved in its replacement.

(b) To carry out at his own expense such further testing, investigation and remedial work as shall be considered necessary by the Engineer to establish beyond reasonable doubt that the whole concrete initially rejected is fully capable of carrying out its intended function in every respect. On no account shall such investigation be confined to pours or sections of pours represented by test results lower than the specified F'_c.

Testing and investigation shall not be confined to compressive strength alone but should also include, at the discretion of the Engineer, tests on permeability and durability under aggressive solutions and any other tests considered relevant.

(c) To propose where accepted by the Engineer that the contract price be decreased by 1% of the delivered concrete cost per 1% of strength deficiency to reflect the reduced value of the concrete provided.

Acceptance of such proposal shall not absolve the Contractor of any responsibility he would otherwise have had in respect of the concrete, but the sum shall be available to wholly or partially offset any expense to claims against the Contractor in respect of future remedial works to the concrete in question.

7
Aggregates for concrete

7.1
FINE AGGREGATE (SAND)

The basic material of a natural fine aggregate is not usually a matter of concern. To some extent this has been 'tested' by the formation process and any weak material broken down. There are some sands (e.g. You Yang sand, a granitic sand from Melbourne, Australia) which are absorptive and may show some moisture movement, but generally the concerns are only with impurities, grading and particle shape.

For too long the approach to sand quality regulation has been to consider what constitutes a 'good' sand, write a specification covering these features and accept or reject submitted sands on this basis. Sands satisfying typical specifications of this type are becoming unobtainable or uneconomic in many parts of the world and it is necessary to devise an alternative procedure.

What matters to the eventual owner of the concrete structure is not the sand itself but the resulting concrete. Essentially this means that a technically satisfactory sand can be defined as one which enables the production of satisfactory concrete. The required concrete properties should be fully specified by the purchaser and the sand properties should be at the discretion of the concrete producer. Possibly the same situation could apply to coarse aggregates, but it is easier to justify with fine aggregates because the effects of a sub-standard fine aggregate tend to be more immediately experienced. Such effects may include retarded set, increased bleeding, excessive air entrainment, poor workability and increased water requirement, the latter in turn leading to increased shrinkage and extra cost.

Seven features of a fine aggregate affect its suitability as a concrete aggregate:

1. Grading.
2. Particle shape and surface texture.
3. Clay/silt/dust content.
4. Chemical impurities.
5. Presence of mechanically weak particles.
6. Water absorption.
7. Mica content.

Any of these, with the possible exception of water absorption, can have such serious effects on concrete as to preclude the use of the aggregate (even under the relaxed and generous criteria proposed by the author). However, this discussion will concentrate on grading, with only brief comments on features 4 to 7. This is partly because the author's views on the other features are not significantly different to those of many others, whereas his treatment of grading is original and has permitted him to make use of sands considered not economically useable by others.

Much of the material in this chapter was presented in a paper entitled *Marginal Sands* presented to an ACI Convention in San Antonio in March 1987.

7.1.1 Grading

Grading can be regarded as the main feature of a sand, and the feature which most frequently stops a particular sand being exploited. However, to a considerable extent, a less than ideal grading can be fully countered by adjusting the mix proportions (i.e. the sand percentage) without additional cost in cement.

The basic concept is to use a smaller amount of a finer sand so as to leave unchanged both the water requirement and the cohesiveness of the mix. In any particular case, the ideal sand percentage is not solely a matter of its grading. Other factors influencing the ideal percentage include cement content, entrained air content, particle shape and grading of the coarse aggregate, and also the intended use of the concrete. As explained in Chapter 3, these factors lead to the selection of a suitable MSF and thence to a suitable combined specific surface of the coarse and fine aggregates. This allows the direct calculation of the required sand percentage from the modified specific surface (see later) of the sand. **This process assumes that the actual grading of the sand will only influence the percentage of it to be used and have no other influence on concrete properties.** While this is the case over a wide range, there must be limits to its applicability. It is necessary to be very clear where the limits are and what happens if they are exceeded.

A sand reaches the **upper limit of coarseness** when there is insufficient paste (cement, water and entrained air) in the mortar to provide adequate lubrication. This occurs not so much due to the coarser sand requiring more paste per unit quantity of sand, but rather because more sand must be used to provide the desired surface area if it is coarser. If the sand quantity is not increased, the overall mix will be too harsh, and will segregate unless of very low slump. If it is increased beyond the limit, the water requirement rises to provide the required total paste volume required. Strength will be reduced, the concrete will almost certainly bleed severely, and workability will suffer in a different way, i.e. it will have unsatisfactory mortar quality rather than an inadequate amount of mortar. A comprehensive mathematical treatment of this problem is given by Dewar in his latest book (Dewar, 1999) but here we will deal only with a few rules of thumb. What is important is that users should recognize the problem when they encounter it. As noted above, this will not occur at a particular sand percentage for all mixes but will depend on several other factors. Some rules of thumb to indicate when the problem should be considered are:

1. Sand percentages in the range of 50% of total aggregates (in low cement mixes) to 65% (in high cement mixes) (very rough guide).
2. Solid volume of sand exceeding about five times the solid volume of cementitious material. With normal sand and cement this can be taken as a sand/cement ratio of about 4 by weight. When fly ash or very heavy or light sands are involved, the volume figure applies. This guide is still not invariably accurate because the limit is affected by the particle shape and grading of both the sand and coarse aggregate and by the use of air entrainment.
3. From a different viewpoint, the problem may arise when the FM (fineness modulus) of the sand exceeds 3.0 in low cement content mixes or 3.5 in high cement content mixes. In Conad specific surface terms the danger signals may be around 40 for high cement contents and 45 for low cement contents.

The **fine limit for a sand** is reached when a further reduction in sand proportion will leave insufficient mortar (sand plus cement paste) to provide adequate lubrication to the coarse aggregate. With a very fine sand it is possible to get quite close to using a cubic foot of coarse aggregate by loose volume in a cubic foot of concrete and the shape and grading of the coarse aggregate makes a substantial difference to where the limit is. The limit will certainly be close, however, when the coarse aggregate approaches 60% by solid volume of the total concrete. Again from the other point of view, the problem is likely to arise with sands of FM around 1.5 (with a high cement content) to 1.8 (with a low cement content) or, in Conad SS (specific surface) terms, in excess of 90 with any cement content. It is also possible that a high cement/sand ratio is intrinsically undesirable in the same way that a heavily oversanded mix is undesirable (e.g. higher shrinkage?). A sand weight less than the weight of cementitious materials should be viewed with suspicion and avoided if possible.

The important point in **coping with extreme sand gradings** is rarely the establishment of the exact limit, rather it is the fact that within these quite wide limits, grading is not the problem that most typical specifications would suggest. It is of course necessary to accurately determine what proportion of sand should be used in each particular case and this is the main strength of the method of mix design evolved by the author.

A recent example of the coarse limit was encountered in Indonesia. The local sand on occasions had less than 3% passing a 300 micron sieve. Its FM was only of the order of 3.0, which did not seem an excessively high figure. However, its SS of 40 to 42 was clearly excessively low. Increasing the proportion of this sand did not solve the problem, which was excessive bleeding. Eventually a proportion of finer sand had to be introduced, even though not readily available. There have been very coarse sands in Singapore and in Australia requiring 48 to 55% of sand but these have all occurred when relatively high cement contents were required. In an extreme case, where the sand is very coarse and only a low strength and therefore a low cement content is required, the following possibilities should be considered:

1. Use of a small proportion of a second fine sand (even if quite expensive).
2. Use of a small proportion of crusher fines with a high ‘fines’ content.
3. Use of fly ash, which has 37% greater volume than an equal weight of cement. In an area where fly ash is inexpensive, more might be used than strictly necessary for strength.
4. Use of air entrainment (as valuable, volume for volume, as cement for this purpose).
5. If no alternative is less expensive, the use of more cement than necessary on strength grounds would certainly solve the problem since it both reduces the sand percentage required for a given MSF and provides more paste to fill the sand voids.

Extreme testing of the fine limit has also occurred. In 1956 (Day, 1959) a case was encountered where the sand percentage calculated by the author’s system came to 15% (virtually all the sand passed the 300 micron [No. 50 ASTM] sieve). It proved possible to obtain a 7 mm (0.25 in) single sized crushed rock and the concrete was made with 10% of this material and 15% of sand (the balance being 75% of an almost single sized 20 mm [0.75 in] crushed rock).

During the early development of the system (in the early 1950s in England) sand percentages of 22 to 23% were used but, although the sand was purchased as ‘plastering sand’ rather than ‘concreting sand’, this was an example of the use of a very low ‘MSF’ on earth dry concrete rather than the use of a very fine sand. It should always be possible to use a proportion of crushed fines (choosing a coarse variety) when the natural sand is too fine for use alone. However, the particle shape of the crushed fines will increase water requirement, and therefore increase cement requirement, at least somewhat.

7.1.2
The economics of selecting sands

It should also be realized that cement content is not the only criterion of cost. There is often a quite wide difference between the price of sand and that of coarse aggregate. This can occur in either direction, but where sand is more expensive than coarse aggregate, use will normally be made of a proportion of crusher fines. Where sand is all cheap and there is a choice, the coarsest available sand will be selected to maximize sand proportion.

The author has always been conditioned to think that higher cement content mixes were necessarily more expensive and therefore that pump mixes, which usually contain more sand and therefore need more water and cement, were more expensive than 'structural mixes' (jargon meaning mixes quite useable with skip placing but not pumpable). In Singapore he found that sand was so much cheaper than coarse aggregate that the sandier mix, in spite of the extra cement, was less expensive (or would have been except for its high clay content). The natural reaction to this is to use the pump mix even if it is not to be pumped, unless low shrinkage is an essential.

Where a coarse and a fine sand are combined in mixes, their relative proportions require careful logic. The assumption is that there is no such thing as an ideal sand grading so that a fairly wide range of relative proportions may give similar concrete quality. The relative proportions will therefore be biased to one extreme or the other of this range according to economic considerations. Surprisingly the relative cost of the two sands makes very little difference because increasing the proportion of fine sand simply results in using less of the sand combination and more coarse aggregate, so very little extra fine sand is used. What matters is the relative cost of the coarsest sand and the coarse aggregate. If coarse aggregate is more expensive then the minimum amount of fine sand will be used to give the greatest total sand quantity.

Two formulas to give a guide to the values of combined SS which might be selected when combining two sands are:

$$\text{Maximum SS} = 66 - 0.02c - 0.024f$$

$$\text{Minimum SS} = 55 - 0.02c - 0.024f$$

where SS=the author's modified specific surface for the combined sands (see below)
c=the cement content in kg/m
f=the fly ash content.

It is not suggested that these values are actually limits. The reader should feel free to work outside them if driven by circumstances but they give some guidance to conservative values for inexperienced users. Another rule of thumb is that sand should not exceed four times the mass of cement (or cementitious materials).

7.1.3
Grading indices

There has always been an attraction in representing a sand grading by a single number which will describe its performance in concrete. For example this would avoid the problem of sand gradings straying into two different zones and would permit adjustment of sand percentages on a continuous scale rather than three large steps.

The original and perhaps most widely known and used grading index is the Fineness Modulus (FM). This is the sum of the cumulative percentages retained on each sieve from 150 micron upwards. This index is used in the ACI mix design system to adjust for sand fineness. However, it is used to indicate adjustment steps rather than give continuous adjustment in a formula.

The Specific Surface (SS) is the surface area per unit weight (per unit solid volume would be preferable but is not usually used). This is difficult to measure directly but may be estimated from measured or assumed values of SS for each individual sieve fraction in a manner similar to FM. If dealing with perfect spheres, halving the diameter exactly doubles the surface area per unit weight. This simple assumption gives a reasonable index for aggregate proportioning but what is really required is a prediction of water requirement. The greater the surface area, the higher the water requirement, but the effect of the finer sieve fractions on water requirement is not as great as surface area suggests (Day, 1959).

Table 7.1 (Popovics, 1982) sets out 10 lists of factors for the numerical characterization of individual sieve fractions. The author's modified specific surface has been added to form an 11th column (the origin of the author's values has been explained in section 3.7). Some of these factors have been used as a basis for selecting the relative proportions of fine and coarse aggregates, some to calculate water requirement, and some (including the author's) for both of these purposes.

Popovics (1992) also sets out 26 formulas, 12 of which were originated by himself, for the calculation of water requirement. Some of the formulas are quite complex and tedious to evaluate, but this would be no disadvantage if the formula were included as part of a computer program. However, only a dedicated research worker could devote the time and effort which would be involved in examining the relative merits of the 26, or even the 12, formulas over a range of actual mix data.

No doubt each proponent of a system (including the author) considers his own system quite simple to use.

It is not proposed to examine all the alternatives in the current volume

Table 7.1 Various proposals for sand grading indices

Limits of size fraction												
Sieve	*d (mm)*	*d_e' (mm)*	*d_e' (in)*	*s_e' (m^2/m^2)*	*m*	*e*	λ	ρ	*A*	*i*	*f_s*	*Modified SS*
3–1½ in	75–37.5	56.25	2.21	106.7	9.56	0.638	9.33	2.53	0.020	0.06	−2.5	
(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(−100)			1
1½–¾ in	37.5–19.0	28.25	1.11	212.4	8.56	1.01	11.34	3.57	0.035	0.12	−2.0	
(50.2)	(50.2)	(199)	(89.5)	(158)	(122)	(141)	(175)	(200)	(−80)			2
¾–⅜ in	19.0–9.5	14.25	0.561	421.4	7.58	1.59	13.95	5.03	0.055	0.19	−1.0	
(25.3)	(25.3)	(395)	(79.3)	(250)	(150)	(199)	(275)	(317)	(−140)			4
⅜ in-No. 4	9.5–4.75	7.12	0.280	842.7	6.58	2.53	17.49	7.12	0.075	0.27	1.0	
(12.7)	(12.7)	(790)	(68.2)	(396)	(187)	(281)	(375)	(450)	(40)			8
No. 4-No. 8	4.75–2.36	3.56	0.140	1685	5.58	4.02	22.3	10.07	0.096	0.39	4.0	

Limits of size fraction												
Sieve	*d (mm)*	*d_e' (mm)*	*d_e' (in)*	*s_e' (m^2/m^2)*	*m*	*e*	λ	ρ	*A*	*i*	*f_s*	*Modified SS*
(6.32)	(6.32)	(1580)	(58.4)	(631)	(239)	(398)	(480)	(650)	(160)			15
No. 8-No. 16	2.36–1.18	1.77	0.0697	3390	4.57	6.40	29.2	14.28	0.116	0.55	7.0	
(3.15)	(3.15)	(3178)	(47.8)	(1003)	(313)	(564)	(580)	(917)	(280)			27
No. 16-No. 30	1.18–0.60	0.89	0.0350	6742	3.58	10.10	39.0	20.14	0.160	0.70	9.0	
(1.58)	(1.58)	(6321)	(37.4)	(1584)	(418)	(796)	(800)	(1167)	(360)			39
No. 30-No. 50	0.60–0.30	0.45	0.0177	13333	2.60	15.94	53.5	28.32	0.24	0.75	9.0	
(0.80)	(0.80)	(12500)	(27.2)	(2500)	(573)	(1119)	(1200)	(1250)	(360)			58
No. 50-No. 100	0.30–0.15	0.225	0.0089	26667	1.60	25.30	76.8	40.06	0.35	0.79	7.0	
(0.40)	(0.40)	(25000)	(16.7)	(3969)	(819)	(1583)	(1750)	(1317)	(280)			81
No. 100-pan	0.15–0	0.075	0.0030	?	0	–	?	?	?	1.0	2.0	
(0.13)	(0.13)							(1667)	(80)			105

Values in parentheses are presented relative to the numerical characteristics of size fractions 3–1½ in (75–37.5 mm). *d*=average particle size, mm; d_e=average particle size, in; s=specific surface (Edwards, 1918); m=fineness modulus; e=water requirement (Bolomey, 1947); λ=distribution number (Solvey, 1949); ρ=stiffening coefficient (Leviant, 1966); *A*=A value (Kluge, 1949); *i*=*i* index (Faury, 1958); f_s=surface index (Murdock, 1960).

but, in view of the widespread use of fineness modulus, some attention should be given to it.

Table 7.2 is given in two of Popovics' books (Popovics, 1982, 1992) and is derived from Walker and Bartel (1947). This table provides an optimum value for the fineness modulus of the combined coarse and fine aggregates.

Table 7.2 is valid for natural sand and rounded gravel having voids of 35%. A value of 0.1 should be subtracted from the tabulated values for each 5% increase in voids. For air entrained concretes, add 0.1 to the tabulated values. The values are for 25 to 50 mm slump concrete, subtract 0.25 for 100 mm slump and for zero slump add 0.25.

Equation (7.1), also from Popovics (1982), gives the water required to provide a 100 mm slump in units of pounds per cubic yard (divide by 1.685 to convert to litres per cubic metre).

$$\text{Water requirement} = c\{0.1 + 0.032[(2^m - 60)^2 + 6570]/(c - 100)\} \tag{7.1}$$

where m=fineness modulus of combined aggregates
c=cement content in lb/yd^3 (= kg/m^3×1.685).

See section 3.10 for Popovics' method of modifying the water requirement for the effects of different slumps, concrete temperatures, and maximum size of aggregate (the equation assumes a 30 mm maximum size).

Murdock (1960) and Hughes (1954) also introduce a term for angularity of grains. This clearly influences water requirement but cannot conveniently be used to give an adjustment to these values (section 7.1.5).

Table 7.2 Optimum values for fineness modulus

Maximum size of aggregate		*Weight of cement*								
No.	*mm*	*280*	*375*	*470*	*565*	*660*	*750*	*850*	*950*	*(lb/yd^3)*
170	*225*	*280*	*335*	*390*	*445*	*500*	*560*	*(kg/m^3)*		
No. 30	0.60	1.4	1.5	1.6	1.7	1.8	1.9	1.9	2.0	
No. 16	1.18	1.9	2.0	2.2	2.3	2.4	2.5	2.6	2.7	
No. 8	2.36	2.5	2.6	2.8	2.9	3.0	3.2	3.3	3.4	
No. 4	4.75	3.1	3.3	3.4	3.6	3.8	3.9	4.1	4.2	
⅜ in.	9.5	3.9	4.1	4.2	4.4	4.6	4.7	4.9	5.0	
½ in.	12.5	4.1	4.4	4.6	4.7	4.9	5.0	5.2	5.3	
¾ in.	19.0	4.6	4.8	5.0	5.2	5.4	5.5	5.7	5.8	
1 in.	25.0	4.9	5.2	5.4	5.5	5.7	5.8	6.0	6.1	
1½ in.	37.5	5.4	5.6	5.8	6.0	6.1	6.3	6.5	6.6	
2 in.	50.0	5.7	5.9	6.1	6.3	6.5	6.6	6.8	7.0	
3 in.	75.0	6.2	6.4	6.6	6.8	7.0	7.1	7.3	7.4	

The concept of specific surface mix design is that an appropriate specific surface for the overall grading be selected allowing for the intended use. A low workability high strength concrete (e.g. for heavy vibration into precast products) would require a low specific surface to reduce water requirement but a high slump mix would require a higher specific surface to avoid segregation (Table 3.1).

The sand percentage is then calculated to provide the required specific surface. The method has produced useable concrete mixes with sand percentages varying from 15 to 55% of total aggregates in particular circumstances but 25 to 50% of sand is a fairly safe range.

The grading zones do not overlap because the 0.6 mm sieve is taken as the criterion. However, looking at the SS values or even the FM values in Table 7.3, it is clear that the **properties of the sands** in different zones are likely to overlap. This can be avoided by defining a Zone 1 sand as a sand having an SS of 38.85 (or, say, 40, or 34 to 44) with Zone 2 being say 48 or 44 to 52, Zone 3 being 56 or 52 to 60 and Zone 4 being 64 or 60 to 70.

It has been contended that, to a very large extent, only the surface area and not the detailed grading of a sand is of importance. This is not completely true in all cases and the following exceptions are noted.

1. The existence of gaps in the grading (i.e. the absence of some sieve fractions) either between the sand and the coarse aggregate or within the sand grading itself can give rise to:

(a) segregation at medium to high slumps.
(b) severe bleeding.
(c) concrete which will not pump.
(d) improved workability under vibration for low slump concrete.

2. Sands which are almost single-sized can give rise to poor workability through particle interference.
3. A proportion of large particles in an otherwise predominantly fine sand can cause problems through interfering with the packing of the coarse aggregate.

It is emphasized that these are rare exceptions, not glaring deficiencies in the general assumption.

7.1.4
Air entrainment

The use of admixtures can be of considerable assistance in solving grading problems. Air entrainment is well known to have the capacity to inhibit bleeding and to assist in overcoming problems of harshness with very coarse or very angular fine aggregates. An unusual use for air entrainment is worth recounting. The mix was specified not to contain any silicious aggregates (including natural sand) because it was to be

Table 7.3 Inter-relationship of old UK grading zones, specific surface and fineness modulus

Sieve size (mm)	*Grading requirements % passing*					
Zone1	*Zone 2*	*Zone 3*	*Zone 4*	*ASTM C33–71A*	*AS1465 1984*	
10.000	100	100	100	100	100	100
4.750	90–100	90–100	90–100	95–100	95–100	90–100
2.360	60–95	75–100	85–100	95–100	80–100	60–100
1.180	30–70	55–90	75–100	90–100	50–85	30–100
0.600	15–34	35–59	60–79	80–100	25–60	15–100
0.300	5–20	8–30	12–40	15–30	10–30	5–50
0.150	0–10	0–10	0–10	0–10	2–10	0–15
0.075	–	–	–	–	–	0–5
SS	29.40–48.31	38.54–58.31	48.00–66.00	56.00–72.00	38.00–57.90	29.40–73.10
FM	4.00–2.91	3.37–2.11	3.00–2.00	2.00–1.00	3.00–2.15	4.00–1.35
Avg. SS	38.85	48.42	52.06	63.70	47.91	41.75
Avg. FM	3.35	2.74	2.44	1.82	2.76	3.17

used in the base of a furnace. This left, as the only available fine aggregate, a crusher dust with almost 20% passing a 150 μm sieve.

The author's system correctly predicted the proportion of this material which would make reasonable concrete and correctly predicted its water requirement. However, especially since a high minimum cement content was also specified, the mix was very sticky and difficult to handle from skips, etc., even though it compacted quite well. These days a superplasticizing admixture and a higher slump would probably be used, but this mix was encountered before such admixtures were readily available in Australia and in any case would have represented extra cost since the minimum allowable cement content already provided excess strength. Instead, an air-entraining agent was used and did produce a substantial improvement. It is interesting that air entrainment can both increase the cohesion of a harsh mix and lubricate a sticky mix since these are virtually positive and negative effects on the same property of the concrete.

7.1.5 Particle shape

We have seen that a fine sand has a higher water requirement, but over a wide range it can simply be used in smaller proportion to give a normal water requirement. An angular sand, or especially crusher fines, also has a higher water requirement for a given grading. However, this does not justify a reduction in its proportion (it may even justify a small increase, thus further increasing water requirement, but this is too fine a piece of tuning to incorporate into a relatively simple system). There is therefore an inevitable increase in water requirement of the mix, and therefore an additional cost in cement when an angular fine aggregate is used. However, the angular fines may be very cheap, or otherwise be the least costly alternative in overall concrete cost, or may be technically essential.

Examples of where crusher fines may be justified are:

1. Where natural sand is very expensive (owing to long haulage distance or otherwise).
2. Where the natural sand is so fine that it would have to be used in a mix of more than the otherwise desirable surface area.
3. Where the natural sand has a high clay content and it is cheaper to accept the higher cement requirement than to wash the natural sand.
4. Where the natural sand is so coarse that the crusher dust is necessary as a filler.

Apart from the above economic considerations, there may also be technical reasons for using or not using the crusher fines:

1. A coarse grade of crusher fines may be needed to fill the gap between the top of a fine sand grading and the bottom of the coarse aggregate grading. This may be essential to provide pumpability or to avoid segregation where high slump is necessary.
2. It should be remembered that a higher water requirement is not purely an economic disadvantage. It also gives increased shrinkage and so may be unacceptable for some purposes even if it is the most economical way of providing the required strength.
3. There will normally be a distinct difference in colour between a crusher fines mix and a natural sand mix. One or other may therefore be architecturally either preferred or rejected for exposed architectural concrete.
4. There may be a substantial difference one way or the other (depending on actual gradings) in bleeding characteristics which may have a substantial effect on surface appearance (coarse crusher fines being particularly susceptible to heavy bleeding but fine dust inhibiting it).

It is quite frequently a satisfactory arrangement to use a combination of crusher fines and natural sand. The author has formed an opinion (rather than definitely established) that there tends to be more benefit than expected from such a combination (section 3.14).

Apart from gradings often fitting well together (crusher fines tending to be deficient in middle sizes and natural sand to have an excess) a small proportion of a fine, rounded, natural sand appears to have a disproportionate effect on reducing the ill effects of angularity. Also the first 2% or so by weight of silt in a fine aggregate appears not to be deleterious so that halving the amount of a silty sand will more than halve the water increasing effect of its silt.

Air entrainment and crusher fines should be approached with a little more caution. Trial mixes will very clearly show a big advantage for air entrainment. However, stone dust inhibits air entrainment and, if its proportion varies, can result in a high variability of air content which may be unacceptable in practice. Note that fly ash (pfa) gives a similar effect on workability to that of air entrainment but is not susceptible to being inhibited or varied in its effect (other than its own inhibiting effect on air content, which is heavily dependent upon its carbon content, as measured by its loss on ignition). So crusher fines may be more acceptable in mixes containing fly ash.

The extent of the effect of particle shape can be as much as 9% water increase with the fine aggregate being entirely of badly shaped (but still well graded) crusher fines. However, 7% increase is more normal for crusher fines and a badly shaped natural sand may cause as much as 3 or 4% increase. Badly shaped natural sand usually comes from glacially formed pit deposits rather than rivers or beaches. (Note that sand flow cone experimenters claim to have found fine aggregates which increase water demand by as much as 20%.)

7.1.6
Clay, silt or dust content

The author's system does not provide for the incorporation of the effect of material finer than a 75 micron (200 mesh) on his specific surface (it is counted the same as material passing the 150 micron (100 mesh) sieve and retained on the 75 micron sieve). This is for the same reason that the effect of angular grains is not incorporated, i.e. it does affect water requirement but it does not justify an offsetting reduction in the proportion of the fine aggregate. A subsidiary reason is that the increase is not solely dependent on the weight of such material but also on its character.

It is arguable whether the 75 micron (200 mesh) sieve is worthwhile for checking fine aggregates for concrete. Certainly it is important how much of such material there is in the aggregate, but the percentage by weight gives only half the story. Some materials, such as the montmorillonite (smectite) clay in sand extracted in Singapore, can have three times as much effect per unit of weight as other fines such as fine crusher dust also passing the 75 micron sieve.

The definitive test for this property is undoubtedly the French 'Valeur de Bleu' (Bertrandy, 1982). This test involves titrating wash water from the fines with methylene blue, which is essentially a dye composed of molecules which are single particles of absolutely standard size. The dye molecules are attracted to the surface of the fines and none remain in suspension so long as any surface area of fines remains exposed. It is possible to calculate the surface area of superfine material from the amount of the dye which has to be added before any remains in solution. This point is determined by placing one drop of the solution on a standard white blotting paper. As soon as any dye remains in solution, a faint blue halo surrounds the central muddy spot. This test is a French (tentative?) standard and is fairly easy to do in a chemical laboratory (i.e. a laboratory mechanical stirrer and a burette are needed). However, there is no point in incorporating it into the author's system because the test result would rarely be available when needed.

The alternative is very simple indeed and is the standard **Field Settling Test**. Both the process of obtaining it and the use of this figure (a percentage of clay by volume when the fine aggregate is shaken up with salt solution or sodium hydroxide in a measuring cylinder and allowed to settle) are very crude indeed but it nevertheless greatly improves the accuracy of the water prediction. The assumption made is that every

100 kg of the fine aggregate will require an extra 0.225 litres of water for each 1% by which its silt content by volume exceeds 6%. For example, 600 kg/m^3 of fine aggregate with 8% silt content will require 6×0. 225× (8–6)=2.7 litres of extra water.

When the silt correction originated in Singapore, the sand was very coarse, requiring over 900 kg/m^3 and the silt percentage was over 25% by the settling test on occasions (9% by weight). This meant that over 20 litres of additional water was required, sometimes almost 30 litres. The figure was initially derived by taking a 44 gallon drum of the dirty sand, inserting a running hose to the bottom and overflow rinsing until the water ran clear. A repeat of the original trial mix before washing showed a water reduction of almost 30 litres. No excuse is offered for the blatant crudity of this 'clay correction' because for several years now it has given good results on many different sands in Australia and SE Asia.

The additional water figure can be translated into an additional cement figure when the required w/c ratio is known. This gives a fairly precise figure for the cash value of washing the sand and so a basis for deciding whether or not to set up a sand washing plant. However, it is often better to counteract the effect of the clay by using a superplasticizing admixture than by accepting it and using additional cement. This view has been confirmed and quantified in the laboratory by Tam (1982).

A final point on the subject of fines contents is that crusher fines dust can give a distinct (but not large) strength increase at a given water/cement ratio. In fact this is not surprising because Alexander has shown that siliceous stone dust can have pozzolanic properties if it is ground sufficiently fine. Also calcareous stone dust (e.g. limestone) will react chemically. However, the author's practice is to use the settling test to allow for the extra water requirement of the fine dust but to neglect the possible strength increase.

7.1.7
Other impurities

Chemical impurities

The question of more exotic chemical impurities is left to others but the two questions of salt and organic impurity must be addressed.

There is an extensive literature on chloride contents and their capacity to promote the corrosion of reinforcing steel. Beach sand is liable to have very high salt levels owing to the deposition of salt by evaporation. Sand dredged from the sea may be less of a problem but without washing with fresh water may still exceed a fully safe level. Salts can also cause efflorescence and higher shrinkage and affect setting and hardening rates.

Organic impurity is quite frequently encountered in pit sands. The author's practice is to combine the colour test (BS 812,1960) for organic impurity with the settling test for clay content by using sodium hydroxide instead of the specified salt solution for the latter test. It is to be noted that the use of pure water will give a different result with the clay taking longer to settle and giving a higher reading. The important point to realize is that the test only establishes whether organic impurity is present and not whether it is deleterious. The colour test can be failed due to the presence of a few pieces of organic matter such as small twigs or other vegetation which are too few and too localized to have any significant effect on strength (but could produce a visual defect on a surface).

Sands failing the colour test should then be tested for setting time and initial strength development. If they are satisfactory in these respects, it is unlikely that there will be any long term problems (although another problem encountered has been of sands which automatically entrain air due to natural lignin).

The usual effect of impurity (if there is any effect) is of retarding or preventing chemical set. If there is no ill-effect on strength up to 28 days the sand is satisfactory. There may be a strength reduction at 1 to 7 days but no loss of strength at 28 days, which may or may not be satisfactory for particular applications. There may be implications, with early strength loss, of setting time extension and consequent surface finishing problems for slabs.

For organic impurity evaluation, comparative mortar cubes should have the same w/c ratio, not the same workability.

Natural impurities are not the only kind and there have been instances of accidental contamination, especially with sugar. One example was of a barge used to transport sand after transporting a load of bulk raw sugar; one result of this was to cause a large floor slab in a multistorey building not to set for several days. It takes very little sugar to cause a problem, e.g. the author has experienced a concrete strength problem later traced to employees emptying the dregs of their morning tea onto the sand pile of a small manual batching plant.

Rivalling the frequency of occurrence of all the above combined in the author's experience has been the frequency of multiple dosing of retarding admixtures. This is outside the scope of the book but it has provided more examples of concrete which has eventually proved quite satisfactory after taking several days to set. The message here is not to panic too early. If a sample sets after being in boiling water overnight (inside a plastic bag, of course) then the concrete in the structure will set eventually. The question is whether it will develop serious settlement cracks in the interim due to prolonged bleeding, or to water soaking into formwork or escaping at joints which are not watertight. It is certainly important to cover the concrete with plastic sheeting or wet hessian in order to stop it drying out.

Weak particles and high water absorption

These are not common in river sands but can be encountered in pit sands. Except in very high strength concrete, or concrete required to have wear resistance or frost resistance, the direct effect on concrete strength is not likely to be a problem. Degrading during mixing, increasing fines content and therefore increasing water requirement, is possible (but more likely in a coarse aggregate). A high water absorption may indicate an increased drying shrinkage and could also indicate a reduced freeze/thaw resistance.

Mica content

Except possibly in very high strength concrete, there does not appear to be a problem with moderate amounts (less than 5%) of mica directly weakening the mortar. Rather the problem appears to be an increased water requirement. Probably the mica which can be seen does not do much harm but it may indicate the presence of finer mica particles which will have much more influence on water requirement and possibly significantly increase the moisture movement tendency of the mortar.

Mica is usually detected visually but can be extracted by the use of a liquid heavier than mica but lighter than sand. However, its effect on the water requirement of mortar and therefore its strength, **this time at fixed workability,** is probably easier to determine and more relevant.

7.1.8
Example of implementation of specific surface concept (Table 7.4)

Example of use:

$$
\begin{aligned}
\text{Say required MSF} &= 25 \\
\text{Air entrainment} &= 4\% && (= a) \\
\text{Coarse aggregate SS} &= 3.1 && (= \text{SS coarse}) \\
\text{Cement content} &= 400\ \text{kg/m}^3 && (= c) \\
\text{Then required SS} &= \text{MSF} - (a-1)/4 - 0.02c + 6 \\
&= 25 - 0.75 - 8 + 6 \\
&= 22.25
\end{aligned}
$$

$$
\text{Therefore required sand percentage} = \frac{(\text{Required SS} - \text{SS coarse})}{(\text{SS sand} - \text{SS coarse})} \times 100
$$

$$
= \frac{(22.25 - 3.1)}{(51.15 - 3.1)} \times 100
$$

$$
= \frac{19.15 \times 100}{48.05} = 40\ \%
$$

7.1.9 Summary

A very wide range of sand gradings can be used with no economic penalty to make a wide range of satisfactory concretes in a predictable manner.

Table 7.4 Calculation of specific surface from sieve analysis

Sieve size		*Example grading*		*Surface modulus*	*Authors modified specific surface*
Metric	*ASTM*	*passing %*	*retained %*		
19.00	3/4	100	0	0×1=0	0×2=0
9.50	3/8	100	0	0×2=0	0×4=0
4.75	4	95	5	5×4=20	5×8=40
2.36	8	85	10	10×8=80	10×16=160
1.18	16	75	10	10×16=160	10×27=270
0.600	30	60	15	15×32=480	15×39=585
0.300	50	20	40	40×64=2560	40×58=2320
0.150	100	5	15	15×128=1920	15×81=1215
0.075	200	1	4	5×256=1280	5×105=525
Passing		1	1		
				SUM=6500	SUM=5115
				SS=6500/100=65.00	SS=5115/100=51.15

High silt content and poor particle shapes do carry an economic penalty in the form of higher water (and therefore cement) requirement. However, the situation is numerically predictable from simple tests.

Only brief advice is given on other sand quality features with no numerical relationships. It is pointed out that the author's mix design system automatically provides a standard of reference against which mix performance can be judged. Whenever mix performance is below this standard it is desirable to establish the cause. Important clues are whether any strength reduction is caused by increased water content and whether it is more severe at early ages.

It should be noted that the above makes no reference to unit weight testing to determine percentage voids or to the use of the sand flow test (section 3.14). These tests give valuable information on the suitability of sands and on the optimum combination of two or more sands. The author's experience is that most clients are unwilling to carry out such tests and therefore the system has been devised to operate without them. However, the reader should not conclude that such testing is not well worthwhile, especially for an organization's central laboratory or a commercial laboratory offering consulting services.

7.2 COARSE AGGREGATE

The properties of a coarse aggregate depend on the properties of the basic rock, upon the crushing process (if crushed) and upon the subsequent treatment of the aggregate in terms of separation into fractions, segregation and contamination.

Most rock has an adequate basic strength for use in most grades of concrete. Even manufactured and naturally occurring lightweight aggregates, which can be readily crushed under a shoe heel, are used to make concrete with an average strength up to 40 MPa (although they do require a higher cement content than dense aggregates). Exceptions to this are some sandstones, shales and limestones (although other limestones are very strong and amongst the best aggregates for many purposes). A different type of exception is that use involving wear and impact resistance can require a more stringent selection of rock type.

Generally however, the stability of a coarse aggregate is more important than its strength. Rock which exhibits moisture movement (swelling and shrinking) will add to concrete shrinkage. Again sandstone tends to be amongst the offenders, but some basalts will also display moisture movement and some breccias or conglomerates may be quite strong mechanically and yet literally fall part under a few cycles of wetting and drying.

Rock from an untried source must be tested for susceptibility to alkaliaggregate reaction. Whilst comparatively rare, this reaction produces such catastrophic results that its occurrence should not be risked without at least a petrographic report. There is a rapid chemical test for reactivity but it is not very reliable.

Another important feature of a coarse aggregate is its bond characteristics (especially in high strength concrete and where flexural or tensile strength is of special importance). This is a composite effect of its chemical nature, its surface roughness, its particle shape, its absorption, and its cleanliness. As an example of the importance of this feature we can use the author's experience with two different basalts in Melbourne. One of these is superior to the other on every tested feature; it is stronger, has a higher elastic modulus, is denser, has less moisture movement and a higher abrasion resistance. However, the other aggregate was better able to produce concrete of average strength over 60 MPa. We assume that this was due to the first aggregate being so dense and impermeable that cement paste had difficulty in bonding to it. It is interesting to note that the subsequent introduction of silica fume appears to have reversed this situation, confirming the effect of silica fume on bond.

The particle shape of the aggregate is influenced by the crushing process. The stone type does have a distinct influence, some stones being more liable to splinter into sharp fragments and/or to produce a larger amount of dust than others. However, the crushing process also has a large influence. Cone crushers are perhaps the most efficient and economical type of crusher but they do not produce as good a particle shape as a hammer mill. Other influencing factors are the reduction ratio (a large reduction in a single stage tending to produce a worse shape) and the continuity of feeding (choke feeding giving a better shape).

The effect of a poor particle shape (flaky and elongated) is to require a higher fine aggregate and water content (and therefore a higher cement content) for a given workability and strength. The best measure of this is the ‘angularity number’, being the percentage voids minus 33. Oddly enough, Kaplan’s work (1958) on the subject suggests that the sharpness of the edges and corners tends to make more difference to this parameter than flakiness and elongation.

The question of particle shape must include considering the relative merits of crushed rock and rounded river gravel. Gravels are often reputed to give inferior results, particularly for high strength concrete. There is no denying that this is true for a given w/c ratio and that it is true generally where tensile or flexural strength is concerned. However, in terms of compressive strength, with equal cement content and equal ease of placing (reduced fine aggregate content) then rounded gravel may give as good or better results, depending on the particular use. Forty years ago, the author made concrete of 85 to 90 MPa from London area gravel (which is one of the gravels which have been claimed to give inferior results for high strength concrete). Gravels tend to have been adequately tested by the formation process as regards weaker particles and moisture movement susceptibility. However, this provides no security against alkali-aggregate reactivity and any coatings on pit gravels in particular should be regarded with suspicion.

The subject of coatings on coarse aggregate is worth consideration. Generally if the coating is removed during the mixing process (and assuming it to be chemically inactive) it is not likely to cause a severe problem. Very fine material will merely add to the water requirement in the same way as fine aggregate silt. This will increase water requirement but, unless excessive, should cause only a small strength depression. However, if a coating remains intact after the concrete is in place, a substantial effect on strength and durability can occur through loss of bond. The amount of fine material adhering to coarse aggregate is often substantially affected by the weather, with more material adhering during wet periods. This effect should be considered when looking for causes of strength variations in concrete.

The ideal maximum size for a coarse aggregate has usually been assumed to be 40 mm or 20 mm (1.5 in or 0.75 in) according to the size of section and the reinforcement spacing. Of recent years there has been a worldwide trend to higher concrete strengths and work done many years ago in USA (Blick *et al.,* 1974) is gradually being rediscovered the hard way in many other places. This work showed that the optimum size of aggregates depended on the required strength level, being smaller for higher strengths. This is provided optimum is defined as that which gives the minimum cement requirement for a given strength (Fig 7.1).

If optimum is defined in terms of w/c ratio or shrinkage or (less certainly) wear resistance, larger sizes may be best. Whilst the optimum size may vary from 40 mm at 20 MPa to 14 or even 10 mm at strengths over 50 MPa, the margin is not usually large and little harm is done by standardizing on 20 mm. The exception to this is where difficulty is experienced in obtaining a high strength, in which case a smaller aggregate should certainly be tried. It is interesting to note that this effect has now been seen to extend further than most would have believed possible. In reactive powder ‘concretes’ with strengths of 200 MPa and more, the coarsest aggregate used is a fine sand.

Another hotly debated question is the relative merit of gap and continuous gradings. A basic difference is in segregation resistance and pumpability. High slump and pump mixes require continuous gradings but low slump, non-pump mixes compact faster with gap gradings. Two further points worth noting are that single sized aggregates do not segregate in stockpiles and that it is more critical that the exact optimum sand percentage be used in the case of a gap grading than in the case of a continuous grading.

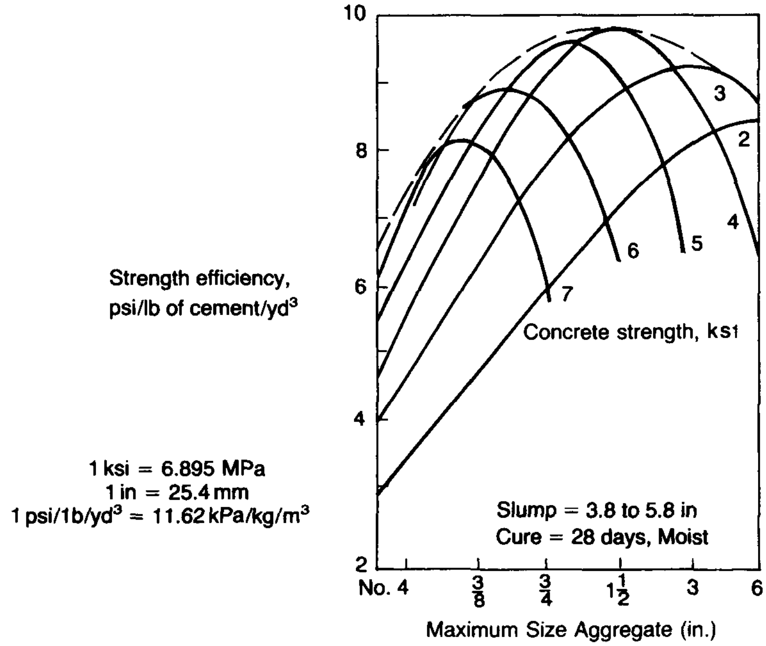


Fig. 7.1 Effect of maximum size of aggregate on mix efficiency.

7.3
LIGHTWEIGHT AGGREGATES

Many types of lightweight aggregates are in use and a full coverage is beyond the scope of the current volume. However, some indication of the possibilities may be of assistance.

Non-structural lightweight concrete is not only outside the scope of the book, but also outside the scope of the mix design and QC systems with which the book is mainly concerned. Such concretes are produced either by the use of foaming agents or the introduction of extremely lightweight aggregates such as polystyrene foam or expanded vermiculite. The range of lightweight concretes is a continuous one. It is difficult to say where non-structural stops and structural starts. There may indeed be some overlap, with some concretes strong enough to be regarded as structural being lighter than others not having enough strength for structural purposes.

Structural lightweight concrete may be regarded as concrete having a strength at least in excess of 10 MPa and, perhaps more importantly, having a good degree of durability. It should also be capable of bonding to and protecting reinforcement. Such concrete is likely to have a density in the range of 1200 to 2000 kg/m^3. Coarse aggregates used include naturally occurring pumice and scoria (of volcanic origin), cinders from coal burning, and manufactured aggregates produced by bloating clay or shale in rotary kilns similar to (and often formerly used as) cement kilns.

The main difficulty with lightweight aggregates is usually that they have a very high water absorption. Some aggregates, especially those manufactured in kilns, may have a relatively impermeable, sealed surface. Those which are supplied as crushed material, especially the natural materials, may absorb 20% or more of their own weight. Such materials must be used in a fully saturated state if difficulty is to be avoided. If this is not done, water will be absorbed during mixing, transporting and placing, with consequent rapid loss of workability. A particular difficulty is that of pumping such concrete. Upon coming under pressure in a pump pipeline, water will be forced into any unsaturated aggregate particles. This tends to cause pump blockages through severe slump loss. The problem tends to be most experienced on two or three storey work where an attempt may be made to pump concrete which is not fully saturated. This may be successful for a time, but as soon as any difficulty is experienced, the concrete comes under greater pressure and the problem is greatly intensified. Once the aggregate is fully saturated, such concrete can be pumped just as well as dense aggregate concrete. Indeed, being lighter, it may well be easier to pump to heights of 50 storeys or more.

It is interesting to note that at least one of the Scandinavian floating oil platforms uses lightweight aggregate concrete. What is particularly interesting is that the aggregate is deliberately used dry. The Norwegians admit that this causes the problems outlined above but state that it is necessary in order to achieve the desired low density. On a dry land project, this would be ridiculous because the concrete would eventually have the same moisture content and the same density whether the aggregate was initially wet or dry. The Norwegians say that this is not the case when the concrete is to be permanently immersed in water from a relatively early age.

The use of saturated aggregate has other benefits than improved slump stability. The weight differential between the mortar and the aggregate is reduced and therefore less trouble is experienced with floating aggregates. This differential is also reduced by the use of air-entrainment and the air also impedes the movement of water through the mix, so reducing slump loss. The entrapped water makes lightweight concrete a little easier to wet cure, having a built in reservoir of water, but this should not be totally relied upon. The density of the concrete is substantially affected by the moisture content and the weight loss on drying can be as much as 200 kg/m^3 with some concretes. It is also important to note that the crushing strength of the concrete may be substantially reduced by its being fully saturated at the time of test. Unlike dense aggregate concrete, lightweight concrete should not be tested fully saturated unless it will be fully saturated in use.

It is interesting to note that it has been proposed to use a proportion of saturated lightweight aggregate in high strength concrete. The objective is to provide water for hydration in concrete which would otherwise self-desiccate (even if sealed to prevent the loss of any moisture) and so be subject to autogenous shrinkage and incomplete hydration.

Lightweight concrete should not be thought of as necessarily permeable, non-durable, or less capable of protecting steel. Such material has been used to produce concrete ships and found to protect the steel very well over many years. It has been shown to give improved resistance to rain penetration in precast housing.

Strength capacity of different aggregates and different mixes varies considerably, some aggregates can be used to produce concretes of 50 MPa and more, but 40 MPa is a more likely figure.

Shrinkage tends to be somewhat higher, and a higher cement content is usually needed for a given strength. These are probably both for the same reason. This is that lightweight aggregates will usually have a substantially lower elastic modulus, and will therefore tend to shed more stress into the surrounding mortar.

The lighter kinds of lightweight concrete also use lightweight fines, but this depends substantially on the type of lightweight fines available. It is generally quite satisfactory to use any fines produced by a rotary kiln type of process, although a proportion of sand will probably be needed to give a suitable grading. However, fines produced by crushing lightweight material are often unsatisfactory. Low density is often a matter of air voids in the aggregate rather than a basic low density material. As the material is crushed finer, more voids are exposed to penetration by the cement paste. There is a tendency to achieve little benefit in lighter concrete and a substantial disadvantage by increasing water requirement. Much structural lightweight concrete uses natural sand as the whole or part of its fine aggregate.

Although a slightly higher fines content may be necessary, structural lightweight concrete is generally amenable to a mix design process similar to that for normal weight concrete. Sometimes it is better to use volume batching for the lightweight material. This would apply where moisture content will vary substantially. However, it is generally a matter of using the different SG of the material in a similar design process. The Conad Mixtune process described in Chapter 3 can be used for structural lightweight concrete. If so used, it is likely to require a 'strength factor' of less than 1. The value may be of the order of 0.7 to 0.9 but there are too many different kinds of such concrete to offer any useful guide. A trial mix will provide a factor which may prove applicable to a range of mixes using the same aggregate.

7.4 BLAST FURNACE SLAG

The blast furnace slag used as a concrete aggregate is quite different to the slag ground as cement. It is the same material in the molten state but has substantially different properties as a result of the cooling process. For use as an aggregate, slag must be cooled slowly to allow attainment of a crystalline state. The material is massive, requiring crushing in the same manner as a natural rock. It is also vesicular, usually to a sufficient extent to make it lighter, but not very much lighter, than a natural coarse aggregate (although it can be deliberately foamed, specifically to make a lightweight aggregate). The vesicularity means that care is needed to use the aggregate in a saturated condition if rapid slump loss and lack of pumpability are to be avoided. It also tends to cause a distinct difference in SG (particle density) between different size fractions. Excellent bond tends to be developed owing to both the vesicularity and the chemical composition of the aggregate and particle shape tends to be better than natural aggregates.

Some sources of slag may have a tendency to cause popouts as a result of remnants of crushed limestone deliberately added to provide the desired conditions in the blast furnace. However, this can be avoided if the limestone is added in smaller particle sizes and combustion is very thorough and even. With this possible exception, the material tends to be a stable and satisfactory aggregate, even under fire conditions. Drying shrinkage is usually relatively low, perhaps because some chemical reaction takes place at the aggregate surface, causing a slight expansion which partially offsets drying shrinkage.

The author has found that crusher fines produced from a particular slag source, when combined with a local dune sand, make a very satisfactory fine aggregate in terms of strength at a given cement content and workability, even compared to a good, long graded, natural sand. However, it should be noted that the granulated slag which can be ground to produce the 'ggbfs' (ground granulated blast furnace slag), although it may look like sand, does not perform well when so used (in the author's experience). This is because it is in a puffed up state like rice bubble cereals and so the grains are weak.

8
Cementitious and pozzolanic materials

8.1
PORTLAND CEMENT

8.1.1
Introduction

No attempt is made in this book to provide a general background and description of Portland cement. Such information is available in almost any textbook on concrete, as well as many specialized books on cement. A particularly recommended reference is the ACI *Guide to the Selection and Use of Hydraulic Cements* (ACI 225, 1985). This is a very comprehensive 29 page dissertation with an equally comprehensive list of further references. Another useful reference is *High Performance Concrete* (Aitcin, 1998) which provides substantial detail on cement, and also on cementitious materials and admixtures.

Some guidance has been provided in Chapters 1 and 6 to the selection of different types of cement for different purposes. What is attempted in the current section is a guide to the extent to which changes in concrete properties may be due to changes in the cement in use.

8.1.2
What can go wrong with cement?

As the user experiences it:

1. Setting—it can set too quickly or too slowly.
2. Strength development—it can develop less strength than usual.
3. Water requirement and workability—it can have a higher water requirement or act as a less suitable lubricant than usual.
4. Bleeding—it can inhibit bleeding less successfully or at the other extreme produce a ‘stickier’ mix than usual.
5. Disruptive expansion.
6. Reduced chemical resistance.
7. Too rapid evolution of heat.
8. Deterioration in storage (either before of after grinding).
9. It can arrive hot, i.e. hotter than usual.

10. It can be delivered from the same depot, and even ground at the same plant, but be produced from a different clinker, i.e. imported clinker using different materials and produced in a different kiln may have been used.

As it is produced:

1. Variation in raw materials.
2. Segregation at any of several stages.
3. Incorrect proportion or uneven distribution of gypsum ($CaSO_4.2H_2O$).
4. Variable firing and grinding temperatures.
5. Unsatisfactory grinding—including overall fineness, particle size distribution and particle shape.
6. Deterioration (including segregation) of clinker in storage.
7. Seasonal variations.

8.1.3 Significant test results

Cement users in some parts of the world can obtain test certificates from their cement suppliers. The following may be of assistance in interpreting the kind of information usually provided on such certificates. Where no test data are obtained in this way, it may be considered too expensive to undertake routine testing on behalf of a single project or small ready mix plant. A solution to this problem is to take a sample either daily, or from each truck of cement (whichever is least). The sample should be kept in a (well labelled!) sealed container until the 28-day concrete test results are obtained and then discarded. A sample is then available, and should be tested if unsatisfactory concrete test results are encountered for which no other explanation can be found.

Where regular test data are obtained, it is useful to maintain graphs of the information provided. As with concrete test data, cusum graphs are far more effective at detecting change points (Chapter 5).

The main results likely to be provided are:

1. **Setting time.** Initial and final set are both arbitrary stages in smooth curve of strength development.

   Abnormal results can indicate incorrect proportion of gypsum, excessive temperature in final grinding (which dehydrates gypsum and alters its effectiveness) or deterioration with age.
2. **Fineness.** Finer cement will:

   (a) React more quickly (faster heat generation).
   (b) React more completely.
   (c) Improve mix cohesion (or make 'sticky').
   (d) Reduce bleeding.
   (e) Deteriorate more quickly.
   (f) Be more susceptible to cracking.
   (g) Generally require more water. (Note that this may be less due to any direct effect of fineness than to the reduced range of particle sizes normally resulting from finer grinding.)

3. **Soundness (Pat, Le Chatelier and autoclave tests).** Intended to detect excessive free lime (perhaps due to incomplete blending rather than wrong chemical proportions). Some experts disagree that the

intention is achieved, but this is beyond the present scope. Magnesia can also cause unsoundness (if as periclase) but perhaps too slowly for Pat or Le Chatelier—needs autoclave or chemical limit.

4. **Normal consistency.** Generally just a starting point for other tests but can show up undesirable grinding characteristics. Where very high strength concrete is involved, large amounts of cement will be required and a very low w/c ratio will be sought. A cement with a high water requirement is at a disadvantage in such circumstances. Interesting uses for this test are as a compatibility check between admixtures and cement or to determine the effect on water requirement of a percentage of fly ash or silica fume, etc.
5. **Loss on ignition.** Mainly a check on deterioration in storage. The test drives off any moisture or carbon dioxide which may have been absorbed. A 3% loss on ignition could mean a 20% strength loss.
6. **Sulphuric anhydride (SO_3).** Check on proportion of gypsum. It has considerable significance for setting time, strength development and shrinkage. The test determines the content of SO_3 from all sources (e.g. added gypsum, oxidized sulphur in fuels, etc.) and in all states. It therefore may not be an accurate guide to the amount of active (soluble) SO_3 present. It is the amount of active SO_3 which affects setting time, rate of strength development, tendency to shrinkage and cracking, etc.
7. **Insoluble residue.** Check on impurities or non-reactive content only. The effect is the same as reducing the cement content by the percentage of the insoluble material.
8. **Compressive strength.** This should be directly related to concrete performance but there can be differences with admixture interactions, different water/cement ratio, etc. In some countries cement is sold as being a particular strength grade. Generally higher strength grades are more expensive but less can be used to meet a strength specification. The selection of a high strength cement becomes important when very high strength concrete is required, since an increase in cement content will not give a strength increase beyond a certain point.

It is very desirable for ready mix producers in particular to develop a good working relationship with their cement supplier. A variation free product cannot be expected, but honesty in reporting current test results, and help in interpreting and compensating for their likely effects on concrete, and co-operation in tracking down any problems is valuable. This kind of co-operation is unlikely if all concrete problems are automatically blamed on the cement, and the concrete producer fails to carry out, and keep proper records of, control tests on concrete.

An important, if relatively rare, occurrence is an unfavourable interaction between the cement and admixtures in use. Examples have been encountered where a particular cement and admixture, both satisfactory with other admixtures and cements, have given trouble in combination. In a recent example the trouble was a false set. A false set is one which occurs for a limited time and can be overcome by continued mixing. This may give no trouble when held in a truck mixer until directly discharged into place but cause a severe loss of pumpability if discharged into a pump hopper during, or prior to, its occurrence. If suspected, such an occurrence can be investigated using a Proctor Needle penetrometer on mortar sieved from the concrete to construct time *versus* penetration resistance curves.

A particularly delicate question is that of the lower value of cement which provides a lower strength. It is of very substantial assistance to a concrete producer if he can rely upon the cement producer advising him of a strength downturn. This enables the concrete producer to increase his cement content and avoid low test results. However, since the cement producer is responsible for the need for the additional cement, there is a natural tendency for the concrete producer to feel that the cement producer should bear the additional cost. It will obviously not encourage the cement producer to provide the early warning if the result is a deduction from his invoice.

The reverse kind of assistance is also valuable. Cement suppliers tend to receive unjustified complaints from customers who have inadequate control systems. It is of value to them to find a regular user who has a good control system so that they can rely on feedback data.

In summary, the development of a good relationship and an effective early warning system with your supplier can be of considerable benefit, and your own good control system is a necessary starting point for such a relationship.

8.1.4 Types of cement

Cement chemistry is extremely involved and not within the scope of the current work, however limited comment on the different types of cement commonly available may be useful. All Portland cement is conveniently regarded as composed of four compounds:

C_2S Di-calcium silicate. Slow acting, low heat generation, best long term strength and durability.
C_3S Tri-calcium silicate. Quicker acting, more heat generated, still good strength and durability but not as good as C_2S.
C_3A Tri-calcium aluminate. Very rapid reaction, high heat generation, responsible for early (but not high) strength and setting, easily attacked by chemicals.
C_4AF Tetra-calcium alumino silicate. Relatively little influence on properties of concrete, present because needed during manufacture.

The relative amounts of these compounds are varied to produce different types of cement to suit different uses:

Type 1– also known as Type A, OPC (ordinary Portland cement), GP (general purpose).
Type 2– modified low heat cement.
Type 3– high early strength or rapid hardening.
Type 4– sulphate resisting cement.
Type 5– low heat cement.

A fifth compound, $CaSO_4$ (gypsum), is interground with the cement clinker to control setting. It is also thought to have a substantial beneficial influence on shrinkage and to produce improved strength. However, an excess can cause slow setting and also unsoundness (destructive expansion). Gypsum can be rendered less effective by excessive heat during grinding.

The reader will be able to work out from the above, or other sources, which compounds will predominate in which cements. However, there are a few matters which are often misunderstood and so should be brought to the reader's attention:

1. Sulphate resisting cement is made so principally by limiting the amount of C_3A. Unfortunately C_3A, whilst of general low durability, happens to be the compound most of use in combating the penetration of chlorides. Too often this cement is assumed to be a general high durability cement and used where chloride resistance is as important, or even more important, than sulphate resistance (e.g. in marine structures). What should be used in these circumstances is blast furnace cement, fly ash substitution, or

silica fume incorporation. Where none of these are available, a higher strength grade of OPC concrete should be used.

2. Low heat cement is generally as sulphate resisting as sulphate resisting cement (since C_3A is also limited to reduce heat generation). However, sulphate resisting cement is not necessarily low heat generating. This is because most of the heat generation comes from the C_3S component (of which there is always much more than the C_3A) and the proportion of this is not necessarily limited in sulphate resisting cement.

It is now coming to be recognized that suitability for different purposes is often better attained by the use of variable proportions of fly ash, blast furnace slag, or silica fume than by the use of different types of cement. These alternative materials, being essentially waste products, used to be thought of as inferior substitutes for cement, used only to reduce cost. It is typical of the reaction of concrete specifiers to new developments that they were often prohibited or strictly limited in proportion.

An interesting justification of fly ash is used on occasions. Faced with a statement that it is a new-fangled, unproven material, it is reasonable to point out that the use of volcanic ash by the Romans has shown such material to be good for 2000 years if correctly handled, whereas Portland cement has yet to show it can last 200 years (and much already has not done so).

8.2 FLY ASH

8.2.1 General characteristics

Fly ash, otherwise known as pulverized fuel ash (pfa), is a pozzolanic material. This means essentially that it is capable of combining with lime (in a suitably reactive form) to form cementitious compounds. As lime is liberated in substantial quantities when normal cement reacts with water, and is present as reactive calcium hydroxide, there is a distinct attraction in adding pfa to concrete.

Fly ash looks like cement to the naked eye, but will not set at all (unless a type C ash, which is a type of ash which contains calcareous material) when mixed with water. It is much finer than cement, has a very rounded particle shape, including some partly broken hollow spheres known as a cenospheres (as opposed to the extremely jagged particle shape of cement) and is of lower density (SG usually 1.9 to 2.4 compared to 3.15 for cement).

Fly ash has a varying ‘pozzolanicity’, i.e. some fly ashes give much better strength than others. No fly ash is as good as cement on a volume for volume substitution basis and but some fly ashes are as good as cement in terms of 28-day strength and better at later ages when substituted on a mass for mass basis and when account is taken of their water-reducing action as well as their strength production at a given w/c ratio.

There are few materials which do not have some drawbacks and with fly ash substitution these include:

1. Reduced early strength.
2. Increased setting time.
3. Reduced heat generation (which is an advantage in hot weather, or for mass concrete, but a disadvantage in cold).

4. Inhibition of air entrainment, if of high carbon content (easily corrected by higher dosage or specially formulated products for use with fly ash, but may give rise to higher variability if carbon content varies).
5. Added complication, i.e. one more factor requiring knowledge and skill to give best results.

Fly ash concrete does not automatically display all the advantages (or disadvantages) of which it is capable. Crude substitution of fly ash for cement can yield better or worse concrete depending on the circumstances and requirements. It could be said that fly ash puts another useful tool in the hands of competent technologists and presents another trip-wire for the uninitiated to fall over. Also there are considerable differences between different fly ashes and there is not an automatic 'best buy' for all circumstances. There are examples of troubles exacerbated if not caused by fly ash and, on the other hand, of the use of fly ash not being permitted through ignorance or blind prejudice in circumstances where it would have been highly desirable.

8.2.2 The composition of fly ash

There are two types of fly ashes, according to the classification in ASTM 618, Type F and Type C. Type F ash is the true pozzolanic material, silica (as SiO_2) being the most important constituent, and alumina and iron oxide are also active (Table 8.1). Type C ash also contains appreciable amounts of calcium compounds and may have some minor cementitious value in the absence of cement. Certainly it is possible to use it in larger proportion than Type F ash in a similar manner to, but not to the same

Table 8.1 Typical chemical composition (%) of mineral additives

	Portland cement	*Fly ash*	*Slag*	*Silica fume*
SiO_2	20	50	35	93
Al_2O_3	5	30	15	2
Fe_2O_3	4	10	1.5	<1
CaO	65	2.5	40	<1
MgO	<2	<2	7	<1
Na_2O	<2	<2	<1	<1
K_2O	<2	<2	<1	<1
SO_3	<4	<2	<1	<1
LOI	<2	<2	–	<2

extent as, a blast furnace slag. Type C ash may be less effective than Type F ash in providing sulphate resistance.

The author's experience is with Type F ash. He has no reason to suppose that, with the above exceptions, the experience would be greatly different with Type C ash, but readers are warned that this is possible.

Carbon is the most important impurity as it can inhibit the action of admixtures, particularly air entraining admixtures. It is measured by loss on ignition which should not exceed 8% and should preferably be very much less. However, the really important requirement is that it should be as consistent as possible since otherwise it may be very difficult to control air content. However, there has been a report (section 8.5) of rice hull ash containing up to 23% of carbon being successfully used in particular circumstances, so possibly higher percentages in fly ash would not necessarily render it useless in all circumstances.

Other impurities are alkalies and magnesium which need to be limited as in cement but are not often a problem.

8.2.3 The effects of fly ash

There are three kinds of effect from the incorporation of fly ash in concrete. These are:

1. Physical effects on both fresh and hardened concrete.
2. Chemical effects on setting process and hardened concrete.
3. Physical chemistry (or surface chemistry) effects on setting process.

Physical effects

The fly ash particles are very similar in size and shape to entrained air bubbles and have many very similar effects, viz:

1. Water reduction. Perhaps of the order of 5% but varies with different ashes. A very few ashes (e.g. some Hong Kong ash) slightly increase water requirement.
2. Reduction of bleeding.
3. Improved cohesion and plasticity.
4. Improved pumpability.
5. Reduced slump loss with time.

Fly ash is not compressible, and probably does not help frost resistance at all (and tends to inhibit air entrainment so that a larger dose of AEA is needed). However this property (incompressibility) makes fly ash even more valuable than entrained air for pumpability. Also fly ash has the benefit that it is present as a clearly defined quantity.

Being so fine, the pfa particles are very valuable as pore-blockers, substantially reducing permeability in the hardened concrete.

Chemical effects

When cement hydrates, it releases free lime. This lime is the softest, weakest and most chemical attack and leaching susceptible of all the constituents of concrete.

The fly ash combines chemically with the free lime to form compounds similar to those produced by the rest of the cement. This reaction is quite slow (7 days before it produces much effect), and generates little heat during the setting process. This is generally a valuable property in hot climates and for mass concrete, but may be a distinct disadvantage in colder climates.

Surface chemistry effect

It appears that fly ash can act as a catalyst or a starting point for crystal growth in the cement paste. Such effects are beyond the scope of this book but it should be realized that there is more to the story than has been told above. This may provide some explanation for a smaller early age strength reduction than chemical effects alone would predict when equal mass substitutions are made.

Dr Malcolm Dunstan (in the UK) and Mohan Malhotra in Canada (Malhotra and Ramezanianpour, 1994) have done interesting work on roller compacted and other concrete with 50 to 60% of fly ash substitution. A very revealing point is that good results are obtained with high fly ash in either earth dry concrete (roller compacted) or concrete with a normal slump attained through using a superplasticizer. However, poor results are obtained with high fly ash at normal water contents. It could be said that the w/c *versus* strength relationship is even more marked in the case of fly ash than in the case of cement.

8.2.4 Dangers to avoid with fly ash

1. Since fly ash is lighter (and cheaper) than cement it might be thought that it would be especially useful in low strength concrete. In fact it does produce much better looking, more segregation resistant and less bleeding prone concrete for a given (relatively high) water to cementitious ratio. However, this is sometimes its undoing. Uninformed or thoughtless people tend to over-water it to a greater extent than plain concrete, yet in fact its strength is **more** affected by a given amount of excess water. Thus fly ash should be used with care and conservatism for low strength requirements.
2. Because strengths take longer to develop, more prolonged curing is necessary for fly ash concrete. It is true that fly ash concrete is substantially less permeable than plain concrete of similar strength, and therefore may be to some extent 'self-curing' in larger masses (and especially for below ground or on ground foundations). However, this does not help the outside 20 mm of exposed concrete which has to protect reinforcement.
3. The same calcium hydroxide which has the disadvantages of being soft, weak and easily dissolved by water or chemicals is the source of the alkalinity which protects steel from corrosion. Therefore, by combining with it, fly ash reduces the chemical protection available for the reinforcing steel. The question is whether or not this is compensated for by the reduced impermeability of the fly ash concrete. The answer lies in the curing: yes, if well cured; no, if not well cured.
4. Because fly ash concrete gains strength more slowly, it is susceptible to creep if de-propped (beams and slabs) too early. The need to prop longer may be an additional cost.
5. Due to reduced bleeding tendency, evaporation cracking will occur slightly more readily.
6. Readiness for trowelling will be delayed—perhaps very significantly delayed in cold weather.

8.2.5 Advantages of fly ash

1. Reduced heat of hydration in the critical period. (This is the period during which heat is being generated faster than it is being dissipated and the temperature of the mass is therefore rising.) In the author's opinion the temperature rise in mass concrete is almost the same as if only the cement and no pfa were present. However, not everyone shares this opinion so you should conduct trials before implementing it.

2. More readily workable fresh concrete—easier to pump, compact, trowel, less bleeding and segregation, better off-form surface usually.
3. Substantially more impermeable concrete (if adequately cured).
4. More durable concrete, e.g. more resistant to sulphate attack than most sulphate resisting cements (Kalousek *et al.,* 1972).
5. Higher strengths possible. Adding fly ash is distinctly better than using cement contents in excess of 400/450 kg/m in most cases. (However, higher cement contents can be used if the cement is low heat.)
6. More economical than straight cement in most parts of the world.
7. Fly ash is particularly useful in marine structures (where curing time is available before inundation) as otherwise there is the conflict of requiring high C_3A to resist chlorides and low C_3A to resist sulphates whereas fly ash concrete resists both.

Summary

The use of a proportion of fly ash is generally desirable except where high early strength is required, heat generation is advantageous or, especially with strength grades below 30 MPa, adequate curing is uncertain and corrosion protection of reinforcement is required. Where fly ash is used, care must be taken to ensure that reported strengths are realistic and not the result of assuming that water cured cylinders necessarily correctly represent poorly cured *in situ* concrete.

The circumstances in which it may be worthwhile specifying that fly ash be used would include hot weather concreting, large sections where low heat cement or ice might otherwise be needed, projects in which exceptionally high strength or good pumpability is needed and projects where high sulphate resistance is needed.

8.3 BLAST FURNACE SLAG

8.3.1 Properties of granulated, ground, blast furnace slag

The properties of cementitious and pozzolanic materials depend on their **chemical composition,** their **physical state** and their **fineness.** This is particularly the case with blast furnace slag. Since it is a by-product of the production of iron, its composition may differ from different sources but is likely to be reasonably consistent from a given source. Table 8.1 shows its composition to be more similar to that of cement than to typical pozzolanic materials. However, to develop satisfactory properties it is essential that the molten slag be rapidly chilled (by spraying with water) as it leaves the furnace. This causes the slag to **granulate,** i.e. break up into sand sized particles. More importantly it causes the slag to be in a glassy or amorphous state in which it is much more reactive than if allowed to develop a crystalline state by slow cooling. In the latter state it is highly suitable as a concrete aggregate but not as a cementitious material. It is important to note that the unground granulated material does not make a good fine aggregate because often the grains are weak, fluffy conglomerates rather than solid particles.

To use as a cementitious material, the granulated slag must be ground as fine or finer than cement. The fineness of grind will (along with the chemical composition and extent of glassiness) determine how rapidly the slag will react in concrete.

Slag cannot be used alone to make concrete but can be used in much larger proportion than pozzolanic materials. Portland cement clinker or some other activator is required to initiate the hydration of the slag. The latter may form 80% or more of the total cementitious material but 60% or less is more usual. An alternative activator is calcium sulphate, pro ducing a product known as 'supersulphated cement'. This cement is beyond the scope of the present volume but those encountering it should note that, whilst it offers valuable properties of chemical resistance and very low heat generation, it requires special care and understanding in use to offset its slow setting and strength development and needs very thorough extended curing.

In Portland blast furnace cement, the slag may be interground with the cement clinker or added as a separate material. The cement clinker is softer than the slag and therefore will be ground to extreme fineness when the materials are interground. Even when sold as a composite 'blended cement' (which term is also applied to fly ash blends) the granulated, ground, blast furnace slag (ggbfs) cement may have been either interground or post-blended.

8.3.2
Properties of ggbfs concrete

Concrete using ggbfs cement will develop strength more slowly than Portland cement concrete. However, if thoroughly cured, it may have as good or better eventual strength. It normally has a greater resistance to chemical attack, and is particularly suitable for marine works. Its normally greater fineness confers resistance to bleeding in the fresh state and lower permeability when hardened.

The glassy surface of the slag may give a slightly reduced water requirement even though it does not have the favourable particle shape of fly ash. The water requirement may however be substantially dependent on the fineness of grind.

It can be added as a separate ingredient at the mixer but is more normally sold interground with cement. There is a long history of extensive use in this form as Portland blast furnace cement, particularly in Europe and the former Soviet Union. The proportion of slag can exceed 80% of such cement.

To some extent this product is sometimes seen as a low grade cement, since it develops strength more slowly and often has a lower eventual strength. However, it usually exhibits better resistance to chemical attack and is noted as particularly suitable for marine works. Obviously the properties of such a material will be very dependent upon the composition of the particular slag. Since ggbfs is a by-product material, there may be a wide variation in quality between cements from different sources. The author has had personal experience of only two sources of slag and the works of local authors should be consulted.

When used in lower proportion, the resulting material is described as a 'blended cement' and this term is applied equally to blends of Portland cement with fly ash. Whilst such cement may be marginally cheaper, and will almost certainly gain strength more slowly, it is by no means necessarily inferior.

8.3.3
Heat generation

It is important to fully appreciate the situation with heat generation. There are three aspects to consider. These are cold weather concreting, hot weather concreting and mass concrete.

Because it can be used in large proportion, ggbfs can give rise to problems with slow setting, slow strength gain and lack of early resistance to frost in cold weather. These same properties can be very advantageous in hot weather. The assumption may be made that the cement will provide reduced peak temperatures in mass concrete as does fly ash concrete. In fact unless a very high proportion of ggbfs (over 75%) or a very coarse grind is used, the cement can give rise to even higher temperatures than with normal Portland cement. This is because, marginally and with some slags, even more total heat can be generated and the slower generation may or may not give a better result depending on whether the heat can be dissipated. It should be clearly understood that there is no question that slag cement generates heat more slowly and so produces distinctly lower peak temperatures in most applications. It is only in situations which are effectively adiabatic (such as foundation rafts more than 3 m thick) that slag concrete may not provide the anticipated benefit. It is certainly particularly useful for general use in hot climates.

8.3.4 Ternary blends

Ternary (i.e. triple) blends of ggbfs, fly ash and cement are sometimes used and have a good reputation. The addition of different proportions of fly ash during batching can give a flexibility of properties to a fixed blend of ggbfs and cement.

8.4 SILICA FUME

Silica fume is a relatively new and very powerful tool at the disposal of the concrete technologist. As with other such tools, the material has to be understood and correctly used if full benefit is to be obtained and deleterious side effects avoided. Being relatively expensive, it is usually used in proportions of no more than 5 to 10% of the cement content of a mix.

The material (also known as micro-silica) is a by-product of the manufacture of silicon, ferrosilicon, or the like, from quartz and carbon in electric arc furnaces. It is usually more than 90% pure silicon dioxide and is a superfine material with a particle size of the order of 0.1 micron and a surface area of over 15 000 m^2/kg (i.e. a hundred times greater than cement or fly ash). Its relative density is similar to that of fly ash at about 2.3 but, owing to its extreme fineness, it has a very low bulk density of only 200 to 250 kg/m^3 in its loose form. For this reason it is usually handled either in a densified form or as a 50/50 slurry with either water or a superplasticizing admixture. In the densified form, particles are deliberately induced to flocculate into clumps which are still as fine or finer than cement particles.

There is disagreement as to whether use of silica fume increases water content or not. This may depend on the particular material but certainly also depends on how it is used. To be fully effective it must be dispersed so that it occupies spaces between cement grains and must not remain in clumps of fume particles. It seems doubtful that this is achievable without the use of a superplasticizer and, in the author's opinion, it should not be used without a superplasticizer. A possible exception may be for shotcrete but even for this purpose the author insists on using a superplasticizer. It may be that, used with a superplasticizer, silica fume does not increase and may even reduce water content at a given superplasticizer dosage. It may also be that if any substantial increase in water requirement results, much of the potential value of the fume will be lost (especially for high strength concrete).

There is a tendency for silica fume to be regarded as only justified for very high strength concrete but this is far from the truth. Its uses are many and varied. It can provide unprecedented reductions in permeability and increased durability and its effects on the properties of fresh concrete are more important for many uses than its effect on hardened properties. These effects include a very substantial increase in cohesion and an almost complete suppression of bleeding or any other form of water movement through concrete (in either the fresh or hardened state). Whilst the suppression of bleeding is desirable in many ways, it does cause exposed flat surfaces of fresh concrete to be very susceptible to evaporation cracking.

Some of the main applications of silica fume in concrete are discussed below.

8.4.1 High strengths

The actual strength level attainable is dependent upon other factors (notably coarse aggregate characteristics) but in many instances silica fume permits the easy attainment of strengths in excess of 100MPa when, for highly workable concrete, 80 MPa might be difficult to attain without it.

The action of the fume appears to be partly chemical and partly physical. It is both superfine and in a highly reactive form. Its pozzolanic reaction with the free calcium hydroxide released by hydrating cement is therefore very effective. The author has described it as being ‘like fly ash squared’, i.e. fly ash with a second order of effectiveness, for this and other properties.

The physical effect of densification, and of improving the structure of the cement paste at its interface with the coarse aggregate, has been considered to be of similar magnitude to the chemical effect.

8.4.2 Durability

Silica fume concrete provides a previously unattainable level of low permeability in addition to the chemical conversion of the most vulnerable calcium hydroxide into durable calcium silicates. It gives a physical uniformity of cement paste structure through avoiding bleeding effects and creating a smaller scale gel structure. Thermal stresses are reduced compared to attempting to improve durability by increased cement content.

Any tendency of the coarse aggregate to alkali-silicate reaction will be forestalled since the alkalis will be consumed in a non-deleterious diffused reaction with the silica fume.

The combined effect of these factors is to provide a new degree of resistance to sulphates, chlorides and general aggressive chemicals. Two aspects which are not necessarily greatly improved by silica fume addition are carbonation and resistance to freezing and thawing deterioration. In the case of carbonation, the consumption of the free calcium hydroxide in the pozzolanic reaction counteracts the beneficial effect of the reduced permeability. However, silica fume concrete has lower electrical conductivity which will assist in providing greater resistance to steel corrosion.

Resistance to deterioration by freezing and thawing poses an interesting question for high strength concrete in general. There is no question either that entrained air still provides greater resistance to freezing and thawing of saturated concrete or that it makes high strength much more difficult and expensive to attain. The question, especially with silica fume concrete, is whether laboratory tests using saturated concrete are realistic. If the concrete is not saturated, there may be no water to freeze and cause damage. A different answer to this question may be appropriate in an exposed high strength column and in a bridge deck.

8.4.3 Cohesion and resistance to bleeding

These properties certainly make silica fume a most desirable ingredient of pumped concrete. A particularly severe test of pumpability occurs in stop-start situations. Many mixes pump satisfactorily on a continuous basis but fail to restart after a delay. The usual cause of this effect is internal bleeding. There is no better cure for this problem than silica fume. Using silica fume and a high solids superplasticizer enabled single-stage pumping of concrete to the top of Petronas Towers, the world's tallest building, in Kuala Lumpur, Malaysia.

Resistance to bleeding also means resistance to bleeding settlement. An important future technique for very high strength columns is to fill steel pipes from the base with fluid, self-compacting concrete. The author has experienced this technique in four-storey lifts but there may be almost no limit to the height attainable from the viewpoint of the concrete. Such columns often involve penetrations by other steelwork at each floor level. In these circumstances any bleeding settlement would be disastrous in causing cracking at vital locations.

Tremie concrete, and particularly any concrete which has to resist free falling through water, also benefits from the incorporation of silica fume, although other thickening agents such as methyl cellulose are also used.

8.4.4
Shotcrete

Silica fume concrete can transform the economics of shotcreting and greatly improve repair performance by its ability to reduce rebound and improve adherence to the substrate in both the fresh and hardened state.

8.4.5
Surface finish

The inhibition of water movement through the mix is very beneficial for surface appearance. Effects such as hydration staining, sand streaks, bleeding voids on re-entrant surfaces and settlement cracking are avoided.

A possible problem is that the properties of the particular silica fume can cause a substantial effect on colour. This is due to any carbon content and is apparently more influenced by the size of the carbon particles than by their percentage by weight.

8.5
RICE HULL ASH

Rice hull ash (RHA) is produced by burning rice hulls (i.e. husks or shells) which invariably contain a large proportion of silica. It has similarities with silica fume and with blast furnace cement. Chemically it is like silica fume in being almost pure silica. Its similarity to slag is that the conditions of production are very important. As slag must be cooled very rapidly to achieve a glassy or amorphous state (glassy **is** amorphous as opposed to crystalline, they are not alternatives) so RHA must be burnt at a relatively low temperature to achieve that state. Burning at too high a temperature gives essentially a very fine, but not reactive, silica sand. However, it is essential that the burning should be complete or the ash will have a high carbon content, which is anathema to the uniform and effective performance of admixtures. However there has been a report (Dalhuisen *et al.,* 1996) of ash with up to 23% of carbon being used successfully. This was in tropical conditions where air entrainment was not required.

Like slag, the particles are initially 'fluffy'. They are much larger than silica fume particles and yet have a higher surface area. It is necessary, and relatively easy, to grind such particles to avoid excessive water demand and resistance to compaction. With such a material, it is clearly important to evaluate product from a particular source for performance and uniformity since it can range from being as valuable as (and similar to) silica fume to being as deleterious as silt when incorporated in concrete.

There are substantial quantities (tens of thousands of tons) of rice hulls available annually in many parts of the world. They constitute a potentially valuable resource if suitably prepared, rather than being a large scale nuisance even after burning indiscriminately to reduce volume.

8.6 SUPERFINE FLY ASH

In some parts of the world a superfine grade of fly ash is available which can be regarded as midway between normal fly ash and silica fume in cost, effectiveness, and desirable dose rate. The material can be highly competitive depending on relative costs and availability. It neither requires such large volume batching facilities as normal fly ash nor is as difficult a material to handle and disperse effectively as silica fume.

8.7 COLLOIDAL SILICA

A French development is of silica chemically produced in a colloidal form rather than resulting as a by-product from ferrosilicon production. The material is even finer than silica fume but, being in a liquid suspension, does not present the same handling difficulties. It is more expensive, but used at a lower dose rate than silica fume. It is claimed to be particularly effective and economical for shotcreting (Prat *et al.,* 1996).

9
Chemical admixtures

9.1
GENERAL

The days when it was defensible to take the attitude that admixtures are an unnecessary complication passed in the 1950s. It is now quite clear that admixtures can **both** solve otherwise intractable technical problems **and** save substantial cost. They also have the potential to **create** technical problems if improperly selected or used.

High strength or high performance concrete is a current hot topic (although its ranking in terms of production volume is nothing like its ranking in volume of technical literature). In presenting the theme report on production of HSC/HPC at the Paris symposium (Day, 1996a) the author remarked that, of the more than 20 submitted papers included in his report, only one specifically dealt with a superplasticizer but all the concrete covered by the reports contained superplasticizer. There may be a temptation to think that the use of silica fume, or high strength, is the outstanding characteristic of high performance concrete but probably its most basic and essential feature is the use of a superplasticizer.

The technology of admixtures is both extensive and virtually a foreign language to many in the concrete industry and related professions. It is easy to provide more detail than can reasonably be absorbed and retained by such persons. This chapter is therefore aimed at providing guidance rather than at providing detailed knowledge.

It is important to realize both the complexity of the situation and the inaccuracies inherent in any attempt to compare the relative value of different admixtures. Different admixtures can have significantly different relative values when used with different cements or other different conditions. A particular brand name of admixture may be differently formulated in different parts of the world. A difference in the time of addition (relative to that of the cement first coming into contact with the water) can substantially affect the performance of an admixture. Different results may be obtained from the same mix and admixtures when mixed in a truck or in a laboratory mixer.

The basic cost of most admixture raw materials is relatively low compared to the selling price of the admixture. This is at least partly due to the very considerable costs of R&D, quality control, technical service and marketing. However, with the possible exception of very large concrete producers with good facilities and very knowledgeable staff, the availability of technical assistance from an admixture supplier may be good value for money.

If one admixture enables the saving of 5 kg of cement per cubic metre of concrete more than another, this may save several hundred tonnes of cement per annum. However, the strength difference at the same cement content would only be of the order of 1 MPa and this may be within the margin of error of the trial mixes used.

If it is accepted that trial mixes may be inaccurate and that other user's production results may not be applicable, the only remaining practical selection basis is an extended parallel trial. This may be simply a matter of using the admixture on trial in one or two trucks per day and always testing these trucks. Over a period it will be accurately seen whether there is any significant advantage from using the new admixture. It may be considered necessary, for a short initial period, to supply the special trucks to a non-critical location or for a use for which a lower grade has been specified.

On the whole it is probably of greater importance to select the correct **type** of admixture and to use it in the most advantageous way than to obtain the most cost effective admixture. It is therefore again emphasized that most concrete producers should be seeking the ideal admixture supplier rather than the ideal admixture, i.e. the correct advice may be more important than the best admixture.

9.2 SPECIFYING ADMIXTURE USAGE

It is very important that concrete **users** do not specify the use of particular admixtures unless absolutely essential for a particular purpose. If they do so, the responsibility of the concrete **supplier** for the performance of the concrete will be substantially reduced and any and every problem encountered will in some way be blamed on the specified admixture. As far as possible the concrete supplier must be left to formulate his concrete and this should include the use of his choice of admixtures. Where a particular admixture is considered essential, this should be discussed with the concrete supplier and an attempt made to have him use it 'of his own volition'. If it became normal to impose the concrete user's choice of admixture on the concrete producer, this would sabotage his entire control system. This would occur because results could not be grouped together for analysis.

As with other aspects of mix design, the purchaser should be end tled to know what is being used in his concrete and to have the right of objecting to unsatisfactory proposals. In general, this right should not be used lightly. The purchaser should certainly refuse permission to use admixtures containing any significant amount of calcium chloride in concrete to contain reinforcement. This is because it is well established that calcium chloride strongly promotes the corrosion of reinforcement.

Where resistance to freezing and thawing is required, the purchaser should certainly specify that air-entrainment be provided. It may also be reasonable to object to an air-entrainer which produces too large a bubble size. This is because it is the **spacing** of the air bubbles which matters, whereas the total **volume** is measured by all typical tests. The spacing can only be determined by microscopic examination of a cut and polished face of hardened concrete. It would only be undertaken if, for example, your local reputable admixture supplier advises you that a particular air entrainer your concrete supplier is using is in fact only appropriate as a car washing detergent.

9.3 POSSIBLE REASONS FOR USING ADMIXTURE

1. To save money—by reducing cement content for a given strength and workability.
2. To improve concrete properties, for example:

   (a) reduction of bleeding or segregation.

(b) compensation for aggregate grading deficiencies.
(c) reduced permeability.
(d) improved pumpability.
(e) reduced shrinkage.

3. To compensate for weather conditions or haulage distance, e.g. retarders and accelerators.
4. Reduction of labour costs—superplasticizers.

9.4 TYPES OF ADMIXTURES AVAILABLE

9.4.1 Water reducers

These are basically lignosulphonates which are natural retarders but may be modified by the addition of accelerators such as triethanolamine (hopefully no longer calcium chloride as in the past).

A water reduction of the order of 5 to 10% is obtained and the admixture is used basically to enable cement reduction. Some of the water reduction is due to the unavoidable entrainment of 1.5 to 2% of air by this type of admixture. The accelerating part of the admixture causes an increases in shrinkage at a given w/c ratio, but this is offset by the water reduction. There is some evidence that early shrinkage is less compensated than later shrinkage and this may lead to slightly increased susceptibility to early cracking.

The time of addition of these admixtures may be important, a delayed addition giving substantially more effect.

In some cases readiness for trowelling of slabs may be delayed even when 24-hour strength is not reduced.

9.4.2 Water reducing strength increasers

These are ‘polymers’—hydroxy-carboxylic acids and polysaccharides. These are sometimes regarded as very similar to lignosulphonates. The cement saving is of a similar order but the action is a little different since water reduction is slightly less and there is a small direct strength increase at a given water/cement ratio.

These admixtures are in some cases a little more effective in cement saving than lignosulphonates (especially at higher cement contents) but are more sensitive to variations in cement characteristics.

Newer types of admixture (described as ‘synergized’ by some manufacturers) often combine polymers and lignosulphonates in an attempt to get the best of both characteristics.

9.4.3 Retarders

Set retardation to any desired extent is readily available with no deleterious effects—with or without water reduction.

Sugar is a violent retarder and very small quantities can produce a dramatic effect.

It should be noted that set retardation is not the same thing as workability retention. Mixes containing lignosulphonates may lose slump more rapidly than plain concrete in some circumstances.

Delayed addition may be very important because a greater water reduction is obtained by a delay of the order of 5 minutes after the water has been in contact with the cement. When retarding admixtures are added already dispersed in the mixing water, the retarder can retard the going into solution of the gypsum which is added to cement during manufacture to control rapid setting. In this way a more rapid set may be caused by a retarder. It is not usually practicable to actually delay addition in ready mix operations, but the same effect may be obtained if the admixture is added in concentrated form and takes some time to disperse through the mix. Suppliers now deny that this problem still exists, it certainly used to, but now some producers add their admixtures to the mixing water with apparent impunity.

9.4.4 Accelerators

Set acceleration, unlike retardation, is only obtainable within limits and with some risk (or certainty) of deleterious side effect. The field of accelerators in particular is one in which development work is occurring and details are not readily available. The information given below is likely to prove outdated. Purchasers will need to carry out their own trials.

Triethanolamine and salicylic acid are only mild accelerators and are not used alone.

Calcium chloride is by far the most economical and effective accelerator. However, it has the severe disadvantage that it strongly promotes the corrosion of reinforcement (and any other embedded steel). Many, but not quite all, authorities claim that it also increases shrinkage quite substantially.

Calcium formate and calcium nitrite produce almost similar strength gains but less effect on setting times. Both are substantially more expensive than calcium chloride.

Sodium silicate and aluminate and sodium or potassium carbonates are powerful set accelerators but reduce strength at later ages.

Hot mixing water or steam curing can also be used to accelerate set and strength gain. Hot water is in fact often a quite suitable choice as an accelerator, especially in cold climates. A recent major project involving thousands of very large precast segments for an elevated roadway again demonstrated this. Faced with a requirement to attain 18 MPa in 7 hours, only two weeks were available to solve the problem. It took only a theoretical analysis and two sets of four trial mixes each to convince the client that hot mixing water was a more economical solution than steam curing, chemical accelerators, or extra cement. The point is, given the very short curing period, that hot mixing water takes immediate effect whereas steam curing has to be applied gradually Of course a superplasticizer was also used and the author's early age system (section 12.2) was an integral part of the solution.

Superplasticizers are very useful for high early strengths, because they enable low w/c ratios which not only increase eventual strength, but also increase the proportion of that strength developed at earlier ages. Also they give a strong dispersing effect which makes more effective use of high cement contents. Some producers, particularly in tropical climates, find that using a superplasticizer is an economical substitute for steam curing precast units. Of course such a substitution provides a very large strength margin at later ages.

9.4.5
Air entrainers

It is of interest that most concrete of up to 30 MPa (4500 psi) in Australia contains entrained air but the practice appears unusual in SE Asia. Worldwide, one of the principal benefits of air-entrainment is greatly enhanced resistance to damage by freezing and thawing, but in Australia, as in SE Asia, this is not a problem.

The other reasons for using air-entrainment are:

1. Reduced bleeding.
2. Improved cohesion.
3. Grading rectification.
4. Reduced permeability.
5. Improved pumpability.
6. Better surface finish.

The amount of entrained air required for these purposes is somewhat less than may be required for high frost resistance, 3 to 4% being normal in Australia.

The disadvantage of air entrainment is that it is an additional factor to control and test, since excessive air can severely reduce strength and pumpability.

Entrained air is generally considered undesirable in mixes of high cement (or other fines) content where frost resistance is not required. However, the author has used entrained air to provide lubrication in mixes where fines were excessive and strength relatively unimportant.

Many investigations show that entrained air is still necessary for resistance to freezing and thawing, even in very high strength concrete. The author is dubious about this, considering that it may only apply to fully saturated specimens used in laboratory investigations rather than to real structures. However, this is unproven and the omission of entrained air in concrete subject to freezing and thawing represents a risk.

9.4.6
'Waterproofers' (or, more realistically, permeability reducers)

These comprise calcium, ammonium and butyl stearates or oleates, asphaltic emulsions, also silicones and methacrylates.

Their action is generally intended to be either to block pores or to produce a hydrophobic (water repelling) action either at the concrete surface or on the surface of the pores in the concrete. This action may have a limited life in terms of years. On the whole, and for most purposes, chemical waterproofers are not worthwhile. An adequate cement content, good curing and a low w/c ratio are the most important factors in achieving low permeability. A failure to provide any of these three will not be ade quately compensated by the use of a waterproofer and if they are all provided, the concrete will be satisfactory for most purposes without a waterproofer. However, this simplistic view is not the full story.

A distinction should be made between concrete which will repel surface water (such as rain), concrete which will retain water under pressure without apparent leakage (e.g. water tanks) and concrete which will not even permit the passage of water vapour under pressure (e.g. to avoid damp spots inside buildings via floor slabs or retaining walls). It is the latter which is very difficult to achieve (impermeable membranes such as polythene sheet or coal-tar epoxy paint, applied outside, i.e. on the side from which the water is coming, are normally used). However, at least one proprietary admixture (Caltite) has an established long term record. Also there is an expanding use of silica fume to accomplish the same objective at lower cost. It should be pointed out that workmanship is extremely critical in achieving watertightness without a membrane. Any admixture supplier who provides an effective guarantee to achieve watertight concrete will certainly insist on being engaged to supervise the production and placing of the concrete.

Repelling surface water is relatively easy although the action may not be permanent. It may be achieved by the integral (i.e. incorporated in the mix) or surface (i.e. painted on) use of silicones or methacrylates or (surface only) chlorinated rubber. These treatments are useful and desirable, for example, when applied to coloured split block concrete masonry. When untreated, the colour of such concrete appears to fade, but this is due to the deposition of efflorescence on the surface. The colour is restored to some extent when the concrete is wet or the efflorescence is removed by acid washing. It may be maintained by a clear surface coating (while this lasts).

Retaining water in a water tank requires only good, well compacted concrete of say 40 MPa (6000 psi) grade which is both cheaper and better than say 25 MPa concrete with an integral waterproofer of the typical stearate type. The concrete does permit the passage of some water, but not sufficient to cause any noticeable loss of contents and not more than will immediately evaporate from the surface, which appears completely dry.

The author has undertaken a limited trial of the admixture known as 'Caltite' which is understood to contain an asphaltic emulsion in addition to an integral chemical waterproofer. It is claimed that the action is to block the concrete pores with the asphalt particles in such a way that the greater the pressure the more effective the plugging action. The trial compared the Caltite mix with a control mix having 100 kg additional cement, and a superplasticizing admixture to substantially reduce water content. Under a pressure of 35 psi the author was surprised to find that the Caltite mix performed better than the control, permitting virtually no passage of water at any stage. However, the control mix was cheaper than the Caltite mix and it also gradually ceased to permit any passage of water after the first few days. Thus the Caltite was very satisfactory, at least in the short term, but its expense may not be essential.

Xypex and Krystol (very similar materials) were originally clear solutions claimed to have the ability to penetrate concrete against the flow of water (i.e. against seepage) and to grow crystals in the pores so as to block the flow of water. They were painted on the surface towards which the water is moving. This sounds like science fiction (or advertising exaggeration) to the author but he has seen a number of situations in which it has been apparently effective. Nowadays these materials are more likely to be used as a component of a grout injected in repair situations, or as a component of original concrete. One interesting example of the effect of Xypex was its use as a basic ingredient in a retaining wall for a Singapore basement (since much of Singapore is on reclaimed land, basements tend to be below water level). The reason for the author's involvement was that the contractor was unable to render the wall since, being non-absorptive, mortar would not stick to it.

The use of fly ash reduces permeability, but silica fume is clearly even more effective. Ggbfs is also reputed to reduce permeability. These materials have been dealt with in Chapter 8.

9.4.7 Pumping aids

These include:

1. Wax emulsions.
2. Thickening agents (methyl cellulose, polyethylene oxide).
3. Fly ash.

4. Silica fume

Wax emulsions and thickening agents do improve pumpability, but the improvement is not dramatic. Expense and difficulty may be appreciable. Fly ash is a big help if available. Silica fume is very effective but also quite expensive.

It has been said that the only satisfactory test for pumpability is to pump the concrete but that the most effective cheap and simple test is to test bleeding. It is probably true that concrete which bleeds will not pump but the reverse is not necessarily the case. It can be seen that the above admixtures are all in effect bleeding suppressants.

9.4.8
Superplasticizers

These are also, perhaps more correctly, known as high range water reducers.

Superplasticizers have become distinctly more important in the years since the first edition. HPC (high performance concrete) can almost be defined as concrete containing a superplasticizer. Their wider use and greater importance have been accompanied by a better understanding of their strengths and weaknesses. It is becoming apparent that denser packing of the paste fraction of concrete is the key to higher strength, greater impermeability, etc. This requires the use of finer materials such as silica fume, finer cement, superfine fly ash, etc. Such finer materials have a higher water requirement which offsets their benefit. The answer to this is to use the fine material together with a superplasticizer to counter the higher water requirement. It has also become apparent that not all Superplasticizers are compatible with all cements. The best way to check on this is to use the admixture at the intended dose in an otherwise normal Vicat setting test. Better still the test can be repeated at different dosage rates to establish the 'saturation dosage' (i.e. that dosage above which no further water reduction is obtained) as well as checking on the possible rapid workability loss which is the nature of the incompatibility of some admixtures and cements. Alternatively it may be found that excessive retardation of set is experienced in some cases. It may also be desirable to include in this test any pozzolanic materials intended for use in the concrete.

The original Superplasticizers were melamine formaldehyde and sulphonated naphthalene. The former originated in Germany and the latter in Japan. These are highly effective water reducers with a short period of effectiveness and apparently no permanent effects (no retardation or air-entrainment). They are relatively expensive (although less so than formerly, now that they are in higher volume production and usage) and cannot be justified on cement reduction grounds for ordinary concrete, as can normal water reducers.

They can be used in three ways:

1. **To produce 'flowing' concrete.** Such concrete is virtually self-compacting and may be justified on labour saving grounds. It may also be worthwhile where excellent surface finish (on vertical formed surfaces) is required or for very congested sections.
2. **To produce very high strength or durability.** At normal workability the water reduction can give high strength increases. This may only be worthwhile when the strength required cannot be obtained by increased cement content. On the other hand a superplasticizer is very desirable with high cement content as the cement may not otherwise be adequately dispersed.
3. **To limit shrinkage.** In thin walls with congested reinforcement a small aggregate, high slump mix may be necessary to achieve full compaction. Such concrete would have excessive shrinkage if the

high workability were attained by increased water and cement content, but not if obtained by using a superplasticizer at normal water and cement contents.

The above remarks apply to what are now described as ‘first generation’ superplasticizers, being the pure materials listed. The situation has now become much more complicated in that there are ‘second and third generation’ superplasticizers which retain their action over a considerable period of time (in some cases more than 2 hours).

The original materials derived their effectiveness not so much from a new property as from an absence of two old properties. They are able to be used at much higher dose rates than normal water reducers because they do not either retard set or entrain air. As an example of this, it was required to produce a highly fluid mortar with a very low w/c ratio to surround and protect a steel tension pile (or ground anchor). High strength was really only essential at the rock anchorage over 30 m below ground level. A superplasticizer was considered, but it was realized that a normal water reducer at the same dosage would produce a similar water reduction at lower cost. It was an advantage that a very long retardation resulted (because the mortar was placed first and the pile was lowered into it). The high air percentage was reduced to a very modest amount by the fluid pressure at the full depth.

There is now an enormous variety of superplasticizers available, from a dozen or more different countries. The original materials have been supplemented and/or replaced by others, including lignosulphonates formulated to entrain reduced amounts of air and produce less retardation. Their cost, relative to the cost of labour, is reducing. The value of very high strength concrete is becoming more widely realized. Perhaps more important still, it is being realized that these materials are not only labour content reducers, but also skill requirement reducers. For all these reasons, the use of superplasticizers is on the increase.

In America, especially, these materials are now called ‘high range water reducers’. This recognizes that they are often not used to superplasticize concrete but only to produce a substantial water reduction. Similarly the emphasis is no longer on high strength concrete but on ‘high performance concrete’, it having been realized that much concrete uses high range water reducers, silica fume etc for reasons other than high strength.

9.4.9
Shrinkage compensators

Finely divided iron and calcium sulpho-aluminate are used as shrinkage compensated.

These materials work but require careful use to avoid the expansion tendency being disruptive. Also it must be remembered that they do not actually work by reducing shrinkage. In both cases an expansion is produced whilst the concrete is kept damp (i.e. before any shrinkage occurs) and the concrete then shrinks normally. The initial expansive tendency is restrained by reinforcement or by abutting concrete and develops a compression which dies away under the later influence of shrinkage. In addition to the risk of excessive expansion causing disruption, there can also be a ‘threshold’ effect in which the expansive tendency is inadequate and the pre-compression is all lost in creep of the concrete, leaving no effect on subsequent shrinkage.

In the USA shrinkage compensating cements are available, and even expanding cements designed to automatically apply prestress to cast-in steel tendons. This is done by the incorporation of calcium sulphoaluminate in the cement during manufacture.

‘Eclipse’ is a new shrinkage-reducing admixture of which the author has yet to have personal experience. However, it is reported to be quite effective in almost halving shrinkage, but rather expensive. The mechanism is understood to be based on reducing the surface tension of water in the pore space of the cement paste. It would seem to the author that not all concrete would retain enough water for this to be applicable but time will tell.

10
Statistical analysis

Statistical analysis is not an exact science. However rigorous and elaborate the statistical techniques used, the conclusions can be no more reliable than the assumptions on which they are based. Where a limited amount of data has been obtained from a one-off experiment or series of observations, it can pay handsome dividends to apply very elaborate analysis techniques to squeeze out the last drop of knowledge. However, QC is not a one-off experiment but a continuing flow of data. Furthermore it is a field which is, or should be, rigidly governed by economic considerations.

The requirement is to ensure a given minimum quality of concrete in the structure. This can be accomplished by using a higher average quality, at a higher cost in materials, or by achieving a lower variability through higher expenditure on control. The higher control expenditure itself can be in the form of a large amount of rough testing with little analysis or in a smaller amount of more carefully monitored testing and a more thorough analysis of the results. A balance should be sought which yields the minimum overall cost for a given required quality. The balance must take into account the standard of personnel and equipment economically available. There is no merit in devising a system which requires that every testing officer be a qualified engineer and every team include a professional statistician, if the result is a higher cost for a given minimum quality.

The concern should not be to apply elegant or rigorous statistics but only to achieve accurate control of concrete quality. Relatively crude statistical techniques can be used if their limitations are very clearly understood and the controller must always be prepared to overrule or revise unrealistic conclusions produced mathematically. It is quite difficult to do this without permitting bias to cloud judgement but there are several factors which save it from being almost impossible. One of these is that in QC work a conclusion is usually provisional and subject to revision as further results are received; thus a downturn in results may be dismissed as a chance variation or testing error when first spotted, but if it is confirmed by the following day's results, it must then be accepted. Another is that related variables such as slump, density and concrete temperature can confirm or deny an unusual result by demonstrating what caused it. Thus if a single low test result is from the lighter of a pair of specimens, it can be neglected, but if a low pair of strengths are accompanied by a high slump reading they must be accepted as fact, but still may not indicate a need for a mix revision—only for better slump control.

Some crude statistical techniques have been used by the author. This has been done quite deliberately since in his opinion more mathematical sophistication would not help. Rather, what is needed by way of sophistication is a very thorough realization of what factors may cause conclusions to be unrealistic, how unrealistic they might be and what can be done to ensure that such conclusions are weeded out and do not lead to inappropriate control action. The total amount of sophistication in a scheme must be limited to keep it within the capability of ordinary practitioners. It must always be borne in mind that the objective is to achieve more economical operation rather than to display virtuosity.

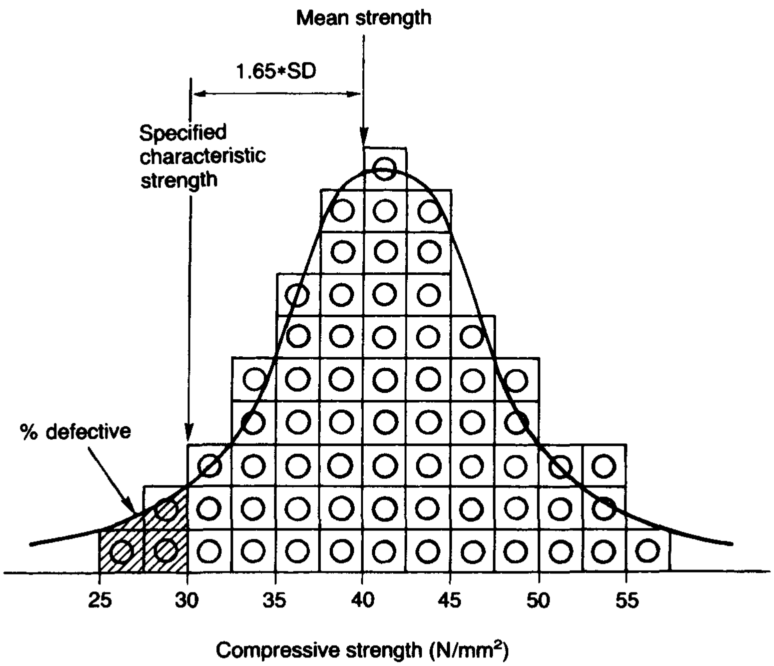


Fig. 10.1 Simulated distribution of test results.

10.1 THE NORMAL DISTRIBUTION

If a mathematical description or **pattern** of a set of results can be found, it may be possible to establish what the pattern is from a limited number of results already obtained and use it to predict what future results will be obtained **if the current pattern continues to apply.** It may for example be possible, without ever having obtained a result below some particular value, to predict that a result below that value will inevitably occur unless action is taken to change the pattern. We shall be in a much stronger position to control concrete quality if it can be established that control action is necessary without experiencing even one 'failure' than if we have to wait for failures before reacting to them. The position will be even stronger if it can be established from early age tests, or even from tests on the freshly supplied concrete, rather than from 28-day results.

If each result is considered as a ball and a number of slots corresponding to strength ranges are set up (e.g. 22.5 to 25 MPa, 25 to 27.5 MPa, 27.5 to 30 MPa, etc.) each result can be placed in its slot giving a picture like Fig. 10.1. Such a figure is known as a 'histogram'. If we have a very large number of balls and divide them into narrower slots, the result may approximate to a smooth curve as shown in Fig. 10.2.

One purpose of introducing Fig. 10.1 was to make it clear that area under the normal distribution curve represents number of results. Just as each ball occupies the same area in the two dimensional representation, so each unit of area in the normal distribution represents a fixed proportion of test results.

This type of graphical representation is called a 'frequency distribution' or just a 'distribution'. There are many different shapes of distribution curves known to statisticians but the particular bell shaped curve shown is called a 'normal distribution'. It can be constructed from a standard table of figures ('ordinates')

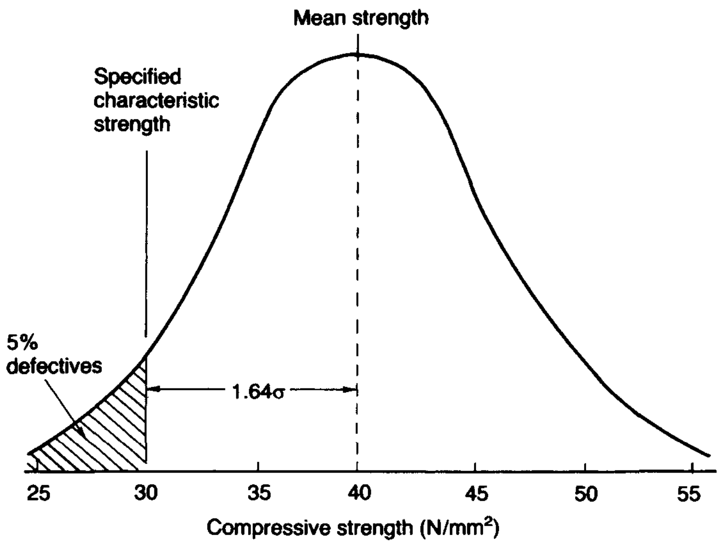


Fig 10.2 The normal distribution.

appearing in any statistics textbook. This table will be accompanied by a second table (Table 10.1) listing the areas under the graph more than a given distance away from the mean (the high point).

The information needed to construct the graph (apart from the table of figures) is only the mean (average) of all the results which we shall call X and a quantity called σ which is the 'standard deviation' and is a measure of how widely the results are spread. The numbers X and a can be read from many simple calculators when a series of results are entered, they can also be automatically produced by a computer. The standard deviation is the square root of the average of the squares of all the differences between each individual result and the average of all results, i.e.

$$\sigma = \sqrt{[\Sigma (x_i - x_m)^2/n]}$$

where x_i=individual result
x_m=mean of all results
n=number of results.

Figure 10.4 shows two distributions with the same mean but different values of standard deviation. Figure 10.3 shows three distributions with the same standard deviation but different mean values.

We are interested in the percentage of results less than a certain strength (i.e. the percentage defective). Looking again at Fig. 10.2, the distance below the mean (or above, the curve is symmetrical) can be expressed as a parameter k (i.e. a variable number) times σ and the area as a percentage of all results. The published tables relate the area to the value of k. Table 10.1 is an extract from such a table.

10.2
PERMISSIBLE PERCENTAGE DEFECTIVE

There is logic in using a 5% defective level (or even a 10% defective level) in that adherence to the assumed statistical distribution is not exact. The

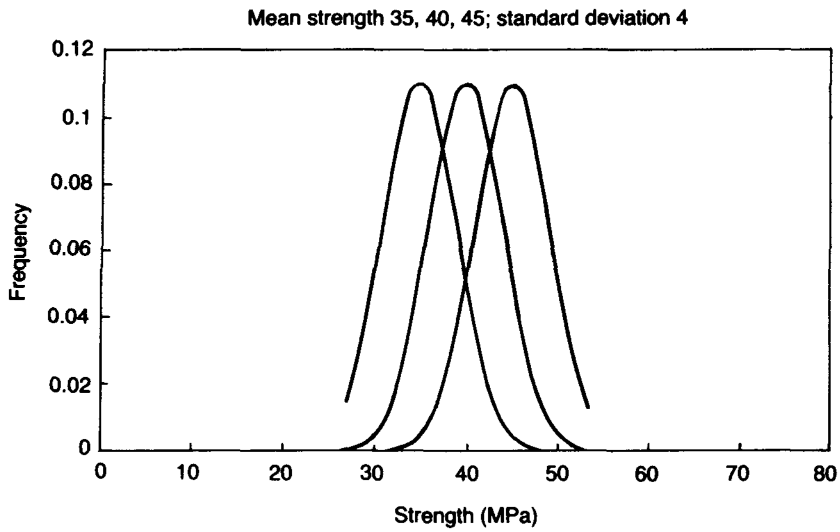


Fig. 10.3 Three distributions with the same standard deviation (σ) but different mean values *(x)*.

Table 10.1 Percentage of results outside statistical limits

A(%)	*k*
0.1	3.09
1.0	2.33
2.5	1.96
5.0	1.65
10	1.28

assumption predicts reasonably well the level below which 5% of results fall (in the author's experience there are likely to be actually 2 to 3% below the level below which 5% are predicted to fall—more about this later) but at the 0.1% level, the assumption has become highly theoretical and any result actually below this level is almost certainly the result of some ascertainable special cause rather than normal variability. So if the intention is to actually predict what results will be obtained, the 5% level is as far as it is reasonable to go and the USA use of 10% may be even more realistic. However, if the results are to be judged by analysis of an adequate number of them rather than by whether any results are actually below a particular level, the Fig. 10.4 situation can be considered because it then becomes a matter not of whether the distribution is accurately followed, but simply of how much incentive it is desired to provide to achieve low variability.

Figure 10.5 illustrates the available options. Figure 10.5(a) shows 5% below specified strength, as used in most parts of the world. Figure 10.5(b) shows the effect of decreasing the permitted percentage defective to 0.1%. This option would provide a greater financial incentive to achieve low variability (i.e. good control) but would substantially increase the average cost of concrete. Figure 10.5(c) shows that, by adjusting the specified strength level, the average cost of concrete can be kept unchanged while still providing an increased incentive to good control.

Any suggestion to specify a 0.1% defective level is certain to encounter the criticism that this is highly theoretical and unrealistic. It is very important to clearly make the point that this is true but immaterial.

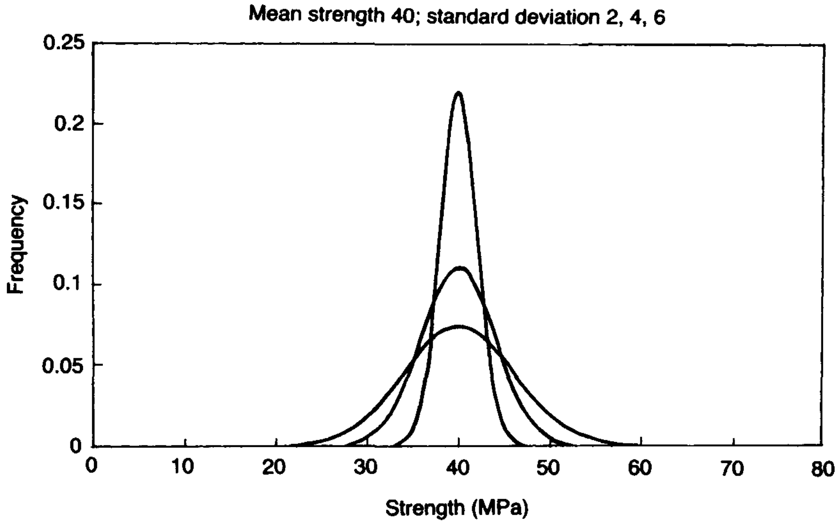


Fig. 10.4 Three distributions with the same mean value *(x)* but different standard deviation (σ).

What matters is to realize that it is possible to make use of any desired relative value of mean strength and standard deviation without affecting the cost of concrete from an average producer. If s is the standard deviation considered to be average, then the required mean strength x for a specified characteristic strength f'_c could be required to be:

$$x = f'_c + k\sigma - (k - 1.65)s$$

or, in the USA,

$$x = f'_c + k\sigma - (k - 1.28)s$$

k can be given any desired value without affecting the mean strength required of an average producer. The larger the value of k the greater the cost advantage given to a lower variability producer and the greater the disadvantage suffered by a higher variability producer. There is no requirement to select a value of k which represents a particular percentage of results (e.g. from Table 10.1). Users should not forget the table and its significance but it may be reasonable to select a value of 1, 1.5, 2, 2.5 or 3 (or even 4, which would have no statistical significance) according to the relative importance attached to mean strength and variability.

Looked at in this way, the American choice of 1.28 is seen to provide a lesser incentive to achieve low variability than the more usual 1.64 or 1.65 and the author would prefer to use a value of 2 or even 3. The reduced incentive may explain a reduced interest and attainment in the USA in matters of QC.

Having discounted the realism or otherwise of the theoretical percentage defective as a basis for choosing the value of k, there is another consideration. This is the accuracy with which σ can be assessed. Section 10.4 below provides details.

Taking the data from Tables 10.2 and 10.3 together, it is seen that the error of estimation of the mean of three results is about five times the error in estimating the standard deviation from the last 30 results and almost four times that from 20 results. A proposal to multiply the standard deviation by 2 or 3 would therefore be reasonable if the a were based on at least the last 30 results. However, it should be realized that a standard deviation change of less than ±25% from its previous value would not be significant.

There is a further consideration in increasing the number of results on which the standard deviation is based. If the results analysed extend across a change point in mean strength, the standard deviation will be artificially inflated. Care is necessary in determining the desired result. As discussed in section 4.2, the

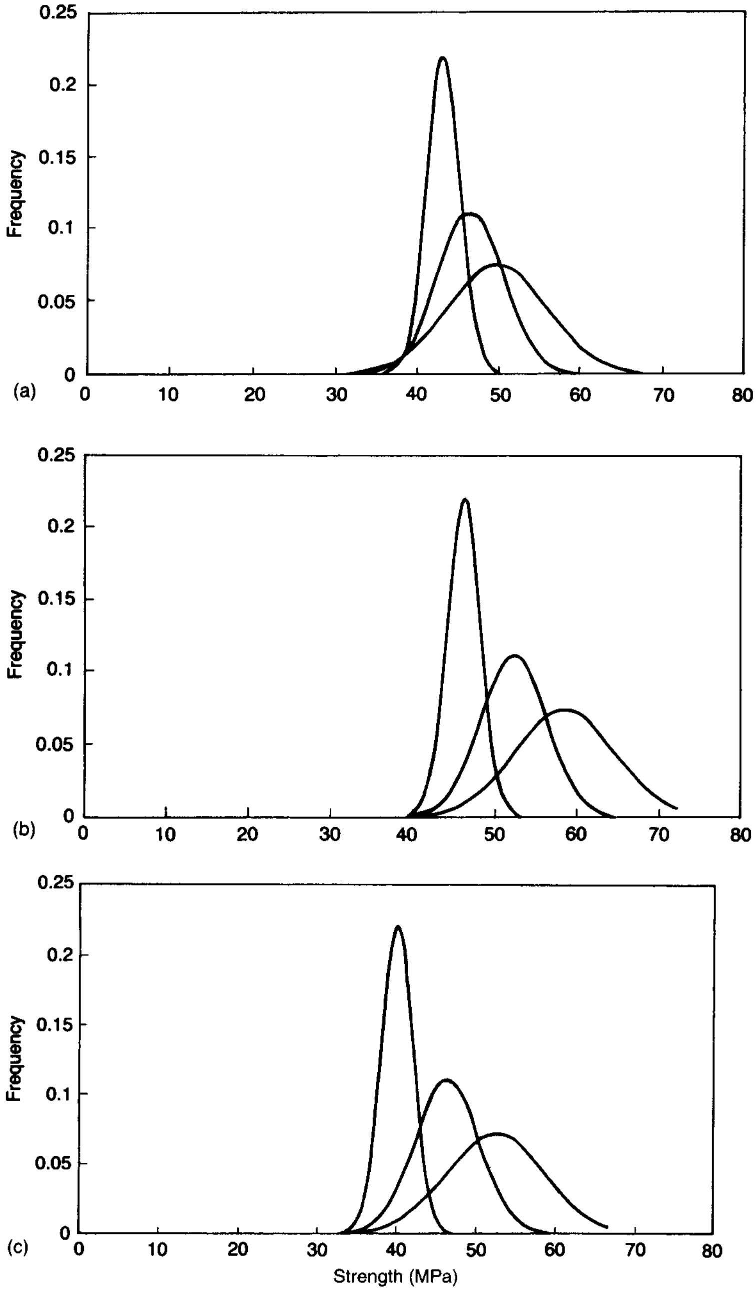


Fig. 10.5 Specification options to encourage better control: (a) common 5% below 40; (b) common 0.1% below 40; (c) common 0.1% below 34. The three distributions in each case are of SD 2, 4 and 6 MPa.

variability between change points is the basic variability of the production process. The frequency, extent, and time to react to change points depend largely on the control system, including control of incoming materials. The purchaser of the concrete will be interested in the overall combined effect of all causes of variability. However, a consideration of the worst concrete supplied would more accurately concentrate on the mean strength and basic variability between the two change points enclosing the concrete in question.

10.3
VARIABILITY OF MEANS OF GROUPS

So far we have considered only how well the assumption of normal distribution portrays the actual distribution of strength in the whole of the concrete. It is now time to consider how well an analysis of a limited number of samples portrays the distribution which would be obtained if the whole of the concrete supplied were made into test specimens and tested. It is conventional to consider that about 30 results are needed to give a reasonably accurate picture but it is instructive to look into the actual situation. One way of doing this is by the use of another distribution called the 'Student's t' distribution. This is a very useful method for evaluating comparative laboratory trials of such things as alternative admixtures or alternative cements but it will not be considered here.

If the whole of the concrete were made into test specimens and divided into groups each of n samples, the mean of each such group would in general differ at least a little from the mean of the other groups and from the 'grand mean' of all samples. In fact the means of the groups would be found to themselves be normally distributed but of course not so widely as the individual results. Statistical theory tells us that the standard deviation of the means of groups of n results is related to that of the individual results by the formula:

$$\sigma\ (\text{groups}) = \sigma\ (\text{individual})/\sqrt{n}$$

So the means of groups of 4 results will have half the σ of individual results and the mean of groups of 25 will have one fifth the individual σ.

If we take limits within which 90% of results fall (i.e. 5% outside each limit) the mean of the group of n results will be within $\pm 1.65/\sqrt{n}$ of the true value. Table 10.2 summarizes this.

At this point it is perhaps necessary to point out that the conformance of practice to theory is nowhere near good enough to justify the use of a second decimal place in Table 10.2. The object of the exercise is to get a feel for the order of magnitude of the errors involved.

It is worth noting that the variability of the results being examined has a strong influence on the accuracy with which they can be assessed. This is a generally applicable statement and is another reason for preferring low variability concrete.

It will be seen that if a single test result is obtained to represent a truck of concrete, or even the mean of a pair, the assessment will not necessarily be very precise, particularly if we are dealing with variable concrete. However, variation within a batch, i.e. within a single truckload, is a different matter to variability between batches, and is largely a matter of testing error rather than variability of concrete, (section 11.5).

Likewise if a day's supply of concrete is assessed on the basis of three samples of concrete, a considerable error may be involved.

10.4
VARIABILITY OF STANDARD DEVIATION ASSESSMENT

In a similar manner, the value of the standard deviation (σ) obtained from analysing a limited number of results will differ from the true value for

Table 10.2 Error in mean for various values of standard deviation

Standard deviation (SD) values>	*2*	*3*	*4*
No of results			
1	3.30	4.95	6.60
2	2.33	3.49	4.65
3	1.91	2.86	3.81
5	1.47	2.21	2.95
10	1.04	1.56	2.08
20	0.73	1.10	1.46
30	0.60	0.90	1.20

Table 10.3 Error in standard deviation for various values of true standard deviation

Standard deviation values>	*2*	*3*	*4*
No of results			
2	1.65	2.48	3.30
5	1.05	1.58	2.09
10	0.74	1.11	1.48
30	0.42	0.63	0.85

all the concrete. In this case the standard deviation of the distribution of standard deviations (no, it isn't a misprint!) is given by SD where:

$$SD = \sigma/\sqrt{2n}$$

A table (Table 10.3) similar to Table 10.2 can be constructed. Although these errors are a little smaller than those in the case of the mean, they are a very much larger percentage error. Note that a group of five will only yield a a value to ± 50% accuracy approximately. What this means is that the variability of a group of less than 10 results simply cannot be determined with reasonable accuracy.

This has had a profound influence on the basis of specifications because, if we persist in trying to judge the quality of concrete on the basis of a small number of samples, it is not possible to give any credit for low variability (unless this is assessed on a basis external to the group of results in question). Even the inaccuracy in the mean value noted previously is large enough to require a large tolerance if good concrete is not to be rejected and this tolerance results in excessive leniency for poor concrete (Fig. 10.5(a). However, there is no objection to framing a criterion involving the mean of the last 3, 4 or 5 results and the standard deviation of the last 10, 20 or 30 results.

10.5
COMPONENTS OF VARIABILITY

One further piece of statistical theory is needed. This is how variabilities due to separate causes combine to give an overall variability. There is a famous example of a wrong assumption about this marring an otherwise excellent paper on concrete quality control (Graham and Martin, 1946). The square of the standard deviation is called the 'Variance'. Standard deviations are not additive but variances are. This can be illustrated using the famous example in question (the standard deviations are in psi).

Source of error	**Standard deviation (psi)**
Cement *(C)*	240
Batching *(B)*	462
Testing *(T)*	188.

The overall error is *not* given by $C+B+T$=890 but by:

$$\sqrt{(C^2+B^2+T^2)}=553$$

The effect of this situation is that the contribution of all but the largest component of overall variability is reduced. Thus totally eliminating cement variability would give an overall variability of $\sqrt{(B^2+T^2)}=499$,a reduction of only approximately 10%. But in the famous paper, the variability of the cement was further exaggerated by including the error in testing the cement and it was reported that cement variability accounted for 48.2% of total variability. This was a very significant error because it suggested that much of the variability was outside the concrete producer's control. Thus one would be led to putting much of the control effort into cement testing, instead of where it was most needed (slump control).

This is a lesson which must be learned if economical control is to be achieved. The primary (largest) cause of variability must be found and control action concentrated on it (see also Pareto's principle, section 4.3.3).

Of course it is necessary to monitor subsidiary causes as well, in order to establish which is the major cause (and to check that what was initially the major cause has not been overtaken by some other cause), however the real control effort must be correctly directed.

10.6 TESTING ERROR

It has been argued elsewhere (section 11.6) that testing itself is a significant source of error on a typical project and that it must be monitored.

The author has experienced two different testing organizations testing the same truck of concrete and getting results differing by as much as 10 MPa (1450 psi) on occasions and as much as 3 MPa (435 psi) on average over a substantial number of samples (Day, 1979).

The error in question covers all aspects of taking a representative sample and casting, curing, capping and testing specimens. It is only possible to fully establish the magnitude of this error by taking two samples from the same truck and this is rarely economically practicable unless serious malpractice is suspected and is to be investigated for a short period. However, the 'within sample' error can be established providing that two (or more) specimens from the same sample of concrete are tested at the same age. The author introduced a system by which the concrete supplier's own control testing was accepted as the project control providing that he produced double sets of specimens at specified intervals and delivered them to an independent laboratory for test. This is much more economical than having an independent sampler on site and avoids the concrete supplier claiming that the independent samples have been incompetently sampled, cast or field cured. The only remaining problem is that someone has to ensure that the selection of trucks for test is unbiased. This system is highly recommended wherever there is any concern about the veracity of the supplier's own testing. However, the net result is often that the supplier's testing is seen to be acceptable and comparative testing is discontinued.

It has been pointed out that even five specimens would not permit a meaningful direct determination of standard deviation for a single sample. However, another piece of statistical theory provides the information that the average difference between many pairs of specimens from different samples is related to the within sample standard deviation by the simple equation:

Within sample standard deviation = average pair difference /1.13

(In the case of sets of three specimens, the difference between highest and lowest, i.e. the range, may be used in the same way and in this case the 1.13 becomes 1.69).

Generally there is no point in converting to standard deviation for our purposes and the average pair difference is directly monitored. The best achievable average pair difference on normal concrete is 0.5 MPa (say, 75psi) and between 0.5 and 1.0 MPa can be considered acceptable. However, the author has encountered laboratories of high repute with a pair difference consistently in excess of 1.5 MPa. The seriousness of this situation can be appreciated when it is realized that even this figure does not include sampling error and that a really top class producer can work to an overall standard deviation of concrete quality below 2.0 MPa. As discussed above we must not fall into the error of saying that testing is three-quarters of the total variability (and remember the 1.13 factor) but nevertheless such testing is grossly unfair to the producer.

10.7 COEFFICIENT OF VARIATION

Another measure of variability is the 'coefficient of variation'. This is the standard deviation divided by the mean strength and expressed as a percentage. The question is which of the two parameters best measures relative performance on different grades of concrete. The argument resurfaces from time to time even though in the author's opinion general agreement that standard deviation should be used was reached in the 1950s. The author has personally monitored thousands of test results covering 20, 25, 30, 40 and 50 MPa grades of concrete from the same plant over long periods of time. There has never been any question in his mind that standard deviation remains reasonably constant over the 20 to 40 MPa grades (i.e. mean strengths from 25 to 45 MPa or 3600 to 6500 psi). This opinion was formed in the early 1950s when he consistently achieved a standard deviation of less than 250 psi on very tightly controlled factory production with a mean strength in excess of 9800 psi. This was certainly abnormal concrete produced in tiny quantities and, being of earth dry consistency, visual water control was very easy. However, if this figure is expressed as a coefficient of variation of less than 3%, it would represent a standard of uniformity impossible to achieve on concrete of normal strength, even under laboratory conditions.

The above firm opinion, even allowing for the quoted high strength experience, must be tempered by an acknowledgment that a slightly higher standard deviation is normally experienced on 50 MPa and higher grades. This appears to be largely due to the greater difficulty in achieving accurate testing, perhaps in turn due to the different mode of failure of higher strength concrete (where bond failure, or even aggregate failure, rather than matrix failure tends to be experienced). The increase in both average pair difference of specimens and overall concrete standard deviation is of the order of 0.5 to 1.0 MPa.

Since publication of the first edition interesting further evidence is to hand. The Petronas Towers project (the world's tallest building, in Kuala Lumpur, Malaysia) involved more than 40 000 cm^3 of 80 MPa grade concrete. Being under a UK type specification, this required a mean strength of approximately 100 MPa (cube, at 60 days). It can be imagined that, in view of the importance of the project, the initial concrete supply was at a conservatively high mean strength of just over 110 MPa. This caused the overall standard deviation for the whole of the 632 samples tested at 56 days to be inflated to 4.7 MPa. However, when things had settled down later in the project, a run of 237 consecutive results gave a standard deviation of 2.8 MPa with a mean strength of 99.3 MPa.

An even lower SD value of 2.6 MPa on 80 MPa concrete for the Chateaubriand bridge is reported (de Champs and Monachon, 1992).

Set against these figures are the decisions of ACI Committees 212 (Mixture Proportioning), 214 (Evaluation of Test Results), and 363 (High Strength Concrete) to adopt coefficient of variation as the meaningful index of variability. The leading advocate of this view was Jim Cook but of course, the decision was that of the committees as a whole. A recent paper by the author (Day, 1998b) suggests that high strength concrete offers more scope for increased variability if either the testing process or the regulating analysis system is of less than the highest standard, but does not necessarily have higher variability. Cook's view is that lower coefficients of variation on high strength concrete are obtained simply because the producer is trying harder than with his normal concrete. This contrasts with the often expressed view that a producer makes his reputation on his high strength concrete but his profit on his low strength concrete. For this reason, Australian concrete producers are certainly trying very hard to achieve low variability on their low strength concrete. However, it may well be the case in the USA, where specifications often do not allow the producer to derive any financial benefit (i.e. any cement reduction) from the attainment of lower variability.

The author's strong advocacy of standard deviation as the measure of compressive strength variability does not mean that the coefficient of variation is a useless parameter. Obviously the same standard deviation cannot apply to such variables as tensile or flexural strength, much less to slump or density. A 5 to 10% coefficient of variation in anything generally represents a variable under reasonable control although, for example, a modern batch plant can achieve much better than 1% in cement batch weight (if properly maintained).

10.8 PRACTICAL SIGNIFICANCE

The most obvious point emerging from the foregoing text is that it is not feasible to take a quantity of concrete small enough to be regarded as a unit for purposes of acceptance or rejection and to represent it by a sufficient number of test results to assess its quality with reasonable accuracy. (The fact that it is also economically ridiculous to consider physically rejecting concrete which is only slightly understrength is only another nail in the coffin.) Since the future progress of the concrete industry depends on encouraging reduced variability, it is absolutely essential that quality be assessed on the basis of a large enough pool of results to enable not only mean strength but also variability to be accurately assessed. Since not even a madman would consider rejecting a month's concreting because it is slightly understrength, there is simply no other way to go than cash penalties or cash incentives (although it is feasible for the real diehards to impose this penalty in the form of increased cement content or increased testing as noted in Chapter 6).

The next point is that we do not wish to sit back and watch the contractor dig his financial grave for a month or so without taking any action. An eventual cash penalty may bring justice to the situation and may avoid him repeating his error, but it will not provide the quality of concrete required in the current structure. Therefore a method of closely monitoring the situation and taking early action to revert to the desired quality is very desirable. This used to mean keeping a graph known as a Shewhart QC chart, however these have been superseded by cusum control charts in the author's system.

As we have seen, a substantial error is possible in assessing the standard deviation, mean and 5% minimum of a small group of results, so that they cannot be used with any degree of fairness to reject or penalize. Nevertheless more than 50%, perhaps as much as 70 or 80%, of such assessments are quite realistic. They are therefore very useful as a guide to the state of affairs provided they are used only as a warning that the situation should be carefully considered and not as a basis for precipitate action. Having isolated the rigid legal requirement as based on an unquestionably accurate assessment of a large quantity of results, it is then possible to informally consider a large number of factors in deciding when a small mix adjustment may be desirable. There will be scope for a small difference of opinion between concrete producer and supervisor from time to time but the latter can afford to concede graciously and wait for the fullness of time to bring retribution if it was merited, secure in the knowledge that the quality shortfall will be minor and the retribution precise, inevitable and indisputable.

A very interesting matter is a comparison of the standard deviations considered normal in Australia and the UK. The author has for many years considered 3 MPa (say, 450 psi) to be a normal figure for an average ready mix plant in Melbourne. Of recent years the better practitioners are attaining 2 MPa, or even fractionally less. In the UK, a figure of 4 to 6 MPa is considered normal. It is not likely that physical control of production is genuinely twice as good in Australia and an explanation is likely in the statistical concepts applied. In the UK, results are corrected or normalized according to cement content so as to provide a basis for combining results from different grades. It would appear that this does not work very well. Having created an artificially higher variability in this or some other manner, the task of detecting change becomes more difficult. When a rigid mathematical requirement (in the form of a V mask) is applied to determine whether an adjustment should be made, the difficulty is compounded. When adjustment is delayed in this manner, a genuinely higher variability is created or allowed to continue. This question is further examined in section 12.6.

11
Testing

11.1
RANGE OF TESTS

A very large number of tests on concrete have been devised. A partial list is given below.

1. Tests on hardened concrete:
   - Compressive strength (cylinder, cube, core).
   - Tensile strength:
   - (a) direct tension.
   - (b) modulus of rupture.
   - (c) indirect (splitting).
   - Density.
   - Shrinkage.
   - Creep.
   - Modulus of elasticity.
   - Absorption
   - Permeability.
   - Freeze/thaw
   - resistance.
   - Resistance to aggressive chemicals.
   - Resistance to abrasion.
   - Bond to reinforcement.
   - Analysis for cement content and proportions.
   - *In situ* tests:
   - (a) Schmidt Hammer, pull-out, break-off, cones, etc.
   - (b) ultrasonic, nuclear.
2. Tests on fresh concrete:
   - Workability (slump and over 20 other).
   - Bleeding.
   - Air content.
   - Setting time.
   - Segregation resistance.
   - Unit weight.

Wet analysis.
Temperature.
Heat generation.

Of these many possible tests, in practice well over 90% of all routine tests on concrete are concentrated on compression tests and slump tests which should be, but are not always, accompanied by fresh concrete temperature and hardened density determinations.

Before considering whether this is a desirable state of affairs, it is first necessary to consider the purpose and significance of the testing.

There are at least three possible purposes:

1. To establish whether the concrete has attained a sufficient maturity (for stripping, stressing, de-propping, opening to traffic, etc.).
2. To establish whether the concrete is basically satisfactory for the purpose intended.
3. To detect quality variations in the concrete being supplied to a given specification.

It is very important to be clear about the purpose of the testing because attempts to fulfil all these purposes simultaneously usually lead to inefficiency in fulfilling any of them. The true essential purpose of the majority of tests is the detection of quality variations.

The selection of compressive strength for the great majority of control testing relies upon two basic assumptions:

1. That all or most other properties of concrete are related to compressive strength.
2. That compressive strength is the easiest, most economical or most accurately determinable variable amenable to test.

The second of these assumptions will be examined in detail later.

The first assumption is probably correct in so far as the purpose of the test is to detect quality variations but is not necessarily correct if the purpose is to establish whether the concrete is basically satisfactory (for example, shrinkage may increase as compressive strength increases if the strength increase is obtained by increasing cement content but would reduce with increasing strength if this was obtained solely by reducing water content).

It may well be impracticable on most projects to use other forms of test for quality control purposes (although rapid wet analysis has been so used). However, especially where we are dealing with standard mixes from a premix plant or a special mix designed for a specific purpose, it is certainly practicable to carry out a much wider range of tests to initially verify a new mix design and to repeat a wide range of tests at say annual, or six monthly, intervals for standard mixes. An excellent example of this is the shrinkage of concrete in the Melbourne, Australia, area. For many years structural designers had been concerned about excessive shrinkage but the only action resulting from this concern was to prohibit the use of pumped concrete on some projects and limit sand percentages on others. However, in 1977/8 the Australian Government CSIRO (Commonwealth Scientific and Industrial Research Organization) carried out shrinkage tests on a range of standard Melbourne area pump mixes and showed a wide range of variation with clearly definable causes. It then became practicable to specify a limiting shrinkage and in most cases to permit the use of pumped concrete since the tests showed that some pumped mixes had a lower shrinkage than some non-pump mixes (the factor involved being the influence of the coarse aggregate).

Similar action is now needed in respect of splitting strength, permeability, durability, abrasion resistance and also workability (other than slump), segregation resistance, bleeding and surface finish characteristics. These were all matters on which we were flying as blind as we used to be on shrinkage at the time of writing the first edition. In the intervening five years there has certainly been substantial action in respect of durability and permeability (with the latter seen as the best available criterion of the former). With a 100 year durability requirement specified by the client for a major project in Melbourne, the author translated this into a maximum VPV of 9%. Here VPV is volume of permeable voids and is determined by the loss of weight on drying an initially saturated sample of concrete. However, this basis was chosen because there was no local experience of other techniques such as the James Instruments adaptation of the two Figg tests or the UK Wexham Developments variant of this type of equipment.

11.2 PERMEABILITY TESTING

The original Figg tests originated in the UK but have subsequently been neatly combined into a single instrument by James Instruments in the USA. A hole is drilled into the concrete (which may be *in situ* concrete or a test specimen) and a plastic plug inserted to create a cell below the surface of the concrete. A hypodermic needle is inserted through the plug to provide access. The first test involves applying a suction to the cell so as to draw in air through the surrounding concrete. The (very small) volume of air is measured by the movement of mercury in a tube through which the suction is applied. The second involves filling the cell with water and using movement in the same tube (but in the opposite direction) to measure the rate at which water is absorbed into the surrounding concrete.

The Wexham variant identifies two problems sometimes encountered with the above-described test. One is that air permeability is substantially affected by moisture content. The other that air may be entering via defects in the concrete or a leaking plug rather than via permeable concrete. These two potential problems are solved firstly by using a slightly larger diameter hole and including an instrument to measure humidity in the hole. Secondly, pressure rather than suction is employed so that any leaks can be detected by bubbles in a soapy water film on the surface.

An additional advantage of these kinds of *in situ* test are that they can be used to measure the adequacy of curing (which has a large effect on permeability). Potentially a contractor could be required to continue or resume water curing until an acceptable permeability is achieved.

11.3 COMPRESSION TESTING

Considering now the accuracy and convenience of compressive strength as a routine control, the situation is not so simple as was thought 20 years ago. In Australia we are fortunate to have the world’s first and most highly developed National Association of Testing Authorities (NATA). We have a better system than most other countries for ensuring that test specimens are cast by competent persons, taken to laboratories with satisfactory curing facilities, capped with a sound cap and tested in a standard manner in a properly calibrated and maintained testing machine. Without being able to quote chapter and verse, but having used both extensively, the author is also coming to the view that the cylinder specimen is more reliable than the cube specimen. Nevertheless it has become apparent that NATA certification is not sufficient to ensure that different laboratories obtain essentially the same test strength on concrete from the same truck of concrete.

Isolated differences of over 10 MPa and consistent differences of the order of 2 to 4 MPa have been documented in Melbourne (Day, 1979, 1989).

There are two aspects to the problem:

1. The technology of compression testing machines.
2. Day-to-day performance variation.

11.4 TESTING MACHINES

A compression testing machine is usually by far the most expensive item in a routine concrete QC laboratory. As such machines are also very durable items, there is a tendency for quite antique versions to be still in service (and indeed they may give better results than a cheap new machine).

It is apparently an extremely simple thing to apply a compressive load to a test specimen using a hydraulic ram. However in practice it is far from simple because the results obtained must be very consistent and must bear comparison with other testing machines.

The author has had a wide experience of operating different classes of compression testing machine over many years, but such general experience is of little value. What matters is access to comparative results on samples from the same truck of concrete and preferably cast by the same person. A requirement that this be done as a regular routine has been part of the author's standard specification for some years and such data is therefore available covering a number of different pairs of laboratories. The Australian National Association of Testing Authorities also organizes occasional comparative tests in which a large number of specimens are cast from a single truck of concrete and distributed to many laboratories. There is a distinct difference in the extent of variation found when each laboratory is 'on its mettle' in a major isolated comparative exercise and that found when the comparison is under everyday routine conditions. In the latter case individual samples can differ by more than 10 MPa (1500 psi) and a consistent average difference of up to 2 MPa (300 psi) can be experienced over a long period. These matters have been reported by the author in two papers to ACI Conventions (Day, 1979, 1989).

A 2 MPa strength difference is equivalent to a cement content difference of between 10 and 20 kg/m^3 (17–34 lb/yd^3). A single testing laboratory may well be controlling a production of 10000 to 100000 m^3 of concrete per month (from several plants). So that the 'high' cost of a testing machine may be little more than the difference in the cost of cement requirement according to two different machines *per month*.

11.5 TESTING MACHINE TECHNOLOGY

Obviously a correct result will not be obtained unless the stress is uniformly distributed over the test specimen (and any incorrectness in this respect will lead to a lower result).

An assumption is made that the faces of both the test specimen and the testing machine platen are absolutely plane and that the load will be applied concentrically. Quite small differences in planarity can make very large differences in contact area and therefore in stress distribution. With cube specimens this problem will worsen with older and higher strength specimens because the older concrete (i.e. 28-day rather than 7-day) will be more rigid, i.e. less subject to plastic distortion. With cylinders the problem is different. Here the capping compound (e.g. where sulphur caps are used) will flow equally at any age. The platen planarity may be slightly less critical but any plastic flow allows stress concentrations to develop unless the original cylinder ends are very close to flat.

Spherical seatings are provided to allow one platen to rotate to compensate for any tendency for the two opposite faces of the test specimens not to be exactly parallel. This introduces its own problem in that, if the spherical seating were effective during the whole test, any eccentricity at all would lead to a bending moment in addition to an axial force, so reducing the failure load. **Therefore spherical seatings must be lubricated with a very light machine oil specifically so that the oil will break down under pressure** and allow the seating to lock solid after an initial adjustment. Extreme pressure lubricants, such as graphite grease, must be avoided as they will produce lower and more variable results. For cubes this is even more important because, since the specimen is tested perpendicular to the direction of casting (and therefore water gain or bleeding), its physical centre may not be its 'centre of resistance', i.e. if the cube is stronger at the bottom than at the top, its centre of resistance would be displaced towards the previously bottom face when turned on its side for testing.

A further influence of the platen/specimen interface, again especially with cubes, is that friction provides a lateral restraint to the Poisson's ratio spreading effect and so increases the test strength. The author (inadvertently) demonstrated this many years ago when he tested cubes coated with a wax curing compound. The compound may have increased the actual concrete strength but it certainly caused a drastically reduced load at failure. The reason for test cylinders to have a height/diameter ratio of 2 is to avoid this effect in the central area where failure actually takes place. This is probably the main reason for the difference between the test strength of cubes and cylinders from the same concrete. It may also be the reason why this effect is reduced at higher strengths (?). However, a further reason is that bleeding voids, which are more likely at lower strengths, may have a greater effect on cubes than cylinders owing to the different orientation during testing.

11.6
BAD CONCRETE OR BAD TESTING?

The author was invited to give a paper on the above topic to the 1989 ACI San Diego Convention (Day, 1989). The paper has not been published, but the conclusions presented, and the fact that an ACI session organizer **requested** a paper on this topic indicates that the question merits close attention.

The first half of the paper presented factual data showing that it is far from a reasonable expectation that a properly presented result from a reputable testing laboratory will be a necessarily accurate representation of the quality of the concrete. Examples were provided of individual differences exceeding 10 MPa, and consistent average differences of up to 2 MPa, in the results obtained by different registered laboratories testing the same trucks of concrete. It was emphasized that the laboratories concerned were NATA approved.

Pair differences exceeding 5 MPa were noted for apparently identical test specimens from the same truck of concrete tested by the same laboratory. Also 7 to 28-day strength gains were shown to be capable of ± 50% variation from sample to sample of concrete of the same mix design using the same materials.

The clear conclusion was that a strength test result is a totally unreliable piece of information. The audience awaited the author's proposal of some more satisfactory means of assessing concrete quality than a compression test.

The second half of the presentation showed that the very same data used in the first half could be analysed to show quite accurately when a genuine change in concrete quality occurred. Cusum graphs of 7 and 28-day strength showed downturns and upturns on exactly the same dates in spite of individual differences. The two laboratories showing the large differences on individual samples nevertheless agreed exactly as to when these change points occurred.

The overall conclusion presented was that an appropriate analysis of a series of test results can yield very reliable conclusions but that any individual test result should be regarded with great suspicion.

Some of the conclusions presented were:

1. Concrete producers are not so good that it is unnecessary to test concrete nor testing laboratories so bad that it is ineffective to do so.
2. There is no better complete replacement for traditional cylinder testing because it is the only way in which the combined effects of batch quantity variation, material quality variation, silt and dust content variation, air content and temperature variations, delivery delays and added water effects can be integrated.
3. We must cease to think of a single test result as an invariably accurate judgment as to whether a particular truck of concrete is or is not acceptable. In the first place it may well not be accurate, **and in the second we should show as much concern for those trucks we did not test as for those we did test.**

   Rather we should regard the analysed pattern of test results as an important part (but only part) of the evidence we require in order to establish whether the totality of concrete being delivered to the project (or leaving the plant) is or is not of the required quality.
4. Before concrete of a particular grade is even ordered, it should be established that it is almost certain to be satisfactory. This may be done on the basis of trial deliveries, laboratory trials, analysis of past data or even just the reputation of the supplier. This assessment needs to take into account variability as well as mean strength. For an important project it may be inadvisable to obtain concrete from a supplier who cannot show either or both of substantial analysis of past data showing low variability and/or a computer batching plant which records the actual batched weights of every truck load delivered.
5. A particular individual (perhaps with assistants on a major or widely spread project) should have the responsibility of visually inspecting every truck of concrete and rejecting or further testing any suspect loads.
6. When a truck is sampled and test specimens cast, there should normally be at least three specimens. This is to permit an early age test and a pair of 28-day tests. The early age (not later than 7 days) is because any necessary mix adjustments must be carried out long before 28-day tests are made. The 28-day test is necessary to establish the current significance of the early age results. Two 28-day specimens are needed partly because the average pair difference is the best measure of testing quality and partly so that one can be brought forward to confirm or amend a low early age test result.
7. The sampling procedure should also include measuring and recording slump and concrete temperature, and also cylinder density on receipt at the laboratory. This is because such information is less expensive to obtain than the compressive strength yet at least doubles the value we can extract from it. Entrained air tests are also useful but this test is little more expensive so it is not invariably justified. Shilstone (1987) has suggested that the fresh density of concrete may be a better quality indicator than slump. If taken it should certainly be combined with an air content determination, but it involves on site weighing equipment and it is not so simple to attain the required precision. Also it is not such a direct check on the relative water content of successive loads. It may be that hardened specimen density is sufficient providing that it is measured on receipt of the specimens at the laboratory (i.e. within 24 hours) and that it is immediately followed up by air testing when a significant density change is experienced. It

may be that fresh density measurement is mainly of use if rejection of trucks is contemplated, but this should be abnormal.

8. The test results should be analysed to detect, at the earliest possible time, any departure from the previously acceptable concrete properties. This can best be done by drawing cusum graphs of early age and 28-day results, slump, temperature, cylinder density, 28-day pair difference and early age to 28-day strength gain.

   Such graphs are of substantial value not only in showing a strength downturn quickly and obviously but also in making it much easier to see whether the downturn is due to basic concrete quality, weather conditions, site abuse (excessive waiting time, water addition, etc.) or only the testing process.
9. It is very desirable to separate the functions of mix amendment and contractual acceptance. Mix amendment should take place based on early age results and can be reversed without excessive cost having being incurred if found unnecessary a few days later. It can therefore be done on relatively slender evidence. Contractual acceptance is best regulated by a cash penalty or cash bonus based on a statistical analysis of at least thirty 28-day results.

   Physical rejection of hardened concrete, or even its further investigation by coring, etc., should be totally unnecessary if these recommendations are followed. One very desirable result of a cash penalty/bonus specification is that it avoids any need to argue about a possible mix amendment based on slender evidence at an early age. The decision can happily be left to the supplier as it is his penalty/bonus which is at risk rather than the structural integrity of the concrete.

The implementation of the above principles enables excellent control of concrete quality at very low sampling frequencies. The reduced volume of testing easily pays for the analysis but much larger savings are made by the elimination of disputes, investigations, delays to program, rejections, etc. The paper certainly did not advocate a greater expenditure on control by adding the cost of elaborate analysis to the cost of the present level of testing. The proposal was rather to minimize the total cost of a given degree of assurance of concrete of a given minimum quality. This cost includes the necessary minimum cost of the concrete, any extra costs imposed by restrictive specification requirements, the cost of testing, the cost of test result analysis and any costs imposed by failures, including further investigation, partial demolition, legal costs, program delays and wasted time in meetings.

11.7 ROUNDING RESULTS

It is extremely bad practice in any technical field to fail to recognize and take account of the inaccuracies inherent in test results. One aspect of this is to avoid expressing results to more significant figures than their accuracy justifies.

In accordance with this various authorities **require** that certain test results be rounded. An example is the Australian NATA which requires that compression test results be rounded to the nearest 0.5 MPa (=75 psi) and densities to the nearest 20 kg/m^3 (approximately 1 lb/ft^3). The author believes that this practice requires reconsideration.

Take compressive strength. Why should 0.5 MPa be selected? The answer is not that is the order of accuracy, because different (competent) laboratories can easily differ by 2 MPa and **average** pair differences can exceed 1 MPa. Rather the answer is that in the days before computers were used, results were ‘worked out’ from tables and 0.5 MPa steps gave about as large a table as was convenient. The tables would have been five times as large had 0.1 MPa been selected.

The important question is what use is to be made of the test result. Originally the answer was to accept it as totally accurate and reliable and compare it to the specified strength. From this viewpoint it should certainly be taken as ±2 MPa and so labelled.

It is bad practice to round calculations before the very last step. The strength of the individual specimen **used** to be the last step but now we have hopefully realized that this should no longer be the case. Action on compressive strength results should always be based on the analysis of groups of test results, effectively ignoring individual results. So it is the mean and standard deviation of a number of results which has significance. It would be better to use less rounded results, but it may not make a great deal of difference. However, when analysing (as we should) such items as within sample ranges (based on average pair differences) and 7 to 28-day strength growth, rounding to 0.5 MPa is fairly obviously unsatisfactory.

It is proposed for compressive strength that it be expressed to 0.1 MPa and given the written qualification '±2 MPa' where appropriate. This (apart from the ±2 MPa) will not consume any more paper and will marginally reduce the computer program.

For density a similar situation exists. It is not so much the absolute density of a single specimen which should be of interest, but the range of densities of all specimens from a single sample of concrete (since this will reveal the competence of the specimen casting and enable its variation to be monitored). Detecting any change in the average density of concrete being produced, i.e. of a group of samples, is the major reason for the test.

The proposal for density is that it be expressed as a four digit integer, since again this takes marginally less computer effort and no more paper. The accuracy limits in the case of density may be much different for different organizations. For those to whom it matters, their control system will be providing a within sample standard deviation. Density may be unlike strength in that small variations in assessment of the same concrete by different laboratories is probably unimportant. Detection of change in average density or change in within sample variation are probably what matters.

11.8
CUBES *VERSUS* CYLINDERS

The world is divided as to whether it is better to assess concrete strength by cube or cylinder specimens. The UK, much of Europe, the former USSR and many ex-British colonies use cubes, but the USA, France and Australia use cylinders.

The advantage of cubes is that they are smaller and do not require treatment (capping) prior to testing.

The advantage of cylinders is that they are less dependent upon the quality and condition of the moulds and that their density can be more accurately established by weighing and measuring.

Both proponents naturally feel that the specimen with which they are familiar is preferable. The debate should be settled on the basis of which gives the most accurate (i.e. repeatable) result. This is best judged by the average pair difference achievable, or the average range of three. Either of these can be converted into the **within sample (sometimes called within test) standard deviation**. In the case of pairs the average pair difference is divided by 1.13 to obtain the within sample σ. For the average range of sets of three, the divisor is 1.69.

The author received his initial concrete QC experience in the UK on cubes and has owned and operated testing laboratories in Australia using mainly cylinders and in Singapore using mainly cubes. Both specimens are perfectly satisfactory and capable of very low pair differences if used carefully and cast in well-maintained moulds. The problem is that the test specimens must be prepared in the field by relatively low level technicians. The quality of training provided is crucial and is often inadequate. The really basic fault is often that the persons training the technicians have inadequate knowledge, practical experience, or dedication to the task.

Capping used to be something of a problem with cylinders, although more of an initial than a continuing problem. Once the proper equipment is obtained and the operator has gained experience, capping was never much of a problem. The capping referred to is the use of a molten sulphur mixture to achieve a smooth test surface on the end of the cylinder.

The essential items are:

1. A heavy, accurately machined steel mould into which to pour the sulphur mixture.
2. A guide along which to slide the cylinder to ensure the cap will be perpendicular.
3. A thermostatically controlled melting pot in which to heat the sulphur mixture.
4. A scoop holding an exactly suitable amount of the mixture to produce a cap.

There are a number of difficulties to be overcome by the uninitiated:

1. Neat (undiluted) sulphur is not suitable because it shrinks too much and sets too quickly. A mixture with finely ground silica, fly ash or other inert material should be used. Proportions are trial and error, depending on the particular sulphur and the particular filler. Some like to include a proportion of carbon black. Commercial blends are available.
2. The temperature of the mixture must be ‘just right’: too cool and it will not flow and sets too quickly giving a thick cap, too hot and it goes rubbery and shrinks too much. Again, it is trial and error.
3. The first cap is difficult because the mould is cold, later the mould gets too hot and causes delay waiting for setting.
4. The mould must be very lightly oiled between each use.
5. The cap must be thin, preferably only 2 to 3 mm.
6. Especially for high strength concrete, a sulphur cap will not overcome a rough cylinder end. The cap will exhibit slight plastic flow under load and allow load concentration on high spots.
7. The hot sulphur emits fumes and requires at least an exhaust fan and preferably a fume hood.

All the above makes it quite clear why users of cubes are not tempted to turn to cylinders but has no bearing on the question of which is the more reliable test.

A significant improvement is that of rubber caps. Instead of sulphur capping, the cylinder is simply fitted with a rubber pad restrained in a metal mould. A suitable side clearance is essential since, under the high pressure, the rubber behaves almost like a fluid. If the clearance is too great the neoprene will be extruded and will provide excessive side restraint. The mould is illustrated in Fig. 11.1.

A recent development which could be very important is a new capping technique called the ‘sand box’. The test was developed by Claude Bouley and Francois de Larrard and was reported in *Concrete International* (Bouley and de Larrard, 1992). The ‘box’ in question is a circular cup, very similar in appearance and function to the restraining ring used in the rubber cap test but deeper (30 mm). The rest of the apparatus is a positioning frame and guide similar to that used in sulphur capping, except that a small, air driven vibrator is incorporated. The technique is to place a 10 mm layer of dry sand in the cup, position the cylinder in the frame and vibrate so that the cylinder compacts the sand (20 s). The cylinder is then sealed into the cup by filling around the periphery with molten paraffin wax.

The test may initially look unattractive compared to sulphur or rubber caps since it involves a capping process with molten material, a vibrator, and does not permit re-use of the mould before testing. However,

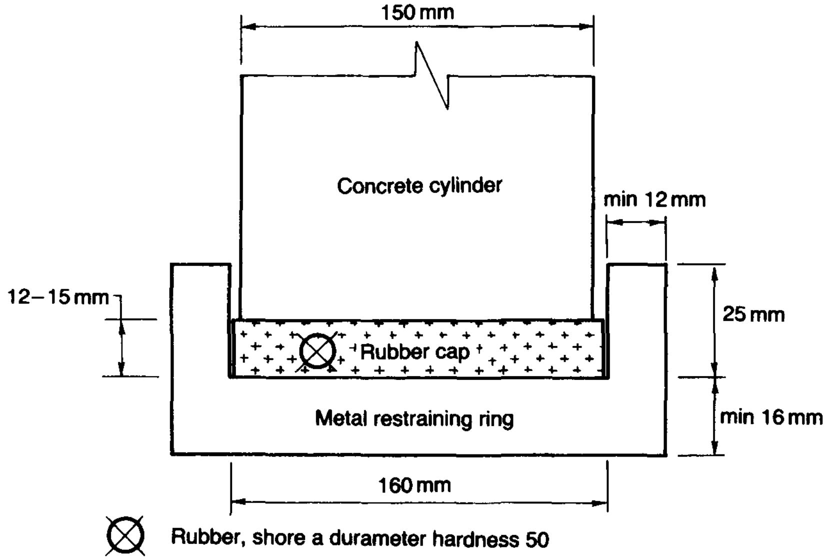


Fig. 11.1 Rubber cap and restraining ring.

it does not appear to involve as much manual dexterity as sulphur capping, avoids sulphur fumes and permits immediate testing of a prepared specimen. It uses only sand and recyclable wax and so should be inexpensive in use. More importantly, it appears to give test results on very high strength concrete only slightly less reliable than the best achievable by end grinding and much better than even slightly substandard grinding. The trials have included successful use on extremely rough cylinder ends which would have had to be sawn off before any other technique could have been used.

The use of large aggregate concrete, except for special uses such as dams, is becoming rare. For high strength concrete, aggregate with a maximum size of more than 20 mm (¾ in) is a disadvantage and for very high strengths a smaller size still, 10 to 14 mm (⅜ to ½ in) gives better results. Therefore previously used specimen sizes of 150 mm (6 in) cubes and 150 mm diameter×300 mm long cylinders can be replaced by 100 mm cubes and 100×200 mm cylinders. Some researchers consider that the smaller specimens will give higher strength (up to about 5% higher) and greater variability. Others find that smaller cylinders give **lower** variability, but the differences are not sufficient to concern us unless they affect a comparison between different laboratories.

Whilst considering such matters, reference must be made to the cube/cylinder ratio. The British Standard nominates this ratio as 1.25 for all circumstances but this is not the author's experience, which is that the ratio varies from over 1.35 to less than 1.05 as strength increases. A formula giving results in accordance with the author's experience, but not claimed to be thoroughly established, is:

$$\text{Cube strength} = \text{cylinder strength} + 19/\sqrt{(\text{cylinder strength})}$$

or

$$\text{Cylinder strength} = \text{cube strength} - 20/\sqrt{(\text{cube strength})}$$

where cube and cylinder strengths are both in MPa or N/mm^2.

Table 1.2, gives an alternative version which has greater official standing.

The smaller cylinders, which weigh around 4 kg rather than 13 kg for the larger ones, are much easier to handle and cap.

11.9 NON-DESTRUCTIVE TESTING

With non-destructive testing (NDT) it is necessary to be particularly careful to clarify the objectives of the testing and the assessment of the results. Clearly the strength of the concrete in the structure is not necessarily the same thing as the potential strength (according to a standard compression test) of the concrete as it leaves the mixer or delivery truck. If it is not clear which of these is being sought, it is unlikely that the relative merits of different testing procedures will be correctly assessed.

In one way, the strength of the concrete in the structure is what really matters. However, even if this is accepted, we still have to consider whether what matters is the **current** strength of the concrete in the structure or its **eventual** strength. If the requirement is to assess readiness for early stripping or prestressing, or termination of curing protection, then the current strength is the more important. If it is the load carrying capacity of the structure, or its durability, then the eventual strength will probably be more significant.

If the intention is to regulate the proportions of the concrete mix currently being produced, it is equally not obvious whether the potential standard specimen strength or the current actual strength in the structure is what matters. If considerations of eventual strength and durability in a particular structure require a 30 MPa (4350 psi) strength but construction efficiency requires 22 MPa (3190 psi) at 22 hours for prestressing, then the latter requirement will clearly rule. If day to day temperatures vary very widely (as they do in parts of Australia) then it could be necessary to supply concrete of 40 MPa (5800 psi) 28-day strength one day and 60 MPa (8700 psi) 28-day strength the next. Of course it is always possible that it is economically preferable to supply 60 MPa throughout, rather than complicate the situation, but this option can be ignored for the purposes of this example.

In the more usual case, a particular concrete mix will have already been assessed as suitable for its intended purposes and testing will be being undertaken only to determine when any change takes place in that mix. In this case any extraneous factor which affects the test result, such as variable compaction of the test specimen, or variable temperature either of the supplied concrete or of the specimen during curing, will add to apparent variability and so reduce the efficiency of the control process.

Assessing the above range of possibilities, it appears that the only case in which NDT testing could be considered as a total replacement for typical compression testing of standard specimens is where an early age requirement ensures such a large excess of 28-day strength that control of that strength is unnecessary. Even in this circumstance, standard testing may still be desirable if any problems are encountered, as otherwise it may be difficult to establish whether the problems are mix problems or usage problems. To some extent the decision would depend on the quantities of concrete involved since the cost of control measures may be to a large extent ‘per pour’ whereas the cost of providing excess strength to avoid or reduce control is definitely per unit volume of concrete. Thus if a few small units totalling, say, 1 m^3 of concrete per day were involved, it would be economical to use an excessively high strength and do little testing of any kind. However, if 200 m^3/day were used in floor slabs to be prestressed at an early age, both specimen testing and some form of *in situ* testing would be obviously justified.

An important consideration is that it is not only the accuracy of a test which matters but also its relevance and the accuracy of the assumptions made in evaluating it. For example a test cylinder left on an *in situ* slab may give a very accurate strength but may have a very different maturity and therefore a very different strength to the slab itself. A pullout test on the same slab may be much more variable but at least it is measuring the actual strength. A standard test cylinder combined with a maturity (e.g. equivalent age) measurement of both the cylinder and the slab **may** be more accurate than the *in situ*-cured cylinder, and as relevant as the pullout test, but it does depend on the accuracy of the **maturity/strength** correlation and, for example, the compaction of the slab. An ultrasonic test would also be very relevant and may be quite repeatable and accurate but would be totally dependent on the strength/velocity relationship assumed, which would be affected by such factors as moisture content.

The reader is referred elsewhere (Bungey, 1989) for further details of various NDT tests but the author certainly sees a place for such tests in the overall control operation. Particular examples are pullout tests on suspended floor slabs prior to early stripping or stressing, and Schmidt Hammer tests on freshly stripped columns. The latter is not a very accurate test (especially if used informally rather than according to the manufacturer's routine) but it is an extremely quick and cheap test which could be used on every column as it is stripped and would give early warning of any severe problems. It has even been suggested that the test could be worth performing even if the strength scale is not read. The implication is that the depth of penetration, or even the sound of the impact, would alert a daily user to any drastic problem. The author found this to be the case with spun concrete pipes, where sound was a good indication and the process could be compared to tapping the wheels of railway carriages to detect cracks.

When regular NDT tests are carried out it is very desirable to enter the results in the control system for graphing and analysis alongside the other test data. Such action will soon establish the extent to which the variation of strength in the structure is a consequence of basic concrete variation.

A development which is currently being pioneered by Dr A.M. Leshchinsky is that of using multiple techniques of NDT concurrently. The idea is that whilst the correlation of any one such set of test results with compressive strength may be upset by some influence (e.g. ultrasonic pulse velocity is greatly affected by moisture content), it is less likely that two or more different tests will be similarly affected. Therefore the use of two or more techniques will give more certainty of a correct assessment than any number of repetitions of the same type of test. This is a further illustration of a point previously raised, i.e. the relevance of a test result may be even more important than its accuracy in many circumstances.

11.10 FRESH CONCRETE TESTS

Fresh concrete can be tested for workability, air content, temperature, density, moisture content and analysed to give its composition. As in most matters connected with concrete, it is again very important to have a clear idea of exactly what it is desired to achieve before deciding which tests are worthwhile and which are not.

11.10.1 Workability

A large number of tests for workability have been devised. Chapter 12 discusses the subject in greater depth, and relies heavily on a book by G.H.Tattersall (1991). At one time it was hoped that Tattersall's replacement for the slump test might eventually become universally accepted but this now seems unlikely.

The problem is that the slump test is a very widely and firmly established test but is a poor measure of the relative workability of **different** mixes. It survives because of its simplicity and robustness and also because it is (when properly conducted) quite a good measure of the relative consistency (i.e. wetness) of successive deliveries of the **same** mix. With today's much more accurate batching and using the author's MSF we can have defined and controlled the other aspects of workability so that it may now be adequate to accept the slump test as defining **consistency** for the particular mix (especially if an 'equivalent slump', adjusted for time delay and temperature, is used). What we must **not** do is to use slump in specifications on the assumption that it defines workability on an absolute scale. It may be acceptable for special purposes to specify slump limits **in addition** to precisely specifying the type of concrete required (the author does this for special wear resisting floors) but generally workability (slump or otherwise) is the business of the concreter, not the specifier. The concreter should be permitted to strike his own balance between the higher cost of more workable concrete and the reduced cost of placing, always providing that such aspects as shrinkage, segregation, bleeding settlement, etc., are given adequate consideration.

Even the above half-hearted endorsement of the slump test does have its limits. Obviously it cannot be used for no-slump (or almost no slump) concrete. Such concrete is likely to be used only in precasting factories and in such locations a V-B consistometer (AS1012.3, 1983) (in which essentially a slump test is performed in a cylindrical container and the time taken to re-form the slump cone into the cylindrical shape under standard vibration is measured) is likely to be convenient.

At the opposite end of the scale, flowing superplasticized concrete is becoming more popular and needs a flow table (DIN 1048) for accurate measurement of its workability. In this test it is the diameter of spread under a slight jolting motion which is measured.

The upper limit for which the slump test can be used is very dependent on the type of concrete. Harsh, gap-graded concrete (MSF of 20 or less, section 3.1) will fall apart on a slump test at slumps not much higher than 50 mm. On the other hand continuously graded mixes of high sand content (MSF of 27 or more) will give a measurable and reasonably repeatable slump up to 200 mm or more.

The technique of carrying out a slump test is also important in obtaining a true reading and it should be realized that the slump itself is measured in different ways in the USA, UK and Australia. These matters are also covered in Chapter 12.

11.10.2 Compacting factor

The compacting factor test has achieved a degree of success in the UK at replacing the slump test but is virtually unused commercially in the USA, Australia and SE Asia. It is a device using two hoppers mounted above each other in a frame, with the lower hopper discharging into a standard cylinder mould. The concept is that the first hopper fills the second in a standard manner and the drop from the second hopper into the cylinder mould subjects the concrete to a standardized compactive effort. The result is expressed as a proportion of full compaction achieved by dividing the weight of concrete in the mould by the weight of a fully compacted cylinder.

The test is a little more accurately repeatable and is a more absolute basis of comparison between the relative workabilities of different concrete mixes than the slump test. However, the test is not greatly superior to the slump test in quantifying variations in water content of successive deliveries of the same mix and, since it is less widely used, the author has based his system on slump.

It is again emphasized that slump plus an MSF (i.e. relative sandiness) and adjusted for time after batching and concrete temperature, is a more meaningful measure of workability than slump alone.

11.10.3 Air content

Entrained air is used for two different purposes: to improve resistance to freezing and thawing and to improve workability and inhibit bleeding.

For the freeze-thaw application a higher percentage (6 to 8%) is required than is normally used for workability improvement and bleeding inhibition (3 to 5%). At the higher percentage, entrained air costs money in the form of needing a higher cement content for a given strength and workability. At the lower percentage, and at concrete strengths of 30 MPa (4350 psi) and below, the water reduction enabled by the air entrainment may fully offset the weakening effect at a given w/c ratio. The water reduction may be of the order of 10% and the strength loss at a given w/c ratio about 5% per 1% of air entrained. It should not be forgotten that non-air-entrained concrete is likely to contain 1 to 2% of voids so that the extent of the **extra** weakening may be only 5 to 10%.

It is obviously necessary to specify the required air content where this is 5% or more, since otherwise it would be omitted on economic grounds by the concrete producer. It would also be reasonable to regularly test the air content in this case.

Where the air is not required for freeze-thaw durability, it may be unnecessary to specify it, partly because it may be provided in any case and partly because fly ash, with particles similar in size and shape to entrained air, has a similar effect (although a smaller water reduction). The amount of entrained air can be deduced reasonably accurately from the hardened density of the test specimens (cube or cylinder). When this density indicates that the air content may have changed, it may be desirable to immediately institute air content testing until the reason for the changed density is established.

11.10.4 Density

Some concrete controllers like to carry out regular fresh density testing. It is certainly true that there is often a good correlation between strength and density for a particular mix. However, as noted earlier, the density of hardened test specimens on receipt at the laboratory may be an adequate substitute for routine control purposes. Where the purpose of the density test is to settle a dispute on the yield of the mix (i.e. whether a nominal cubic metre is in fact a full cubic metre) it is certainly necessary to carry out a very formal fresh density check. In any case it is desirable to carry out such a check initially or very occasionally to verify or modify the assumption that it is adequately represented by the hardened specimen density. **In such a test it is very important not to omit the use of a glass top plate** since, however carefully it is done, striking off level is **never** accurate enough (usually the measured density is too high without a plate, but it can be too low).

When such arguments get to very fine tolerances, the question arises as to whether the concrete supplier must provide a full cubic metre of hardened concrete. Obviously the purchaser is entitled to fully compact the concrete as regards entrapped air, but is he entitled to vibrate out some of the entrained air? Also, if the concrete displays bleeding settlement, is it the volume before or after this which counts? These differences are quite small but in a situation where a great deal of concrete is placed with low labour and formwork costs (e.g. thick, unreinforced aerodrome paving) they can constitute a substantial proportion of the profit margin. There is no ‘correct’ answer to the foregoing questions, they are subject to negotiation, but it is as well to realize the situation if negotiating.

The correlation between strength and density arises because air and water are the two lightest ingredients of concrete and cement is (almost always) the heaviest ingredient. The only other factor likely to influence is the specific gravity of the coarse aggregate. In lightweight concrete the moisture content of the coarse aggregate may also be a significant factor.

11.10.5 Temperature

The cost of measuring the temperature of concrete at the time of casting test specimens is negligible, so it should always be done. There is often a good correlation between temperature and strength (higher temperature, lower strength) arising mainly from the increase in water requirement at higher temperatures. However, it is possible that early age strength will **increase** with increasing supply temperature, the additional maturity being sufficient to more than offset the increased water requirement. This is more likely to occur with say a 3-day test than a 7-day test and in cold climate countries rather than hot ones.

11.10.6 Moisture content

It would seem that, with the low cost and ready availability of microwave ovens, there should be an increasing use of measuring moisture content by drying a sample of wet concrete taken back to the laboratory. The author's experience is that the largest source of error would be in a non-representative ratio of mortar to coarse aggregate in the sample. This could be counteracted by sieving the concrete through a normal garden sieve and drying only the mortar fraction.

11.10.7 Wet analysis

The UK RAM (Rapid Analysis Machine) is an apparatus designed by the Cement and Concrete Association (UK) to split a sample of fresh concrete into its constituent parts. It is well known but apparently little used outside UK. The author's comments are made without the benefit of personal experience of using a RAM. Again we return to the twin questions of the accuracy of the result and a clear understanding of the purpose of the test. The principle of being able to analyse a sample of delivered fresh concrete is superficially extremely attractive. However Neville (1995) reports an investigation by BRMCA which found that the measured cement content may be inaccurate to the extent of ± more than 40 kg/m^3 and tended to underestimate the true value by more than 20 kg/m^3 on average.

As regards the relative proportions of the dry ingredients, the test may be more useful in some areas than others, and at some time in the past rather than today in other areas, i.e. it depends whether the supplier is likely to be trying to cheat. However, in projects served by a computer batching plant as described elsewhere in this section, the results would probably have more to do with mixing efficiency, sampling technique and test accuracy than with actual batch proportions. This is because hard copy computer records can be used to settle any question of deliberate deception.

As regards variability of the grading of input aggregates, a direct test of this together with a computer simulation of combined grading may be more accurate and economical.

To a considerable extent the answer to the usefulness of the test in routine control (there is no question of its usefulness for research and such uses as mixer efficiency tests) should be settled by graphing the results alongside compression tests and other data to examine the degree of correlation. The author has not had the opportunity to do this. It would seem that the best correlation would be anticipated from strength and w/c ratio. The author did have limited success 20 years ago in establishing the w/c ratio of fresh concrete by a method involving measurement, using a hydrometer, of the SG of water into which a standard volume of mortar extracted by wet sieving from a concrete had been thoroughly shaken.

There is also another patent, yet to be exploited (McLaughlin, private communication) in which water content *is* measured by the dilution of a concentrated solution mixed with the concrete.

11.10.8
Conclusion

It can be seen that the question of which tests are worth doing, and how frequently and thoroughly it is worth doing them, is greatly influenced by the circumstances. The circumstances include the extent of the remaining variability and its sources and also the assumptions made about the co-operativeness and trustworthiness of the concrete producer by the organization imposing the control (which may or may not be part of the producing organization).

12
Changing concepts

12.1
WORKABILITY

12.1.1
New rheometers

Since the first edition substantial work on new types of rheometer has been published. This includes the BTRHEOM (Fig. 12.1) of de Larrard *et al.* (1996, BHP96), the ICT (intensive compaction tester) of Invelop Oy, Finland and the rheometer of Wallevik (Wallevik and Gjorv, 1990).

There is no doubt that such equipment gives much more consistent and meaningful results than a slump test but the latter seems unlikely to be displaced as the usual on-site control technique. However, the rheometer of de Larrard has been designed to be small and robust enough to be used on site.

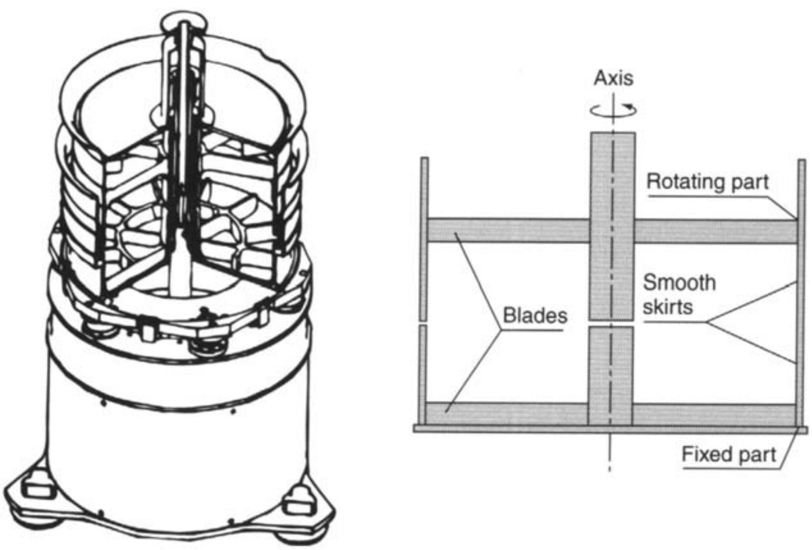


Fig. 12.1 The BTRHEOM rheometer.

Where such equipment is really valuable is in developing mix design techniques. This is a situation in which a real measurement of workability is required, rather than a control on the relative water content of successive trucks of production concrete.

12.1.2 Truck mounted workability control

What promises to be a most important new development is the mounting of workability control devices on agitating trucks. The basic concept is not new. Equipment has been available for many years to measure the energy output of the motor turning the agitator drum. This has a definite relationship to the workability of the concrete but the relationship has been too complex to provide useful guidance when delivering different sized loads of different concrete.

What is new is to apply the capabilities of computers to analysing and recording large amounts of data in a short time. Two developments of this type have recently entered limited production use in Australia. One of them, the Boral Panda system, concentrates on recording details of the delivery and especially of water content wherever it is added. It gives a slump indication using a hydraulic pressure transducer to measure engine power. It ensures an adequate remixing time is allowed after any water addition and automatically cuts off water supply when the maximum designed water content is reached. Data transfer by bar code reader takes place from batch plant, to truck, to site testing officer, and eventually to laboratory computer for incorporation in quality control analysis.

The other system, Compumix, was developed in Canada in the early 1990s. This system uses a complex neural network technique to calibrate typical trucks and from then on can determine slump fairly precisely. In particular it is able to calculate exactly what water addition is required to convert any given size of load of any type of concrete from one slump to another. It is claimed that one addition of this nature is much less detrimental to concrete quality than several increments providing the same eventual workability. The Compumix system actually takes charge of the mixing sequence. It automatically mixes at the optimum speed until it detects that the concrete is fully mixed, and then slows down to agitating speed. This is repeated after any water addition. It is claimed that over-rapid mixing is detrimental and can even cause segregation in concrete which has already been uniformly mixed. Compumix data are retained in the truck computer, if necessary for several days, but will usually be downloaded into the laboratory computer control system at the conclusion of each shift.

Both of the above systems (used by different clients) will be delivering data into the Conad control system in the near future. Both should result in improved control but an analysis of their relative effect is obviously some time away. The Compumix system is clearly based on a higher technology analysis of mixer performance but it remains to be seen whether this translates into better control of concrete quality.

The implementation of techniques of this sort is quite expensive, since the equipment is required on a per truck basis rather than per plant (as with computer batching) or per company (as with a control system such as Conad). As with most other things in the concrete industry it will need to prove its value in terms of cement economy or reduced rejection and/or dispute rate. Initial indications are that this may be quite possible.

The following account of the Compumix system has kindly been provided by Dan and Christine Assh.

THE COMPU-MIX TRUCK-MOUNTED MIXING PROCESS AND MANAGEMENT CONTROL SYSTEM.

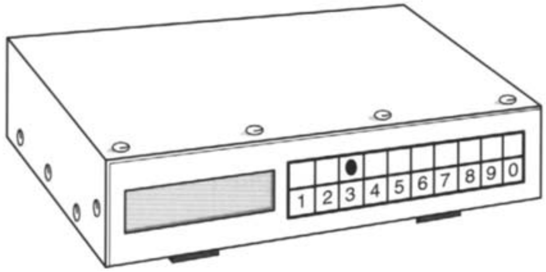


Fig. 12.2 Compu-Mix master cab control panel.

Introduction

In recent years, ready mixed concrete producers have tried to bring tighter control to the production of concrete. That is why the Compu-Mix control system was developed. There are three main sources of variability when delivering concrete: materials; batching of the materials; and mixing and agitation. Compu-Mix handles the third source, bringing control to a critical part of the process. Compu-Mix is a control system (Fig. 12.2) installed on each concrete mixer and its goal is to ensure the best treatment possible for the concrete loads in regard to mixing, agitation, and slump adjustment.

Independent studies carried out by various ready mixed concrete producers (in North America, South Africa and Australia) and by the SEM, a consulting firm founded by Michel Pigeon, PhD, a leading researcher in concrete technology, have shown that Compu-Mix brings:

- Better slump control.
- Enhanced workability for a given slump.
- More consistent entrained air.
- Important reduction in production variability, whether in usual day-to-day production or intensive jobs like pouring a bridge deck in 20 hours. The reduction of variability is approximately 33%, whether the plant is dry-batch, a wet-batch, or a premix.
- Delivery time savings in certain applications.

Compu-Mix can also provide a complete management system to follow delivery operations with:

- Load histories.
- Driver-independent truck tracking statuses.
- Tachograph information to monitor driving habits.

Description of Compu-Mix process control

The primary function of the control system is to control mixing and agitation to ensure that a specific sequence is performed, and that all trucks of a fleet can perform the same sequence. To do so, the system controls the speed and number of turns of the drum, independently of engine speed, during charging, mixing and agitation, which speeds and number of revolutions are all programmable to fit specific plant requirements. More specifically, the control system will:

- Control drum speed during charging.
- Perform a short high-speed mixing cycle at the plant that lasts approximately the wash time.

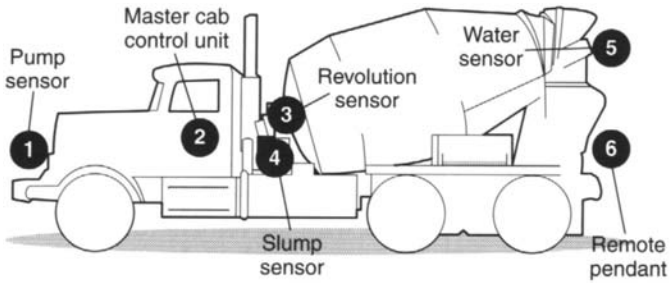


Fig. 12.3 Compu-Mix sensor locations.

- Perform a low speed mixing that allows the truck to safely continue mixing while en route to the site.
- When all mixing cycles are finished, automatically slow down the drum to an optimized agitation speed, designed to keep the concrete homogeneous and fresh longer.
- Measure slump and assist the operator in bringing the slump to the desired value.
- When water is added, automatically engage a specialized mixing cycle to ensure that this water is well distributed in the complete load and that water can react with the cement (this should minimize the detrimental effect on strength of the water addition).
- Inhibit discharge until a certain percentage of mixing is completed (programmable 0 to 100%).

Compu-Mix slump control

With advances in computerization, Compu-Mix can now precisely measure the slump from 0 to 200 mm (0 to 8 in) even when part of the load has been discharged. The measurement is accurate to ±10 mm (0.4 in) and the reading is provided directly in mm or tenths of an inch, as opposed to pressure readings in psi provided by other ‘slumpmeters’.

Compu-Mix also assists the driver in adjusting the slump. The purpose is to reach the desired value in a single attempt, saving time by avoiding the ‘add a little, mix a little’ guesswork and also avoiding the detrimental effect on strength brought by multiple water additions. The on-board ‘slump change expert system’ displays to the operator, on the remote control screen (Fig. 12.3), the predicted slump change (in mm) as water is being added. The driver simply opens the valve, keeps it open until he sees the desired change in slump (or target slump) indicated on screen, and then shuts it off. This operation could be automated using a solenoid valve but so far has never been required.

Finally, the mixing cycle following the water addition ensures that water will be distributed throughout the load and that the slump is uniform. This way, the slump will not have to be readjusted again because of insufficient mixing the first time (often mistaken for the load drying up while discharging).

Adjusting slump in truck *versus* in plant

Even with the best moisture probes, batching at the right slump every time with a precision of ±10 mm is extremely difficult. Too many things can vary. Obviously it is much easier to know what the slump is *after* the concrete is batched! The initial absorption is the highly unpredictable part. The slump is much easier to adjust after the initial hydration and absorption by the aggregates have occurred, and after having factored in any water left in the drum. Compu-Mix allows the concrete producer to take advantage of this.

The procedure suggested by the Compu-Mix developers is to target the batch 30 mm below the desired slump. After 2 min of mixing, Compu-Mix will provide an accurate slump reading and then the driver may adjust the slump using the Slump Change Expert System, and should hit the desired slump with a precision of ±10 mm. The remaining mixing will be performed while traveling to the job, which saves time.

The science behind Compu-Mix slump control accuracy

The slump reading is more than a simple pressure reading. The developers of Compu-Mix determined that to make an accurate assessment of slump, one should take at least 20 000 readings, at controlled drum speed, and take into account the volume of concrete left in drum and also the shape of the drum and blades. Compu-Mix was then programmed with complex models using artificial intelligence to be able to perform a global analysis of all these factors. The method developed also has the important advantage of requiring very little recalibration due to wear of the drum.

Slump adjustment was also refined far beyond the commonly used linear function. The water required to change the slump of the load by, say, 10 mm, depends of course on the volume of concrete remaining in the drum, but also on the initial slump; that is one of the reasons why these two variables are monitored constantly by the control system. The model structure in Compu-Mix can easily be calibrated to suit almost any concrete type.

Why does Compu-Mix reduce variability?

Aside from accurate slump control, executing the same mixing cycle on every truck of the fleet, Monday to Friday, and controlling agitation speed has proven to reduce production variability, because mixing has an effect on strength and on entrained air. Also, the imposed mixing following any water addition ensures that the water will be distributed throughout the load, again ensuring a more homogeneous concrete and lower variability. Finally, an adequate and extended mixing cycle reduces bleeding and segregation, and brings enhanced workability for a given slump (study by SEM). Combined with controlled agitation at low speed, it also reduces the loss of strength occurring in longer deliveries, so that concrete life is extended (Riadh Azouzi PhD, University Laval, Quebec, PQ, Canada) and variability induced by different delivery times is reduced.

Consistent slump is a part but not all of the equation. Even if a plant produced perfect slumps with a precision of ±5 mm, different sequences of mixing-agitation performed by different drivers naturally induce variability. Wet-batch and premix plants benefit as much from the control system as dry-batch plants because none of them can control what happens after the concrete has left the plant mixer.

Table 12.1 is a summary of a few studies comparing the variability of concrete produced with and without the control system for various concretes and conditions.

COMPU-MIX AS A MANAGEMENT SYSTEM

History logs for quality management and liability protection

Although the primary goal of the control system is to avoid problems, Compu-Mix provides a complete history of the load that allows users to retrace valuable information about the mixing cycle actually performed, the slump and water additions, volumes remaining, and times of the different steps of the delivery. This has helped to solve disputes between concrete producers and contractors.

The history indicates the slump as batched by the plant, the water added at the plant by the driver, the ensuing slump, the slump on arrival at site, the water added on site and the final slump. The records will also provide a time stamp of each action (water addition, discharge) and the volume of concrete left at that time.

To retrieve the histories, Compu-Mix is simply downloaded approximately once a month into a common IBM-compatible computer. These histories can then be used for ISO-9000 recording. A graphic program is supplied to rapidly visualize and analyze the information gathered.

Tachograph information and truck-tracking system

Compu-Mix is heading in the direction of a complete control system, designed to perform all tasks of monitoring and control that could be needed on a ready mix truck. Tachograph and truck

Table 12.1

Concrete and Plant Type, Testing Conditions	*Standard trucks* σ *(no. samples)*	*Compu-Mix trucks* σ *(no. samples)*	*Reduction* of σ *(%)*
25 MPa with entrained air, 8 m^3 premix plant, Canada, 1992. Testing over 60 days.	2,87	2,39	
(118)	(71)	17	
30 MPa with entrained air, 8 m^3 premix plant, Canada, 1992. Testing over 60 days.	2,59	1,66	
(80)	(68)	36	
32 MPa with entrained air, 8 m^3 premix plant, Canada, 1992. Testing over 60 days.	4,77	3,61	
(107)	(151)	24	
25 MPa, dry-batch plant, S.Africa, 1995. Testing over 15 days.	2,3	1,5	
(112)	(124)	35	
25 MPa, wet-batch plant, S.Africa, 1995. Testing over 15 days.	2,79	1,87	
(144)	(128)	33	
32 MPa with entrained air, dry-batch plant, Canada, 1994, 20-hour bridge deck pour.	2,68	1,73	
(76)	(70)	35	

Concrete and Plant Type, Testing Conditions	*Standard trucks σ (no. samples)*	*Compu-Mix trucks σ (no. samples)*	*Reduction of σ (%)*
32 MPa with entrained air, dry-batch plant, Canada, 1995, 16-hour bridge deck pour.	2,63	1,40	
(38)	(34)	47	

tracking information have been added to the process control system. A major advantage of the Compu-Mix truck-tracking is that most of the statuses are automatic, and not driver entered, which reduces the chance of error and fraud. This can be done because of the intelligent monitoring performed by the control system.

The tachograph records truck mileage, speed, acceleration and engine rpm (Fig. 12.4). This information combined with that of the Compu-Mix process control allows one to monitor driving practices, generate automatic statuses, detect some cases of time loss or fraud, like unauthorized stops, long washout times and stolen concrete.

Conclusion

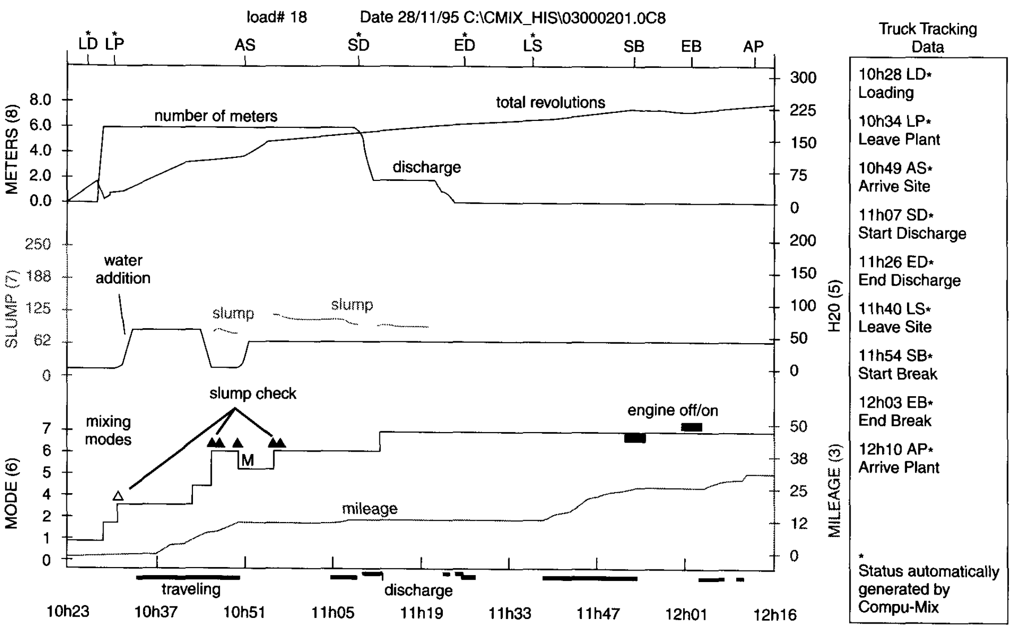


Fig. 12.4 Compu-Mix history example including tachograph and truck tracking data.

The Compu-Mix system brings control to an important part of the process of making ready-mixed concrete. It includes slump measurements using artificial intelligence and a slump change expert system to adjust slump. The studies performed have shown reductions of variability around 33%. Such reductions in compressive strength variability can bring significant cement savings when the amount of cement in a mix must be adjusted depending on the production variability. The higher the safety margin required, the higher the potential for savings. In North America, where the safety margin required is rather low, the payback for the producer was evaluated to be around one year, assuming the trucks delivered at least 4200 m^3 a year.

Further reading

Study of the effect of controlling mixing-agitation on ready mixed concrete variability, conducted at the Milton plant, by Canada Building Materials (CBM), DSS Automotion Systems, and Riadh Azouzi, PEng., PhD, January 1996.

Study of the effect of Compu-Mix on compressive strength variability and delivery time duration conducted at the McCowan plant, by the Dufferin-Custom Concrete Group, DSS Automotion Systems and Michel Guillot, PEng., PhD (MIT), October 1994.

Study of the effect of Compu-Mix on compressive strength variability conducted at the Woodbine plant, by Dufferin-Custom Concrete Group, DSS Automotion Systems, and Michel Guillot, PEng., PhD (MIT), October 1994.

Study of the Influence of the Compu-Mix system on some properties of ready-mix concretes, by the Service d'Expertise en Matériaux Inc., January 1993.

12.1.3
Slump

The mix design and quality control chapters have used slump as a measure of relative workability. It is important to realize that this is a matter of convenience and that the slump test is a very poor measure of the relative workability of **different** mixes. One reason for retaining slump as a criterion is that it is so deeply ingrained in the theory and practice of concrete technology. Another is that slump in combination with the author's MSF does have a little more validity as an absolute criterion than slump alone. A third, probably the most important, is that it is a sensitive detector of a change in water content between successive deliveries of the **same** concrete mix.

What is important is not to stop using the slump test but to realize and allow for its limitations. For example a limiting slump value is often included in a job specification. With few exceptions, this is not the best way to achieve the specifier's objective. First of all there should be an objective for the specification of anything, rather than it having been included in a previous specification and so mindlessly continued in the current document. The objectives may be to avoid high shrinkage, segregation and bleeding or to avoid an excessive w/c ratio leading to inadequate strength or durability. However, any of these faults can be encountered at almost any slump, however low, and avoided at any slump, however high. It is also easy to detect from a theoretical mix submission which mixes will be subject to one or other of these problems. The contractor should therefore be permitted to submit his mix for approval at whatever slump he chooses providing it is designed to accommodate his own slump limit without detriment. It is quite possible to produce fully flowing (250 mm slump or more) concrete having none of the potential faults noted and to produce almost all these faults in a 50 mm slump mix.

The rejection of a truckload of concrete on the basis of slump should also be approached in a reasonable manner. The slump test is both quite sensitive to small changes of water content and very easy to perform inaccurately. Certainly the truck driver should always be allowed to insist on the test being repeated. An extra 10 mm of slump probably involves about an extra 3 litres of water per cubic metre of concrete and may depress strength by about 1 MPa. The person charged with concrete acceptance should be kept continuously aware of the current strength margin of the mix in question and therefore of whether or not it is essential to reject slightly overslump concrete on strength grounds (and similarly for any shrinkage limit which may have been specified). It is more usual to find that a need to reject first arises on the grounds of wet properties or surface appearance. Slump variation will cause colour variation on a fairfaced wall and slump in excess of that designed for can involve segregation, bleeding, delayed finishing and/or floors of poor wear resistance.

While continuous perfection is impractical, a slump test will only be asymmetrical if it has been produced by an asymmetrical process. It is often possible to know where the slump operator has stood, how he has used his scoop and how he has held his rod, all by looking at the resulting slumped concrete after the test. A failure to rotate the scoop will usually cause a higher coarse aggregate content opposite the point of discharge from the scoop. This will often cause the cone to lean towards the point of discharge on stripping. It is not easy to rod the foot of the cone opposite the operator if the rod is held in a 'dagger grip'. To accomplish this the operator must project his elbow over the slump cone in order to rod parallel to the side of the cone around the entire circumference. An alternative is to use a 'rope grip', i.e. to hold the rod as though pulling a rope.

The slump test is based on a standardized degree of semi-compaction, unlike compression test specimens which should be fully compacted whatever it takes. Therefore it is important that the correct number of strokes be used in the slump test whilst being only a required minimum in compacting compression specimens. It is also important that the rod have the correct end shape. A flat ended rod (e.g. a piece of reinforcing bar) pushes coarse aggregate to the bottom and tends to leave a hole rather than compact. The British rod has a hemispherical end, which is a distinct improvement over a flat end. However, the Australian and American rods, which taper to half the original diameter before having a hemispherical end give greater compaction. It should also be realized that slump measurement is different in the UK, US and Australia. In the UK, measurement is to the highest point, in the US to the point on the centreline of the original cone and in Australia to the average of the original top surface. One may have personal preferences but the important thing is to be consistent on a particular project and to be on the lookout for new operators who may have been trained by site engineers of different nationality.

A concept currently being evaluated by the author is that of an 'equivalent slump' (Day, 1996). As Bryant Mather has so firmly pointed out (Mather, 1987) slump loss is proportional to temperature and leads to the (strictly incorrect but workable) view that water requirement increases with temperature. Everyone realizes that slump reduces with time. Putting the two effects together, it is clear that slump only has a real meaning if accompanied by a time and temperature reading. The author's current proposal is to combine the time and temperature into an equivalent age according to Arrhenius (see section 12.2 on early age strength for more detail). Thus an 'equivalent slump' could be evaluated, being the slump which would have been obtained had the concrete been kept for 30 min at a temperature of 20°C. It can be imagined that if compression specimens were stored at anywhere from 10 to 30°C and tested at anywhere between 10 and 40 days, poor correlation would be obtained with w/c ratio. This is what we are currently doing with slump tests (i.e. ignoring time and temperature effects).

It would be quite easy to arrange for a slump value to be converted into its equivalent value as it is entered into a computer, although less easy to arrange for this to be available during a field acceptance test. What becomes quite clear when these matters are considered is the absurdity of some rejection decisions currently taken in the field. A slump of, say, 150 mm taken 15 min after batching on a cold morning may indicate a lower water content, and therefore a stronger concrete, than a slump of 50 mm taken an hour after batching on a hot afternoon. Rules of thumb could be developed to allow approximately for this effect with at least more equity and realism than assuming that a slump is a slump and that's it.

With the above points considered, adequate attention given to correct sampling and remixing of the sample; correct bedding, cleaning and moistening of a rigid metal baseplate; and use of a square mouth scoop (because a round mouth scoop leaves mortar behind in the sampling tray) the slump test can give more reliable guidance than is often the case. Nevertheless one does encounter the occasional cheeky operator who asks what you would like the slump to be before carrying out the test. Suitably instructed, such persons are at least usually competent, since they obviously know what causes incorrect results.

12.1.4
Compacting factor and Vebe tests

These tests are two widely used attempts to extend workability testing into the range of lower slump and no slump mixes. 'Widely used' however refers to numbers of countries rather than numbers of building sites. The compacting factor is used in the field to some extent in the UK and the Vebe, which originated in Sweden, is used in some precasting factories. Both are widely available in concrete research laboratories.

The compacting factor uses a double hopper arrangement to drop concrete into a standard cylinder mould. The first (highest) hopper merely charges the second hopper in a standard way. The drop from the second hopper into the cylinder mould is intended to exert a standard compactive effort. The degree of compaction achieved is measured by comparing the weight of concrete in the cylinder, when struck off level, to that required to fill the cylinder with concrete vibrated to full compaction.

The Vebe test measures the time taken by a standardized vibrating table to remould a standard slump cone into a cylindrical form.

12.1.5
Flow test

The German DIN 1048 flow table is the most widely used method of measuring the workability of concrete which is too fluid for the slump test. The equipment consists of a metal sheeted, hinged, wooden flap 700 mm square, one edge of which can be raised 40 mm and then dropped.

Concrete is placed at the centre of this table in what is essentially the bottom two-thirds of a standard slump cone. The flap is lifted and dropped 15 times, after which the spread is measured as the mean of the two diameters parallel to the sides of the table.

This table is quite different from the ASTM flow table which, whilst it may give as good or better results in the laboratory, is not suitable for site use.

12.1.6
Two-point test

G.H.Tattersall of Sheffield University has for many years been the leading theorist on workability. His books (Tattersall, 1991, 1976) should be consulted not only for his own views but also for the wealth of references and information on the general field of workability.

Tattersall's contention is that concrete is a **Bingham material.** This means essentially that the material has an initial resistance to deformation, which has to be overcome by an applied force before subsequent deformation, which is then proportional to the excess of the applied force over the initial resistance. This is

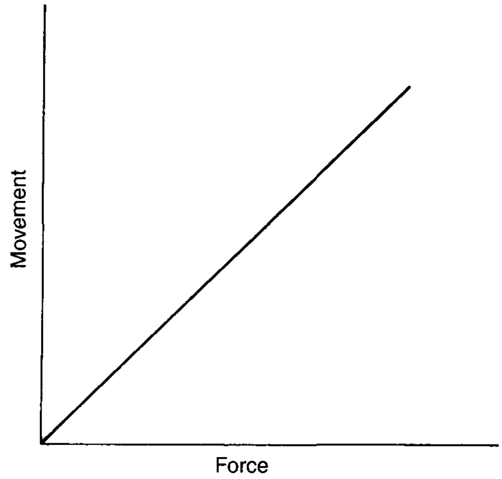


Fig. 12.5 Newtonian fluid.

in contrast to many other materials which are **Newtonian fluids** (Fig. 12.5). A Newtonian fluid does not show any resistance to initial deformation and its rate of deformation is at all times proportional to the applied force.

$$M = kF \tag{12.1}$$

$$M = g + hF \tag{12.2}$$

The principal significance of this is that whilst a single point is sufficient to characterize a Newtonian fluid, at least two points are needed to characterize a Bingham material. The different points may be generated by a test with different speeds of movement for which resulting force is measured or different force or energy levels for which resulting movement is measured.

The possibility clearly exists that different concretes will have different g and h values (Figs. 12.6 and 12.7 and equation (12.2)) so that their relative workability would be ranked differently according to the energy level used in the test or practical application.

If the test used is at a different energy level than the actual practical application, it can give the wrong conclusion about which concrete is preferable.

Tattersall describes the parameters g and h as the yield value and the plastic viscosity of the concrete.

The view being presented by Tattersall is that all existing tests (and there are many times the number discussed above) are 'single-point tests' capable of giving the wrong answer in this way. His solution is to use a workability test consisting of a container of concrete stirred by a rotating paddle at a selectable range of velocities, at each of which the energy input is measured.

This view has been presented since the 1970s (Tattersall, 1976) and before, without being seriously challenged. What is contentious is not the correctness of the theory but the practicality of its application. Even if the test itself is rejected for production use, the findings and implications generated by its research use should be understood and applied by all concrete technologists. At a primary level this can mean only having an understanding of the inadequacy of current tests and avoiding precipitate action based on them. At a higher level it can mean understanding the relative effects of a range of factors such as water content, admixture dosage, grading changes and supplementary materials such as fly ash and silica fume. At the most advanced level, Tattersall proposes use of the test in production not merely to measure workability, but also

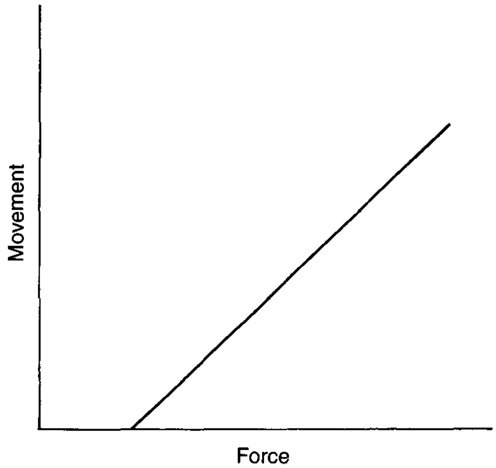


Fig. 12.6 Bingham material.

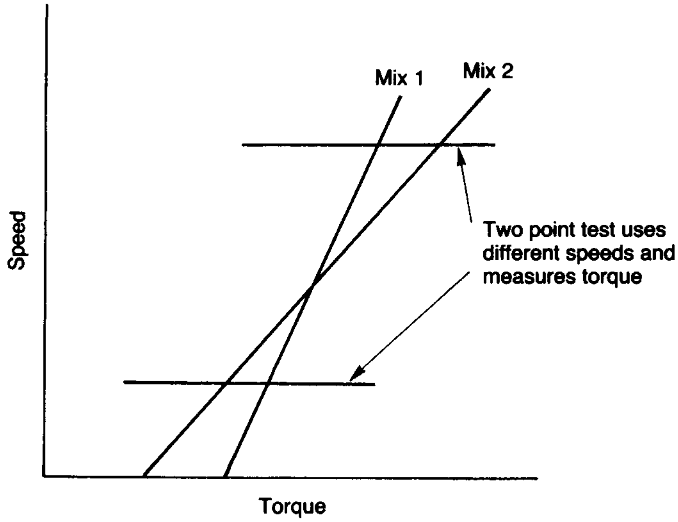


Fig. 12.7 Intersecting workability curves.

to detect the causes of variation purely from changes in the slope and location of the output velocity/torque relationship.

Reference to Tattersall's book (1991) is recommended, but some of his clearer and more important findings on the relative effects of a range of factors are given below.

Slump as a criterion

Slump is effectively the lowest energy input level (only gravity) and therefore corresponds to the F axis of the graphs being presented. So the steeper the slope of the line, i.e. the lower the value of h, the more representative the slump will be of the workability at higher energy levels (e.g. vibration on site). It is

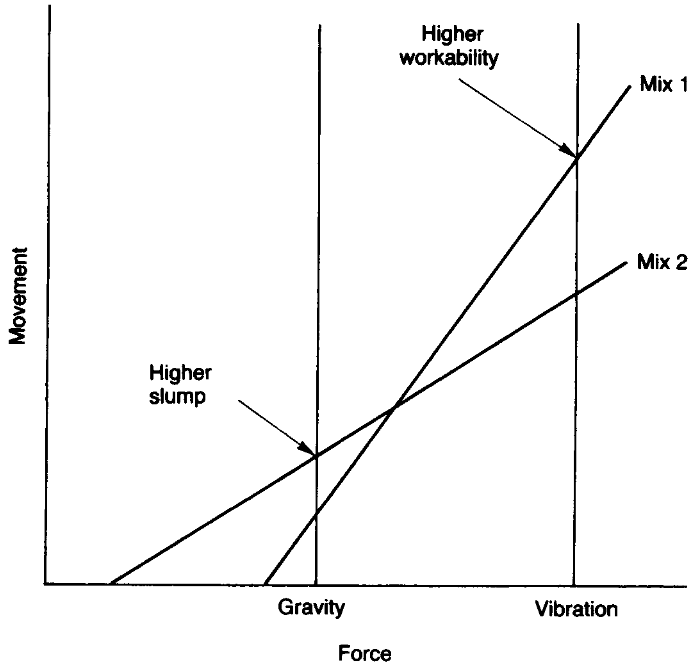


Fig. 12.8 Incorrect assessment of workability by slump.

concrete which displays a very low slope (i.e. a high *h* value), which gives much less workability under vibration than is predicted by the slump testing (Fig. 12.8).

Water content

Water content is apparently the only major variable which affects both *g* and *h* and so produces a fan shaped array of *g—h* curves.

Water reducing admixture content

The use of a water reducer has been found to produce a parallel movement of the *g—h* curve. So at a given slump, a replacement of water by admixture will produce a rotation of the *g—h* curve indicating a reduced workability under vibration.

Fines content

An increase in fines content will generally produce an increase in *g* (i.e. a reduced slump) and a reduction in *h* (i.e. an improved perfor mance under vibration relative to slump). However, these effects, especially that on *h,* depend on the characteristics of the particular mix. This is not surprising because it is well known that there is an optimum sand content below and above which performance will be less satisfactory. What is shown by the work of Tattersall, and others using his equipment, is that a different optimum may be found at a different energy input level.

Particle shape

Tattersall has disappointingly little to offer on particle shape, but what there is, is interesting. This is that particle shape has much more effect on *h* than on *g* and therefore more effect on workability in practice than on the slump test. So an increased angularity of either coarse or fine aggregate will produce a lower workability which will not be fully reflected by the slump test.

Fly ash

Results quoted by Tattersall show a variable effect depending on the initial cement content of the mix. A larger water reduction is obtained at lower cement contents by a given percentage replacement by weight of cement by fly ash. The effect on *g* is greater than that on *h* and therefore the improved workability will be fully reflected in the slump test. However, if water is removed to give the same slump, the *h* value would increase and the concrete would be less workable under vibration than anticipated from the slump. The effect is similar to that obtained by the use of water reducing admixtures.

Silica fume

Silica fume is found to have a very large effect on cohesion. It therefore substantially reduces slump but it can reduce *h* by up to 50% in some circumstances, i.e. the mix can have a much reduced slump and yet be easier to compact under vibration when silica fume is introduced. The situation is complicated by the usual (and very desirable) practice of combining the use of silica fume with that of a superplasticizing admixture. It is easy to underestimate the effective workability of silica fume concrete from a static test such as a slump test.

Vibration

The incorporation of vibration in Tattersall's equipment (not normal) gives the interesting result of effectively reducing *g* to zero. Since the slump test effectively only measures *g,* it can be seen that slump may not be very relevant to the performance of concrete under vibration.

12.1.7
Significance and use of Tattersall's concepts

Tattersall's proposals to use his two-point test as a means of either specification or quality control are ingenious but do not appear likely to be taken up industrially. In the case of specification, a process of calibration by site trial mixes is recommended. It is likely to be difficult to convince contractors that this is worthwhile.

In the case of quality control, the information which can be gleaned from two point testing is certainly interesting. However, the problem in this case is that the information can, using new techniques, be obtained from other sources. As readers of Chapter 6 will already know, variations in batch quantities of water, admixtures and all dry materials can be retrieved and analysed directly from suitable equipment. Slump is a useful indicator of water content variations and the greater accuracy of the two-point test would be welcome. However, such accuracy is not really essential providing it has been understood that trends and change points rather than individual values are of importance.

Substantial changes in particle shape of either coarse or fine aggregates are hopefully both infrequent and directly observable. Where such variations are frequent and significant, a direct test program may be worthwhile. The sand flow test (section 3.14) may be a better control tool for this purpose.

The two kinds of use which **do** appear likely to prove valuable in practice are the settlement of disputes and the design of concrete mixes. For the former, it would be useful to have two-point testers located in independent commercial testing laboratories. This would enable testing of any mix whose slump was claimed to be unrepresentative of its site performance. For the latter, one can imagine every substantial ready mix supplier finding it worthwhile to have one in each laboratory. This would enable every new ingredient or new mix design to be checked for undesirable tendencies prior to wide commercial release.

12.1.8 MSF and the two-point test

No work has been done on this combination but there would appear to be some possibility of a correlation between MSF and behaviour in a two-point test. Since MSF is intended as a measure of cohesiveness, there should be little doubt that an increase in MSF would cause an increasing value of g. The interesting question is whether there is any correlation between h and MSF at constant slump. If this is so, the author's con tention that the wet properties of a mix can be substantially described by its slump and MSF would be strongly supported.

It may be that there are other such relationships waiting to be explored, especially that between sand flow and the two-point test, and that a substantial part of the value of the two-point test lies in its likely ability to investigate such matters.

12.2 MATURITY/EQUIVALENT AGE

Concrete gains strength with age. It also gains strength more rapidly the higher the temperature. It is desirable to establish a relationship between strength, time and temperature so that the strength of a particular concrete after any particular time and temperature cycle can be established from a knowledge of its strength after any other time and temperature cycle. There have been two attempts to achieve this and both are detailed in ASTM C 1074. Although the two terms 'maturity' and 'equivalent age' are sometimes used in a qualitative way as interchangeable, they each have a precise meaning in numerical terms.

Maturity is the age of a particular concrete expressed as degree-hours, i.e. as the area under a temperature-time graph.

Equivalent age is the age at which a particular concrete would have developed its current strength if maintained at a nominated standard temperature.

Both of these definitions are incomplete in that the base temperature in the case of maturity and the standard temperature in the case of equivalent age remain to be nominated.

The **maturity** concept was developed in the UK in the 1950s and is generally attributed to Saul (1951) or Nurse (1949). The base temperature should theoretically be that temperature at which concrete does not gain strength. This is often taken to be either +10°F or −10°C, or as +11°F or −11°C (which are almost the same temperature). It is also often taken as 0°C for convenience, although concrete does gain strength at 0°C.

The **equivalent age** concept is older and more accurate, but also more complicated. The concept was not originated specifically for concrete but as a general concept for all chemical reactions. The general formula is attributed to Arrhenius. The concept was applied to concrete in the 1930s in USSR in the form of coefficients by which the length of time at each temperature should be multiplied to give equivalence.

The relationship is exponential and is given by the formula:

$$EA = \sum\left(t\,e^{-Q\left(\frac{1}{T_a}+\frac{1}{T_s}\right)}\right)$$

where EA=equivalent age (hours)

Q=activation energy divided by the gas constant

T_a=temperature (°K) for time interval t

t=time (hours) spent at temperature Ta

T_s=the reference temperature (°K=°C+273).

The reference temperature (T_s) is the standard curing temperature at which test specimens are kept. In many parts of the world it is 20°C (293°K). The Q value can range from below 4000 to over 5000 depending on the characteristics of the particular cement. It is often taken as 4200.

Extensive use of the system, since the first edition, has proved very advantageous in a number of early strength requirement situations and, more surprisingly, in one application where it was important to avoid a high early strength.

It has become apparent that it is not only a matter of inaccuracy which dictates that the theory should not be directly applied to the prediction of 28-day results. Graphing large numbers of results on a semi-logarithmic scale has shown that the results do form a straight line but that there is invariably a distinct change in slope at some age. Initially, as described below, the system was used to predict 7-day results and the average 7 to 28-day gain was then added to give the predicted 28-day result. In effect the assumption was that there may or may not be a change of slope at or after 7 days, and even that the line may or may not be straight after 7 days, but these were immaterial so long as the line was straight prior to 7 days. The technique of adding the average 7 to 28-day gain to the 7-day result to obtain a predicted 28-day result has been used—and well justified—by the author for many years (section 5.16).

What we have found is that the whole strength *versus* logarithm of equivalent age graph, from as soon as the concrete has any strength to 120 days (so far) tends to be composed of two straight lines. The break in the line can be in either direction (i.e. it may be steeper before or after the break) and it has so far been detected at ages between 2 and 7 days (Figs 12.9 and 12.10). The computer program has therefore been amended to enable the user to select any desired age as the control age. It is recommended that the initial action should be to make a substantial number of specimens from a single trial mix and test them at a range of (temperature monitored) ages. When plotted on the strength *versus* log equivalent age graph the change point should be easily seen and that age adopted as the control age.

Applications to date have included:

- Very large heat cured precast units where a strength of 17MPa at 7 hours was required. Here it was found that heated mixing water was far more effective than heating the precast unit after casting.
- Concrete for secant piles, where it was desired to drill intermediate piles taking pieces out of the two adjoining piles so as to form a continuous wall. The requirement here was an eventual strength of 70 MPa from the durability viewpoint but the strength was not to exceed 15 MPa at an age of 7 days. This required the use of a large proportion of blast furnace slag and fly ash in addition to the use of a retarding admixture.
- Concrete for a 40 000 m^2 industrial floor slab to be prestressed at less than 24 hours, as soon as a strength of 18 MPa was attained. This project did not in fact use the system described but it did enable assessment of its potential. During production, cylinders were tested at intervals until the required

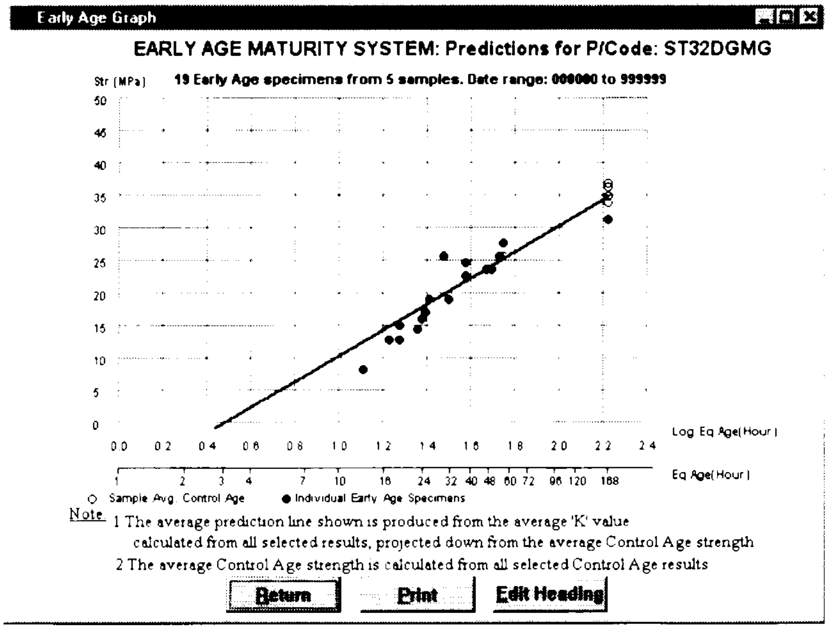


Fig. 12.9 Early age specimens graph.

strength was obtained. Temperatures were not monitored but results were obtained at a variety of ages from 6 to 24 hours after casting

12.2.1 Prediction of standard 28-day and current *in situ* strengths

The use of this concept enables more powerful solutions to two problems:

1. Prediction of 28-day strength from an early age test.
2. Establishment of strength of *in situ* concrete.

Previously problem (1) was approached by setting down a fixed accelerating (heated curing) regime and experimentally determining a correlation curve.

Problem (2) only became soluble without *in situ* strength testing by use of either the maturity or the equivalent age concepts combined with *in situ* temperature logging. An initial approach was to construct a strength-maturity or strength-equivalent age curve experimentally in the laboratory. Having logged *in situ* temperatures, either the maturity or equivalent age could be determined at any time and the corresponding strength read from the graph. The weak aspect of this technique was the basic assumption that the *in situ* concrete was of identical strength to that previously used to construct the curve.

The strength-equivalent age relationship plots as a straight line on log-normal graph paper. It therefore only requires two points to define the relationship. If it is assumed that for any particular mix the slope of the line will not change, then only a single point is needed to define the relationship. The author has devised a technique by which a computer program automatically continuously updates the average value of the slope

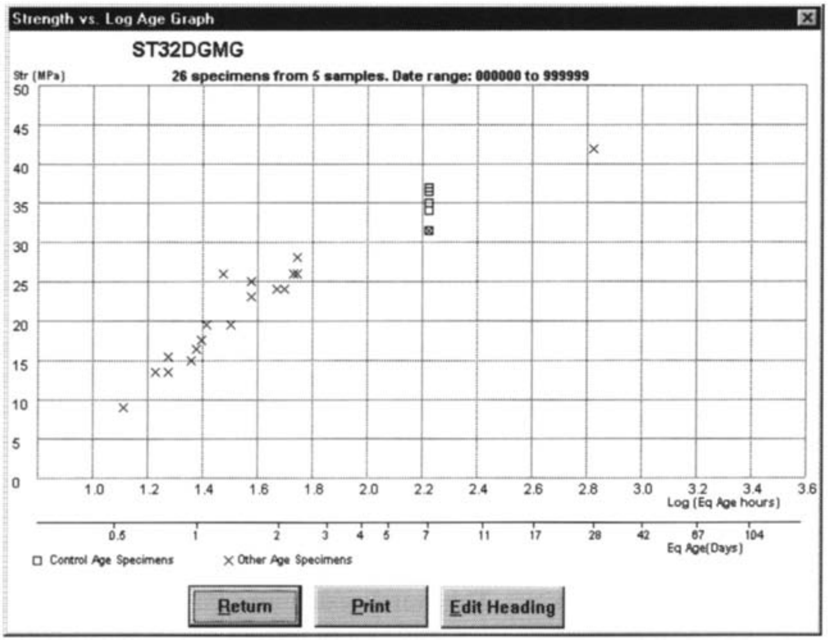


Fig. 12.10 Scatter plot of all specimens, showing change of slope at two days.

for any particular mix from a series of early age and 7-day strength and equivalent age results. The early age may be anywhere in the range of, say, 15 to 75 equivalent hours. This number of hours may be accumulated at normal ambient temperatures or through any *ad hoc* range of higher temperatures to give an earlier answer. At the price of continuously mon itoring specimen temperature, the need to adhere to a fixed routine of either time or temperature is avoided.

Having provided a prediction of 28-day strength within 24 hours (if desired), the computer program uses this prediction to calculate the equivalent age at which any nominated strength will be attained. A meter such as the **James Maturity Meter** can be used to continuously read the equivalent age of the *in situ* concrete and so provide a precise assessment of readiness for stripping or stressing. The author has also programmed another instrument, the **Data Electronics Datataker 50,** to directly read strength for a given mix design. The prediction constant is updated for each use by means of a magnetic card and an adjustment for the particular batch of concrete is telephoned to site after testing a cylinder at 15 to 20 hours.

12.2.2
Limitations of the equivalent age concept

Concrete which has been heated:

1. Too early, or
2. Too rapidly, or
3. To too high a temperature.

will attain a lower 28-day strength than the same concrete cured at normal temperatures. The limiting values to avoid such problems differ for different cements and especially for different combinations of pozzolan and cement. It does not follow that routines which involve a loss of 28-day strength should not be used, only that the loss should be understood and allowance made for it if necessary.

It can be anticipated that concrete containing a pozzolan or ggbfs will withstand higher curing temperatures without loss of potential 28-day strength. Such concretes may show an **increased** 28-day strength through higher temperature curing.

Any particular curing regime for any particular concrete can be readily checked by comparing the strength *versus* logarithm of equivalent age curves for heated and normally cured test specimens. As a rough guide, a delay of 2 equivalent hours at 20°C, a rate of rise of 0.5°C per minute and a maximum temperature of about 70°C will usually avoid any significant loss of 28-day strength when using normal Portland cement.

The equivalent age concept is less accurate at predicting 28-day strength than at predicting 7-day strength (Guo Chengju, 1989). The author prefers to use it to predict 7-day strength and then to add the average 7 to 28-day strength gain previously established for the mix in question.

Carino (1984) has recently concluded that a parabolic relationship may be simpler to use and equally, or even more, accurate than the Arrhenius relationship. The author has not experimented with such a relationship since it is easier to continue using the Arrhenius relationship now that it has been incorporated in a user friendly computer program.

12.2.3 Low temperature application

It is necessary to protect concrete from freezing and thawing damage, and also from dehydration, until it has attained a critical strength beyond which further protection is not essential. This has been recognized for many years and various national codes have laid down specified periods of protection. In some cases the protection period is varied according to ambient temperature but much greater precision and flexibility is now feasible by defining the protection period in terms of measured equivalent age or of *in situ* strength determined from equivalent age.

12.2.4 Temperature cycles and stresses

The author is unaware of any definitive work on the subject (other than relating to freezing and thawing) but the subject of temperature variation should be considered. It is well known that the number of cycles of freezing and thawing rather than the lowest temperature reached is the significant parameter in frost damage. This means that more such damage may be experienced in a marginal climate where concrete may freeze and thaw 50 or 100 times per annum than in a much colder climate where concrete may remain solidly frozen for several months.

Hearsay evidence suggests that a similar situation may occur at high temperatures, although to a different extent. Thus a concrete specimen which is cast hot and stays hot until it attains substantial strength, or is heated and stays heated, may be less damaged than one which is cast hot and taken into an air-conditioned laboratory.

The possibility is that changing temperature may cause bond stresses at the paste/aggregate interface and/or microcracking in the paste or mortar fractions. It seems likely that such events would have greater significance for tensile and flexural strength, and for durability, than for compressive strength. A thermally caused reduction in compressive strength may be the tip of an iceberg in terms of total resulting damage.

12.2.5
Time is money

The increasing sophistication of the building and construction industries causes the cost of any delay, planned or unplanned, to be much greater than formerly. At the same time techniques are being originated to cut the cost and inconvenience of equivalent age monitoring. In combination these factors should result in a substantial increase in the future of equivalent age monitoring.

Two quite different applications are distinguished:

1. The determination of *in situ* strength.
2. The use of the technique for QC purposes, i.e. so that a concrete producer can make adjustments to his mixes at 24 to 48 hours rather than 7 days.

12.3
CASH PENALTY SPECIFICATION

This section was first published by the author (Day, 1982b) as an article in *Concrete International: Design and Construction,* September 1982 under the title: Cash penalty specifications can be fair and effective. Permission granted by the American Concrete Institute to reproduce it here is gratefully acknowledged.

A cash penalty of twice the cost of the extra cement which would have been required to avoid defectiveness is proposed. It is shown in detail that if this is based on the statistical analysis of any 30 consecutive 28-day test results, very little inequity would result to either party (in contrast to the substantial risk of inequity under current specifications based on inaccurate, small sample criteria). The aspect of legal enforceability is considered and examples are provided of a suitable cash penalty provision used in a major Australian structure, and of several situations where cash penalty provisions would have been desirable.

A good specification system accomplishes the following (Day, 1961):

1. Ensures the detection and penalization of unsatisfactory concrete.
2. Avoids the penalization of good concrete.
3. Encourages good quality control.
4. Avoids any doubt of fairness and eliminates disputes.
5. Is based on sound theoretical principles.

Typical concrete specifications around the world continue to levy one penalty of rejection and continue to base judgement on criteria which are known to be inefficient at distinguishing the actual quality of the concrete assessed (Chung, 1978). The result of this ostrich-like attitude is to leave supervising engineers in untenable positions, to subject concrete suppliers to gross unfairness on occasions, frequently to allow unsatisfactory concrete to be supplied with impunity, and worst of all, to fail to encourage responsible producers of low-variability concrete.

12.3.1 The proposed system

The quality of concrete is assumed to be represented by the mean and standard deviation of strength. Quality should be specified by the requirement:

$$\bar{x} \geq f'_c + k\sigma$$

where $\bar{x}$=mean concrete strength
f'_c=specified strength
σ=standard deviation of strength
k=constant.

Any deficiency in strength can be readily assessed in terms of inadequate mean strength. The cost of remedying that deficiency can be readily assessed in terms of cement content.

For a limited extent of deficiency, a penalty of twice the cost of remedying the deficiency could be imposed. This penalty is negligible for small deficiencies, but if the criterion is sufficiently accurate, the penalty will be sufficient to ensure that no concrete supplier can make additional profit by supplying understrength concrete. This penalty system benefits producers of low-variability concrete and encourages improved quality control.

The key to this system is the determination of the values of mean strength and standard deviation with sufficient accuracy, and the selection of a suitable value for *k*. It is immaterial whether the cement-content change required to provide a given strength change is truly a constant for all concrete, providing the change is never more than twice the assumed value.

12.3.2 Accuracy of assessment

The gross inaccuracy of assessment encountered under most specifications arises from an inadequate number of test results (Chung, 1978), and from attempting to assess the quality of an amount of concrete sufficiently small to accept or reject as a whole. There is no such requirement in a cash-penalty specification.

A secondary reason for basing a criterion on a small number of results is to enable a judgement to be made quickly, thus limiting the amount of defective concrete supplied before a halt is called. This pious intention becomes a joke when the results are obtained at 28 days.

The solution to this dilemma is to separate the functions of (1) acceptance/penalization and (2) detection and arrest of adverse quality.

An interesting and valuable result of operating under a cash-penalty scheme is that the interests of the supervisor and the concrete supplier coincide in their joint desire to detect and eliminate adverse trends at the earliest possible moment. This cooperative type of relationship is in contrast to the traditional requirement to establish with legal precision that concrete strength is inadequate and then require the unwilling supplier to rectify the matter.

The suppliers generally recognize that rapid reaction to warnings of low strength from the quality control engineer can save the supplier money. A graphing system can provide such information based on a few early age test results and will enable the supplier not only to avoid extensive periods of low strength but also to reduce the overall variability (a double saving in potential penalties) (Day, 1981).

The standard error of assessment of the mean strength of a group of n test results is $\sigma/\sqrt{n}$, while that of its standard deviation is $\sigma/\sqrt{(2n)}$.

The standard error of assessment of the criterion $\bar{x}-k\sigma$ is therefore:

$$\sqrt{\left[\frac{\sigma^2}{n}+\frac{(k\sigma)^2}{2n}\right]}$$

where k=1.28 (a 10% defectives criterion)
σ=3 MPa (435 psi)
n=30 results.

The expression gives a standard error of approximately 0.74 MPa (107 psi). This means that 90% of assessments will be within ±1.65×0.74 =1.22 MPa (177 psi) of the correct value.

If it is further assumed that a 1 MPa (145 psi) strength change requires 7 to 8 kg/m^3 (12 to 14 lb/yd^3) of cement change (the actual value could range from 5 to 10 kg/m^3 (8 to 17 lb/yd^3) for different concretes), then the inaccuracy amounts to a maximum of ±10 kg/m^3 (±17 lb/yd^3) in cement content, or a cost of around \$0.70 (Australian)/m^3 (approximately \$0.56 (US)/yd^3).

12.3.3 Operation of the system

The specification might then read as follows.

'The specified strength of the concrete shall be XMPa and for every 1 MPa (145 psi) that the mean strength of any 30 consecutive samples minus 1.28 times the standard deviation of strength of those samples falls below XMPa, the contractor shall pay a penalty of \$1 (Australian)/m^3 (\$0.80 (US)/yd^3) of the whole of the concrete represented by the 30 results in question.'

(\$1 equals twice the cost of the 7.5 kg (16.5 lb) of cement assumed to be required to increase the concrete strength by 1 MPa (145 psi)).

To avoid occasional unmerited penalties under such a specification, the concrete supplier would have to work to 10 kg/m^3 (17 lb/yd^3) excess cement content, increasing the cost of concrete by \$0.70 (Australian)/m^3 (\$0.56 (US)/yd^3) above the cost strictly required, with the idea that this increase in cost is justified by the quality control benefits of the entire system.

On the other hand, a concrete supplier would occasionally escape penalization when actually supplying concrete as much as 1.22MPa (177psi) under strength. On average, though, the supplier would be paying a penalty of \$1.22 (Australian)/m^3 (\$0.98 (US)/yd^3) to set against the cement cost saving of around \$0.70 (Australian)/m^3 (\$0.56 (US)/yd^3).

Figure 12.11 shows the average penalty which would be applied and the 90% confidence limits on that penalty for strength shortfalls up to 4 MPa (580 psi). The graph shows there is very little risk of any significant unmerited penalty and even less chance of the cement saving outweighing the penalty.

12.3.4 Effect of *k* value changes

The effect of an increasing *k* value would be to increase the required mean strength. This could be offset by a reduction in the specified strength below that used in the structural design. The effect of such a compensated increase in *k* value would be to provide a greater incentive to attain a low variability in the concrete strength by imposing a larger safety margin on suppliers of higher variability concrete. The actual

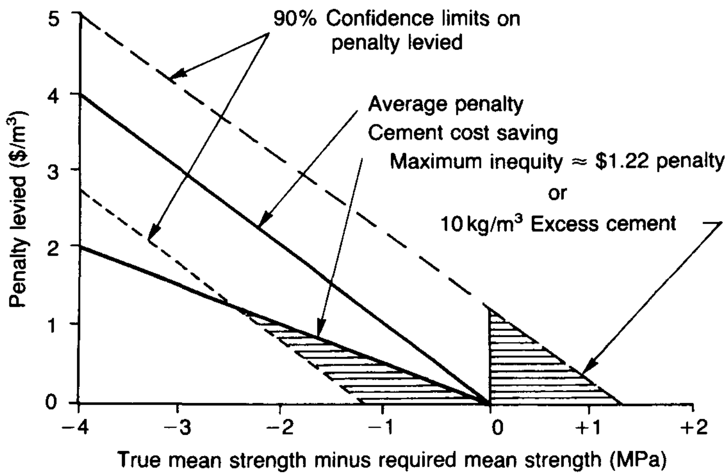


Fig. 12.11 Graph of average penalty applied.

minimum strength (say, the three standard deviation limit below which only one in a thousand results would fall) would be raised by such a specification.

In the author's view, an increased incentive to reduce variability and increase security against the occurrence of very low strengths would be highly desirable. It is suggested to use a *k* value of 3 and to reduce the specified strength by 5 MPa (725 psi) in compensation.

For a *k* value of 1.28 (existing US practice) and a specified strength of 30 MPa (4348 psi), the effect of this would be:

1. σ=2.5 MPa (362 psi) (good control):
   (a) required mean strength = 30 + (1.28 x 2.5) = 33.2 MPa (4812 psi)
   (b) effective minimum strength = 33.2 − (3 x 2.5) = 25.7 MPa (3725psi).
2. σ=5 MPa (725 psi) (poor control):
   (a) required mean strength = 30 + (1.28 × 5.0) = 36.4MPa (5275psi)
   (b) effective minimum strength = 36.4 − (3 × 5.0) = 21.4MPa (3101 psi).

For a *k* value of 3.0 (preferred), and a specified strength of 25 MPa (3623 psi), the effect would be:

1. σ=2.5 MPa (362 psi):
   (a) required mean strength = 25 + (3 × 2.5) = 32.5 MPa(4710 psi)
   (b) effective minimum strength = 32.5 − (3 × 2.5) = 25 MPa(3623 psi).
2. σ=5 MPa (725 psi):
   (a) required mean strength = 25 + (3 × 5. 0) = 40 MPa (5797 psi)
   (b) effective minimum strength = 40 − (3 × 5. 0) = 25 MPa (3623 psi).

The effect of the change would be to worsen the competitive position of the high-variability supplier and limit the occurrence of occasional low strengths in the concrete supplied. The low-variability supplier would be virtually unaffected, except for the supplier's improved competitive position.

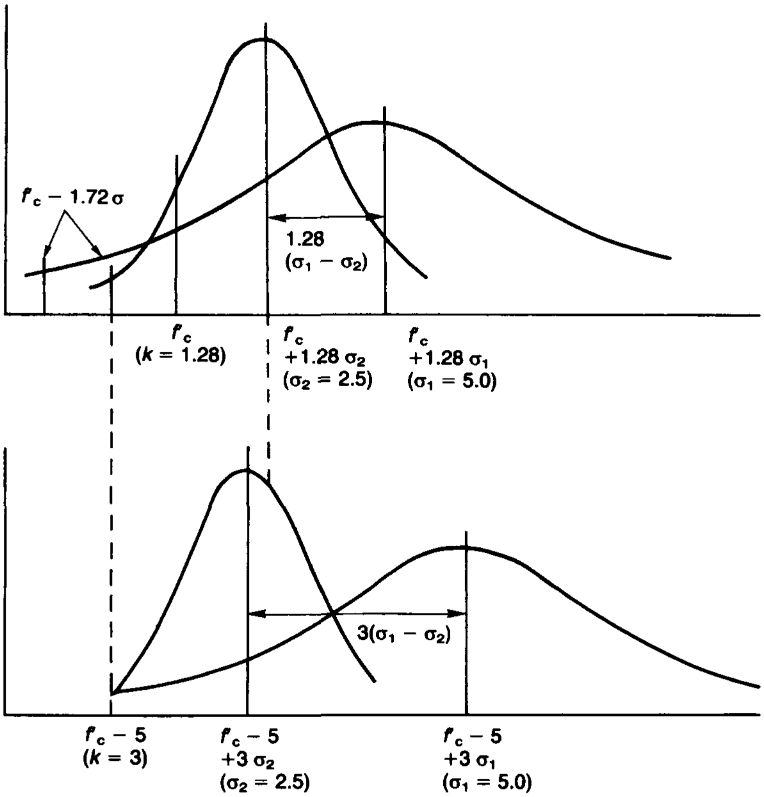


Fig. 12.12 Effect of compensated increase in *k* is to improve competitive position of low-variability supplier and rule out low results from high-variability supplier.

Figure 12.12 shows the relative situation under exact compliance with a 10% defective criterion for both high and low-variability suppliers. The upper graph shows that under the present (US) 10% defective basis, the low-variability supplier has a reduced incentive and the high variability concrete includes some deliveries of very low strength. The lower graph shows an enhanced competitive position for the low-variability supplier under the proposed 0.1% defective basis. Both suppliers in this case provide effectively the same minimum strength.

The benefits of low-variability concrete are substantial:

1. Helpful to the concrete placing crew.
2. More uniform compaction.
3. More uniform appearance.
4. More accurately assessed on a given number of test results (possibly less frequent testing required).

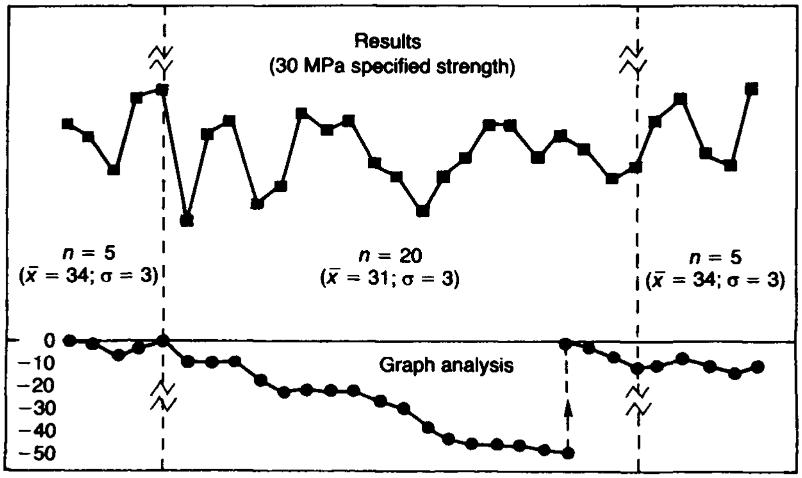


Fig. 12.13 Graphical analysis of run of understrength results which merits a penalty.

12.3.5
The influence of change points

The proposed technique assumes that there will be a gradual drift of either mean strength or variability and that it will be legitimate to select 30 results incorporating the worst period. Analysis has shown, however, that changes are usually 'step' changes rather than gradual drifts. Thus, a specific number of results constitute the low period and all of them (and no more) should be analysed to represent the low period rather than taking an arbitrary 30 results. This is too complicated and indefinite for use in a specification but could be applied with mutual agreement in practice. The effect of analysing 30 results overlapping a change point is to give an artificially inflated standard deviation which is only slightly compensated for by the increased mean strength obtained from the inclusion of a few higher results and, therefore, causes a higher penalty. An alternative, slightly lower penalty based on the actual defective period can be offered, but the specification can be strictly enforced without substantial unfairness.

Figure 12.13 shows a run of understrength results which merits a penalty. Under the proposed specification, the lowest 30-result section (representing 600 m^3 (785yd^3) of concrete) must form the basis. A penalty of \$2.28/m^3 would be applied, totalling \$1368.

Close analysis, however, reveals that the low strength concrete is confined to a 20-result section (representing 400 m^3 (523 yd^3) of concrete). The penalty/m^3 based on the 20 results would be greater but the overall penalty would be less at \$1136. The latter penalty is the more equitable and is the one which should actually be imposed. However, the difference is only \$232 and the 30-result basis is reasonably satisfactory and much simpler to incorporate into a specification.

The assumption is that the concrete supplier would have had to spend approximately \$1.50/m^3 in extra cement on the 400 m^3 (523 yd^3) of concrete to avoid penalization (total saving: approximately \$600 in cement cost), so the net cost to the supplier is approximately \$600. Obviously, the supplier would prefer to pay this penalty rather than delay the work and pay the costs of coring and investigating 400 m^3 (523 yd^3) of concrete, with the risk that some or all of it might be rejected.

12.3.6
Importance of quality of testing

It is of obvious importance that the test results forming the basis for a cash penalty should provide an accurate assessment of the quality of concrete as supplied by the producer. This is by no means something which is easy or can be taken for granted.

A minimum requirement is that samples should be taken, cured and tested by a competent, accredited, and preferably independent organization.

The best criterion of testing accuracy is the average difference of pairs of test results from the same sample of concrete. This average difference should not exceed 1 MPa (145 psi) for normal concrete (specified strength less than 50 MPa (7246 psi) and possibly excluding very low slump mixes). It is suggested that the highest of a pair of specimens is likely to be a better estimate of the true concrete strength than the mean of the pair.

The person responsible for result analysis should be alert for clearly established cases of incomplete compaction and improper curing and testing, and should be prepared to exclude such results from a penalty assessment. The previously recommended graphical analysis system, including analysis of related variables such as slump, strength, and testing, has been found valuable in distinguishing causes of variability and early detection of problems.

Parallel tests by two laboratories on the same truck of concrete reveal useful information and should be arranged from time to time.

The whole question of the reliability of concrete testing results is a matter which has received far too little attention. However, it is not a valid reason for failing to institute the type of cash penalty specification advocated here, as it causes even more trouble under existing types of specification.

No one can afford cheap testing. The best prospect of reducing testing costs is to reduce the frequency of testing, made possible by better testing, better specifications, better analysis of results, and a reduction in the variability of concrete.

12.3.7
Legal enforceability

Extremely crude forms of penalty are sometimes encountered, particularly on government work. Such penalties are enforced on the basis that future contracts will be withheld if they are disputed.

In British and Australian law, the key to legal acceptability is to relate the penalty to the harm suffered. It is assumed that a building owner would prefer to pay for the grade of concrete specified rather than accept a lower grade of concrete at lower cost. If the owner is supplied a lower strength concrete than specified, then he must have suffered harm in excess of the cost difference (in terms of margin of safety, durability, etc.) between the two strength levels.

Actually, the penalties considered here are too small to be worth a contractor's expense to legally challenge. However, the penalties are sufficient to ensure his co-operation in avoiding them.

What the law does object to are penalties specified to scare the contractor into compliance.

12.3.8
Experience in Australia

Although this proposal is now 20 years old, it has been applied to only one major contract to the author's knowledge. This was the Victorian Arts Centre (the Melbourne equivalent of the Sydney Opera House). On only one occasion did the results actually merit a cash penalty, which was paid.

However, thousands of cubic metres of concrete have been supplied to dozens of structures using the previously discussed control system, but without the cash penalty provision. On no occasion has it proved necessary to actually remove concrete from any of these structures.

Generally, concrete suppliers have been responsive to requests to adjust cement contents based on early age analysis. However, there have been frequent occasions when the strength provided, assessed as above, has fallen below that strictly required, for extended periods, by 1 MPa (145 psi) or less.

Such minor deficiencies have no structural significance but do waste time in repeated requests and reports and arguments with concrete suppliers (who are ever optimistic that the 7- to 28-day strength gain will improve on current production). Suppliers complain that precise enforcement is unrealistic, yet without strict controls, deficiencies would no doubt tend to gradually increase. A cash penalty as proposed would avoid all need for such argument. The deficiencies would be acceptable with the penalty paid, but it is suspected that deficiencies would rapidly disappear in such circumstances.

There have been suggestions that, in fairness, penalty clauses should be balanced by bonus clauses. This is not recommended because excess strength beyond that specified is of little benefit to the owner. The type of cash penalty clause advocated here is a real benefit to the good concrete supplier. He can aim at the mean strength truly needed without restriction. If he slightly miscalculates, the penalty is very moderate and involves no cost of delays or further investigation. He is defended from unfair competition by less competent or less scrupulous competitors. Finally, he can include his own bonus in his pricing if he wishes.

12.3.9
Conclusions

It is concluded that a cash penalty of twice the cost of the cement deficiency can be accurately established by the analysis of a group of 30 consecutive test results. Such a penalty would be capable of regulating concrete strength with fairness. The system would result not only in an improved degree of contractual compliance but also in a cooperative attitude in day-to-day control between the contractor and the supervising engineer. It would provide an effective incentive to improve control which would, over a period, produce significant improvements in concrete production techniques.

12.4
WHAT IS ECONOMICAL CONCRETE?

This section appeared in *Concrete International* (Day, 1982a). It is quoted verbatim as the author's views have not changed. Permission granted by the American Concrete Institute to reproduce it here is gratefully acknowledged.

The question 'What is economical concrete?' may seem a ridiculous question, but consider the example of the Rialto project in Melbourne. This project is very unusual in that the concrete supplier, the builder and the eventual owner were one and the same. It involved (amongst nearly 100000 m^3 of total concrete) 6000 m^3 of a 60 MPa (8700 psi) grade, which was the highest grade of concrete yet specified for such a project in Australia. Accordingly construction started with a very conservative mix which actually provided a mean strength of over 80 MPa (11 600 psi) and a characteristic strength of approximately 75 MPa (10 875 psi). Considerable cement content reductions (say, 100 kg/m^3 (170 lb/yd^3)) were clearly possible but no reduction was in fact made on the following grounds:

1. The possible saving of say $60000 was trivial compared to the total project cost of several hundred million dollars.
2. The huge strength margin virtually ensured that there would be no delays due to strength problems.
3. The very high early age strength permitted early stripping, etc. with no concern for damage, weather conditions, need for intensive *in situ* or early age testing, etc.
4. The additional safety margin against any unexpected factors was also of some value.

As another example, Australia's billion dollar Parliament House is a major concrete structure, containing about one quarter million cubic metres of concrete. At around 25 million dollars, the cost of the concrete supply represents about 2.5% of the total cost. It really would not matter very much if this cost increased 5% to 2.63% of total cost.

Of course, the extra cost in the case of the Rialto would be a little less trivial if the same argument were applied to the whole of the concrete in the project but the real point is that this attitude could never be taken by an independent concrete supplier because the cost would probably exceed the entire profit margin. The strength margin (but more likely 5 MPa (700 psi) than 15 MPa (2000 psi)) could therefore only come about by either the owner specifying a higher grade or the builder ordering a higher grade than specified. Either party might take this action on the basis of expediting construction, or at least of avoiding any risk of delay. In fact the best way of organizing this is for the owner to specify a higher strength but to impose a cash penalty rather than rejection or further investigation for strength shortfalls of up to 5 MPa (700 psi) (or whatever margin has been allowed). The same effect could be obtained by offering a bonus for excess strength (of course within a strict limit) and not raising the specified strength.

The benefits accruing from the proposed technique (of specifying a higher strength than strictly necessary and providing a cash penalty for strength deficiencies within the margin) would be:

1. A relaxed attitude to minor strength deficiencies by the owner.
2. A keener attitude to minor strength deficiencies by the concrete supplier.
3. A smoother running project.
4. The provision of better concrete, probably at only a very marginal overall cost increase.

There is yet one remaining possible turn of the screw of increased strength margin. This is to obtain the extra margin not by specifying a higher strength but by specifying a lower percentage defective at the original strength. This would have the effect of putting a higher premium on low variability and could be a substantial factor in discriminating in favour of better producers and so providing a beneficial pressure towards improved performance by the industry. If a strength increase of the order of 5 MPa (700 psi) is desired, it would amount to around 1.5 times the standard deviation. In most of the world, a 5% defective level is used, so that a mean strength of specified strength plus 1.645 times standard deviation is required. Raising the margin to 3 times standard deviation would go close to the 1 in 1000 defective level (mean–3.09 × standard deviation) and would mean, for a typical 3 MPa (435 psi) standard deviation, providing a margin of 9 MPa (1300 psi) between mean and specified strengths. The margin would vary between 6 MPa (870 psi) and 12 MPa (1740 psi) from the best concrete producers (SD of 2 MPa (290 psi)) to the worst we should tolerate (SD of 4 MPa (580 psi)). With such a pressure to improve, it is likely that in 5 or 10 years time, we would find the good operators down to below 1.5 MPa (220 psi) SD (margin of around 4.5 MPa (650 psi) as currently typical) and the rough operators out of business.

Perhaps an intermediate solution would suffice. A margin of much less than 5 MPa (say, 2 MPa (300 psi)) is probably quite adequate for the operation of a cash penalty system and this would be provided with an SD multiplier of 2 (giving around the 2.5% defective level of 1.96×SD). Incidentally it is time we stopped thinking of SD multipliers primarily in terms of permissible percentage defective. The real grounds on which they should be selected is the relative value we place on mean strength and standard deviation in assessing concrete quality (on this ground, a multiplier of 3 is highly desirable). The relationship between the desirable mean strength (or the 10%, 5% or 0.1% defective level) and the strength used in structural design calculations should be a subsequent rather than an initial decision, but is clearly an **independent** decision.

Interestingly, the cost of the additional strength margin now being proposed (or more) has often been incurred in the past by the specification of a 20 MPa (3000 psi) characteristic strength together with a minimum cement content requirement of the order of 300 kg/m^3 (500 lb/yd^3). There is however a very substantial difference in the results of the two specification bases. Whilst the former offers distinctly better concrete, a smoother running project (due to the cash penalty basis) and a pressure towards a better performing concrete industry, the latter offers scope for cheating on cement content, for the use of sub-standard aggregates and oversanded, high shrinkage mixes and, most important of all, a removal of any incentive for the technical competence of producers.

There are two important provisos which should be made in advocating cash penalties and greater emphasis on standard deviation:

1. The standard deviation (and the mean strength—but that is much easier) must be accurately determined.
2. The cash penalties (which may be described as 'liquidated damages' or 'provision for reduced durability' or formatted as a bonus clause rather than a penalty) should be very moderate, only about twice the cost of the additional cement which would have avoided any penalty (i.e. about 10 kg/m^3 per MPa (12 lb/yd^3 per 100 psi) of deficiency so, in Australia, about a $2 penalty per MPa of deficiency).

The requirement for an accurate SD is easily satisfied under a cash penalty system because it is not necessary to identify *which* concrete is slightly understrength—only how much and how defective. Therefore the penalty can be levied on the concrete represented by 30 consecutive results with great accuracy (section 12.4).

Does anyone have a convincing counter argument? If not, how long do you think it would take to implement this proposal? 5? 10? 20 years? It may be of interest that the outline of this argument was advanced in papers published by the author in 1959 and 1961 (Day, 1959, 1961).

12.5
INDUSTRIAL FLOOR SURFACES

The author has been involved in several floors and gained knowledge of others since the first edition, two of them of over 40 000 m^2. The first edition techniques set out below are still workable and result in excellent floors but there have certainly been further developments. Floors have used up to 100 MPa concrete strengths and have used silica fume and superplasticizing admixtures. One floor was cast in 50 m^2 panels prestressed at less than 24 hours to avoid cracking. Massive two and three bladed ride-on power trowels have been shown to be able to cope with concrete at a slightly later stage of stiffening than is possible with hand-operated disc compactors.

The principles which remain unchanged are that of delayed finishing to obtain a hard surface and permitting movement (shrinkage, or contraction under prestress) to occur without restraint if very large areas without joints are to be achieved without cracking.

The use of face numbers as a measure of floor flatness is also on the increase but the author has yet to have personal direct experience of it and so readers are referred elsewhere.

12.5.1 Background

The following notes are derived from the author's personal experience in specifying and supervising many floors of the type described. The participation of Barry Crisp of Lovell-Smith and Crisp, Consulting Engineers, in the initial development of the technique is acknowledged.

The type of floor envisaged is laid in strips 4 to 12 m wide and 50 to 80 m long. The end joints are sleeved dowel construction joints and the side joints are tongue and groove with no joint reinforcing. There are no intermediate sawn joints and no cracking is experienced provided the subgrade is adequate and detailing is correct. Thickness is normally 125 to 150 mm (rarely 175 mm) and a 40 MPa characteristic strength concrete is used. Reinforcing is normally 5 to 7 mm (rarely 8 mm) hard drawn rods in a 200 mm^2 welded mesh.

It is critical that the floors be laid with a completely flat soffit on a slip layer composed of 20 to 30 mm of clean sand overlaid by a polythene sheet and that edge details permit shrinkage movement without restraint.

The typical use for such floors is in high racking, narrow aisle warehouses, or warehouses with small hard wheeled traffic. The high racking use demands a floor flat to within 2 to 3 mm (the use of F numbers and techniques in measuring flatness is desirable but beyond the scope of the current volume) so that no sway develops in a high fork lift truck travelling at full speed with load raised and little side clearance. It is clearly necessary that no significant wear be experienced for several decades. To date the initial floors are over 20 years old and still giving satisfactory service.

Other uses for such floors include heavy industrial use in workshops and to provide dust free and sterile floors in the food processing and electronics industries.

It should be noted that the author has not yet had the opportunity to use either superplasticizers or silica fume in floors under his control. These two materials offer the prospect of reduced labour cost (but at increased material cost) without loss of quality. It is possible that they will actually provide substantially increased quality whilst reducing the level of skill involved.

12.5.2 Wear resistance

The wear resistance of a concrete floor is largely a property of the extreme surface layer. If it is to be free of dusting and retain its original surface completely, only the properties of the hardened cement paste will be involved. For long term resistance to very abrasive conditions, such as earthmoving equipment and tracked vehicles, the amount and hardness of the aggregate in the extreme surface layers and its bond to the matrix is also very important.

The hardness of the cement paste will be largely dependent on its w/c ratio, its chemical composition, and its degree of hydration (i.e. curing).

The best floor surface will contain the maximum amount of hard coarse aggregate interspersed with the minimum amount of mortar containing a high proportion of coarse sand and cement paste of the lowest possible w/c ratio which has been very thoroughly hydrated. To some extent these requirements are incompatible in that the coarser the sand, the less dense the packing of the coarse aggregate can be, and the lower the cement content, the harder it is to achieve a low w/c ratio.

It is easy to see that the worst possible floor will be one with a thick surface layer of mortar containing a high proportion of fines, placed at high water content and not cured. The only incompatibility here is that a high cement content can add to the problem although it must tend to reduce the w/c ratio. The foregoing is a fairly precise description of the surface which results from the early application of 'driers' consisting of stone dust and cement.

The eventual surface is a combined result of the original concrete mix and the finishing process. Both are important but the finishing process is the most significant factor.

12.5.3
Bleeding

A very significant factor is the bleeding tendency of the concrete. This has two aspects:

1. Increased water content in the surface layer,
2. Water voids formed beneath coarse aggregate particles and perhaps even beneath sand particles or even under a whole surface layer of mortar.

Bleeding is reduced by increased cement content and by increased content of fine sand. However, the latter especially is deleterious in other respects and must be avoided. The use of air entraining agent is an interesting point. Entrained air is one of the best means of inhibiting bleeding but must otherwise reduce wear resistance. Probably with a mix which would otherwise bleed badly and a poor finishing technique, entrained air would improve wear resistance. However, with a low bleeding mix and good finishing technique, air entrainment would certainly reduce wear resistance.

We see now that the best concrete to use is one with a harsh grading, low sand content, low fines in the sand, moderately high cement content and very definitely low slump or incorporating silica fume. It is not a full compensation to use a higher cement content to justify a higher slump. Such concrete is reasonably economical to purchase but is more expensive to place than more typical concrete. Pumped concrete raises an interesting point. Obviously in grading and in slump it is very antithesis of what is needed. However, pumped concrete cannot be as bad as the kind of oversanded 20 MPa concrete so popular amongst the majority of total incompetents, because concrete will not pump unless it has a fair cement content and is resistant to bleeding.

The characteristic strength of the concrete will need to be at least 30 MPa, preferably 40 MPa or more, however the harsh grading and low slump will provide 40 MPa at a cement content normally used for 30 MPa (say, 350 to 360 kg/m^3).

The best way of reducing bleeding has always been to use low slump concrete. However, it is possible to virtually eliminate bleeding at normal slump by using silica fume and a superplasticizer. It *is* instructive to note the difficulty in initially introducing what seems likely to be a substantial improvement. The technique **not** using these materials has been shown to provide very satisfactory floors lasting several decades. The anticipated labour saving will be heavily discounted as unproven by contractors. It is therefore difficult to justify the change to any client for a one-off floor.

However, any large organization with a need for several floors over a period of time would be well advised to try the new developments.

The use of fly ash gives some of the advantage of silica fume. Also it reduces water content and bleeding but does not justify the use of a higher slump.

12.5.4 Mix specification

Unless using silica fume, the requirements are a harsh grading, a low sand content, low fines in the sand, a low w/c ratio and a low slump. This is one occasion on which it is useful to specify slump limits, since the kind of concrete is known. The requirement for a harsh grading, etc., can be briefly stated but should be ensured by stating a **maximum** cement content. The maximum cement content specified will depend somewhat on the quality of the cement available but should be increased only slightly and with reluctance.

The recommended specification is therefore:

- Characteristic strength of 40 MPa.
- Slump limits of 30 to 45 mm.
- Maximum cement content of 350 to 360 kg/m^3.

It is preferable to use 25 to 35% of fly ash rather than all cement except that in very cold conditions this may delay finishing operations excessively. (However, such floors are normally done inside a building so heating may be practicable). If using fly ash, the requirement should be a maximum of 250 kg of cement plus 100 to 150 kg of ash, depending on the quality of ash available.

12.5.5 Compaction

The harsh, low slump concrete selected will certainly require compaction by both internal and surface vibrators. Such a mix (of, say, 30 to 45 slump) compacts readily under 'poker' vibration but is quite difficult to strike off level once it is placed too high. It is therefore important to judge the thickness of the spread concrete as accurately as possible so that after compaction by internal vibration there will be only a very little surface trimming to do by a twin beam vibrating screed. It is also important to use a very stiff vibrating screed because more flexible varieties have been known to ride up as much as 20 mm or more in the centre of a 6 to 7 m wide strip when used on this kind of concrete.

It is to be appreciated that one objective of the mix design and placing technique is to leave little or no surplus mortar on the surface of the compacted concrete. Unless tackled in a proper manner such a concrete appears almost impossible to handle but an experienced crew with proper equipment makes it look easy.

12.5.6 Surface working

Having placed, compacted and struck off level the concrete, it is imperative to leave it absolutely untouched until bleeding is complete and the bleeding water removed. Of course with the right concrete, this stage may be quite short. The point here is that a limited amount of bleed water can pass through concrete with a minimum of disturbance but if the concrete surface is disturbed at this stage, the surface layer effectively acquires infinite w/c ratio. Furthermore, particularly after the application of traditional ‘driers’ (stone dust and cement) the action of a steel float is to make the surface layer impermeable to further bleed water. Any such bleed water then accumulates immediately below the surface layer and is the cause of the surface flaking sometimes experienced.

Further working of the surface requires the simultaneous existence of two factors. One is an absence of bleed water (without the surface actually being dry). The other is the right state of hydration (without the concrete having become too hard). Although these two requirements do often coincide naturally, this is not always the case. If the concrete becomes ready to trowel whilst there is still water on the surface, the water must be squeegeed away as gently as possible. If the surface dries before the concrete is firm enough to work, it is permissible to apply a very fine mist spray of water (or erect windbreaks, sunshields, etc.) to avoid presetting cracks.

Satisfactory working of the surface is certainly possible by skilled trowel hands, but it is better and more economical to use mechanical equipment. Working of the surface has three purposes: to produce a smooth surface (or lightly textured by a very stiff broom at a late stage in the finishing process), to compact the surface layers and to squeeze water from the surface layers. The extent of improvement of wear resistance which is possible has not been adequately realized. The author has shown (in unpublished work) that extraction of water by vacuum processing does not produce a corresponding strength increase, unless it is followed by revibration. This makes again the basic point that strength (and wear resistance) is governed by the ratio of water plus voids to cement, not pure w/c ratio.

12.5.7
Power trowelling

It cannot be too strongly emphasized that early trowelling is highly detrimental to the floor surface. However, leaving finishing too long will also cause severe problems.

The flatter the surface left by the original striking-off (with a vibrating screed), the harder it can be allowed to become before finishing commences. Ideally it should be left until a man can walk on it (in soft shoes) without leaving an impression.

The first pass of the finishing equipment must completely level the floor. Preferably this will happen through depressing the slightly high areas with only a very small transfer of mortar from one area to another and with little or no additional mortar being brought to the surface. Working too early will bring additional mortar to the surface and reduce wear resistance. Working too late, the surface will not reshape and slightly low areas will receive only a surface skin of mortar rather than being fully reworked.

The best equipment to use at this stage is a rotary disc compactor incorporating a hammering or vibrating mechanism. In the absence of such equipment, a normal, four bladed, rotary trowel will be used with its blades initially adjusted to flat or almost flat. The surface can (and must) be left longer before working with the disc compactor than with the trowel alone.

Each pass of the finishing equipment releases further bleed water from the surface layers and there must be a delay between passes to allow this water to evaporate and the gel structure of the cement paste to stiffen again. Failure to allow a sufficient delay between passes will cause softening of the surface and extend total finishing time.

After the first pass it should not be necessary to redistribute particles, especially not coarse aggregate particles. The objective is to settle the aggregate particles more firmly into the cement paste and to displace as much as possible of the entrapped moisture. Where a disc compactor is not available, the blade angle of the power trowel should be gradually increased, so reducing the contact area and increasing the pressure exerted on the concrete surface.

After three or four passes of the disc compactor, further water will not be squeezed from the surface and finishing reverts to use of the power trowel to obtain the desired surface texture.

12.5.8 Floor finish

If power trowelling is continued for a sufficient length of time (with appropriate delays) a high strength concrete will 'burnish', i.e. become shiny and black.

The black 'layer' essentially has no thickness yet floors have remained black and shiny after more than 10 years of intensive use.

Where anti-slip properties are essential, it is possible to stop just short of the burnishing stage and impart a very light texture with a stiff broom. However, the burnished floor is not as slippery as it looks unless allowed to become wet and greasy.

12.5.9 Black floors

There is no question that burnished floors (if all cement, less so with fly ash) are black but there is some disagreement as to why this should be. Some consider that iron from the floats and compactors is involved. Certainly it is true that the colour of cement paste is heavily dependent on its w/c ratio. It is a very light grey when of high w/c ratio and very dark grey when of very low w/c ratio. It seems likely however that the explanation is that this is the natural colour of cement clinker. This will be seen if cement is shaken with an excess of water which is then decanted. Some black grains will be seen remaining. The cement particles look grey because they are coated with a fine grey dust. The suggestion is that the black surface results from the removal of this dust by the trowelling. This would explain why the black layer has no thickness, as can be seen on a drilled core or a removed chip of the concrete. The concrete simply has a black face but no layer of this can be removed.

12.5.10 Dry shakes

The addition of any material to the surface of the type of concrete described (prior to trowelling) is almost always unnecessary and likely to be detrimental if anything. It may be a slight improvement to scatter coarse carborundum grains on the surface. However, if the grains do not project above the surface and no wear occurs, it is difficult to see how they could provide any non-slip effect.

It is possible to improve the wear resistance of a normal floor to be composed of 20 or 25 MPa concrete by the use of a dry shake. The use of proprietary materials in a controlled manner is certainly likely to be effective but the overall cost may exceed that of the 40 MPa burnished floor without providing as much resistance to wear. Some such products can provide an almost white and very non-slip floor of high wear resistance where a shiny black finish is unacceptable.

To the casual observer, a dry shake finish may be indistinguishable from the use of 'driers'. Many find it difficult to accept that one can improve the surface and one worsen it.

There are in fact three quite vital differences:

- The intention in using the material.
- The material applied.
- The time at which it is applied.

The intention in using 'driers' is to enable earlier and easier finishing and to produce a smoother, more visually attractive surface. The intention in using a 'dry shake' is to improve the wear resistance. This is to be accomplished by reducing the w/c ratio of the surface layer and by introducing more hard material into it.

If follows that in the case of driers a mixture of cement and fine stone dust will be applied to soak up the bleed water and be immediately (and gently) steel floated to give a smooth finish.

The dry shake on the other hand will contain no dust whatever (except cement) and will contain hard coarse (say, 1 to 5 mm) granules. The granules may be a coarse sand (quartz is very hard) or crushed hard basalt or a harder, more expensive material such as carborundum or metal. It will be sprinkled onto the surface at a much later stage than driers, after bleeding has ceased and most surface water gone. This material will not make trowelling easier (it may make it harder) or give a smoother finish. However, it will, by adding dry cement, reduce w/c ratio and it will ensure the presence of wear resistant particles at the very surface.

12.5.11 Curing

Thorough curing for several days is absolutely essential for high wear resistance. However, it is also essential that the floor be dry before being subject to wear as the wear resistance of a 'young' floor is much reduced when it is wet.

The point is that the cement will continue to hydrate and provide better strength and impermeability as long as it is kept wet but that it remains comparatively soft until dry.

The use of a wax or chlorinated rubber based curing compound may offer some degree of early protection as well as satisfactory curing. Polythene is better than wet hessian for curing because it can be used earlier without affecting the surface texture.

Silicates and fluorosilicates are effective in producing some increase in wear resistance in lower quality floors. However, very high quality floors of the black and shiny variety may be so impermeable as to make such treatments ineffective.

Where floor slabs crack, it is often as a result of thermal stresses rather than drying shrinkage. The greatest risk is at the time of minimum overnight temperature on the first morning after placing. This is the time at which the ratio of stress to strength is likely to be a maximum. The use of a polythene sheet for curing provides a small degree of heat insulation by entrapping some air. A larger benefit is that it eliminates evaporative cooling.

The preferred practice is to use **both** a chlorinated rubber or wax curing **and** a polythene sheet. The curing compound should be applied progressively immediately trowelling is complete and the sheet prior to leaving on the first day. The sheet may be recovered and reused after 24 hours.

It is important to realize that some risk of surface crazing is created if a polythene sheet remains in position for several days to avoid use of a curing compound. This is because the surface layer will dry quickly on removal of the sheet whilst the body of the slab remains wet. The function of the curing compound may be seen to be to provide controlled slow drying as much as to retain moisture for curing.

12.6
HOW SOON IS SOON ENOUGH?

The first edition contained a 21-page account of an investigation using a massive computer analysis of synthetically generated data to clearly establish the superiority of cusum analysis over any other system known to the author for the early detection of change. The graphical presentation of data in the first edition is not repeated here but is available on the accompanying CD-ROM (see *How Soon is Soon Enough* on that CD) and the introduction and conclusions from the first edition are repeated below.

The two most significant points arising were:

- No computer analysis is as efficient and reliable as the eye examination of a cusum in detecting a small change in mean strength.
- The mathematical significance of a downturn in a single variable (i.e. strength) is in any case immaterial when the significance of the downturn is confirmed by simultaneous changes in other variables such as slump, temperature and density.

Nevertheless we do have clients who like to use the built-in autodetection of strength downturns, if only during the early stages of familiarizing new staff in the use of the system. It also enables Head Office staff to feel more comfortable in leaving distant small branches to operate their own control. However, we at Concrete Advice Pty Ltd are of the opinion that this security function would be better provided as an incidental result of centrally analysing all a company's data in a search for improved economy and greater knowledge of the company's material resources and operating techniques.

The investigation was an extensive and on-going computer comparison of the relative efficiency of various specification and analysis systems in detecting the onset of a change in concrete quality. A large range of systems have been examined, including a numerical cusum analysis system and the current American, Australian and British methods. However, no such system is as well able as a cumulative sum ('cusum') graph to pinpoint retrospectively the exact time of onset of a change.

The economic value of a more efficient analysis system is briefly compared to that of other factors affecting the attainment of the desired concrete quality, such as better equipment, more skilful personnel and higher testing frequency. It is pointed out that a more efficient detection system is equivalent to a higher testing frequency in achieving early detection. It is shown that the average number of results required to achieve detection of a change is directly proportional to the standard deviation of those results. Since early detection in turn enables a reduction in variability, a self-intensifying cycle of variability reduction is commenced.

12.6.1
The problem investigated

The question 'How good is good enough?' has been raised and answered by others (Abdun-Nur, 1962; Chung, 1978). However, it is possible that it was not the most important question to ask. Whatever the level of concrete quality required, it is obvious that from time to time there will be fluctuation in the quality being provided. The question asked here is how soon after its onset a quality downturn can be detected and rectified. The 'How soon is soon enough?' question has already been asked by Mather (1976). His answer was 'before the concrete is discharged from the mixer'. The author would agree with this in so far as the possibility of substantially defective concrete is concerned. However, only compressive strength will enable fine tuning of a mix taking into account small variations in the quality of all materials including cement, admixtures and aggregates and interactions between them affecting bond, air content, etc.

For many years the author has been presenting real data examples of control (Day, 1981) and claiming high efficiency at early detection of problems. Working with real data has the disadvantage that it is necessarily limited in extent and hopefully displays a quite limited frequency of strength downturns. If two analysis systems are being compared on real data, the situation is not necessarily clear when one detects a downturn and the other does not. It may be that the downturn was purely a temporary chance aberration and that the analysis system which did not detect it was the better, since it avoided a false alarm.

The solution to these problems is to use synthetic data. It is possible to program a computer to carry out thousands of comparisons of alternative analysis systems until their relative efficiency is clearly established in numerical terms. While this is by no means the whole story, it certainly has the potential to provide a factual and unbiased basis for comparison. The approach is an alternative to the use of statistical techniques to generate operating characteristic curves. Experience has shown that such curves are not well understood by many persons involved in concrete QC and (perhaps therefore) have not provided the industry with a clear picture as to the performance to be expected of control systems in real situations. The techniques used here are also capable of generating data which does not conform exactly to a normal distribution but to slightly different patterns encountered in actual data.

The author has used visual inspection of cusum graphs (unaided by any limit lines or V-masks but greatly assisted by simultaneous plotting of related variables such as slump, density and temperature) as essentially his sole QC tool since the mid 1970s and has not personally felt the need for the type of numerical backup investigated here. However, the reality is that, in general, control systems are not going to be operated by keen, experienced concrete technologists but by technicians, many of whom may be only mildly interested and reasonably competent. It is therefore of substantial value to have a system which, as an early age result is entered into a computer, immediately and automatically converts it into a predicted 28-day result and analyses whether or not it constitutes a change point. When such a warning appears on screen, the technician can view the cusum graphs and determine:

1. Whether the warning appears justified or possibly the result of a statistical aberration (by consideration of the performance of the related variables).
2. When the downturn actually occurred (this permits consideration of correlation with delivery times of input materials and also of how much concrete may be at risk of being sub-standard).
3. What caused the problem (again by comparison with related variables).

The investigation to date has covered not only the performance of the formal systems as they appear in the respective national codes, but also the potential performances of the basic concepts embodied in those codes. It has not yet extended to the use of data with modified normal distributions.

Many detection systems and many variants of those systems have been examined in detail but only a small illustrative fraction of the data generated was presented. The main systems investigated were:

- The American ACI 214.
- The Australian AS 3600.
- The British BS 5328.
- Numerical Cusum.

The basic techniques separately examined are the running means of 3, 4, 5 and 30 results and individual results. The Shewhart system was originally included in the investigation but it gave the worst performance and was eliminated.

The questions of early age and/or accelerated testing, of monitoring batching performance, of analysing related variables such as slump, density, temperature, etc., have been addressed elsewhere in this volume. For the purpose of this investigation it is assumed that a continuous string of test results is being received and converted into predicted 28-day results. The relative efficiency of the different techniques in detecting a downturn in such a string of results is examined (Table 12.2).

The assumption made by the author, after 40 years of plotting quality control charts for concrete, is that the downturn is usually a sudden event or 'step change' rather than a gradually worsening trend.

12.6.2
Investigation details

First-stage program

A Lotus spreadsheet computer program was set up to automatically produce a string of 100 random, but normally distributed, results of any selected mean and SD. It then appends a further 30 results of the same standard deviation but a lower mean. This enabled examination of the performance of a control system in respect of whether it raised false alarms during the initial stable period of 100 results and how long it took to detect the imposed change point at the 100 result mark. The results

Table 12.2 National criteria as in national codes

	ACI 214	*AS3600*	*BS 5328*	*N Cusum*
false alarm frequency	52.36	93.6	46.81	70.74
average detection delay	1.75	12.90	2.64	4.11
maximum average detection delay	8.06	20.15	7.26	10.54
Adjusted (by constant margin) to comparable false alarm frequency:				
adjustment in char strength (MPa)	1.75	−0.60	6.50	NA
false alarm frequency	63.8	64.5	77.8	71.2
average detection delay	6.4	10.4	12.0	6.3
maximum average detection delay	17.6	20.3	22.5	16.0

were automatically analysed by up to six different detection systems at a time and the results reported as:

1. The number of results prior to a false alarm in the first 100 results; if the number is 100, there were no false alarms.
2. The number prior to the first detection of change after the imposed change point; if the number is 30, there was no detection.

For each selected standard deviation and depression of the mean, the program carried out 20 replications, i.e. it produced 20 fresh sets of artificial results for analysis and stored the results of each before passing on to the next appointed SD and depression of the mean. The program permitted a table of any number of SDs and mean strengths after the drop to be entered. It would then work through the table without requiring attention, taking from 4 to 15 min per set of 20 trials depending on the computer used (ranging from an 80386 to an 80486–66 IBM compatible desktop). In each case 35 was used as the initial mean strength, the assumed specified strength being varied as SD changed.

It was found that the detection efficiency was strongly related to the ratio of the imposed reduction in mean to the SD (subsequently called the 'drop ratio'). The normal pattern was to select five SDs covering a wide range. For each SD a range of 5 second means (i.e. means after the imposed strength reduction) was then selected to cover a range of strength reductions of 0.4 to 1.75 times each of the selected SDs. This permitted the influence of both varying size of change and varying SD on the performance of the six systems to be examined. Although many alternatives were tried, it was found desirable to standardize on a set of SDs and drop ratios.

Second-stage program

A small second Lotus program takes the results of each set of 20 trials (all being of the same nominal SD and second mean, but each with a freshly obtained set of random numbers) and analyses them for :

1. Average number of results prior to a false alarm.
2. Average and maximum number of results between change point and detection.

These parameters are output in the form of a single row of numbers which also includes full details of the system parameter settings and the intended and actual means and SDs of the results. The rows could then be sorted and analysed according to any selected criteria when entered in a third Lotus spreadsheet.

Third-stage program

The third spreadsheet automatically receives 25 rows of data for analysis, summarizing the relative performances of six analysis systems at 25×20=500 imposed change points of assorted SDs and changes in mean.

One (duplicated) row of this spreadsheet averages each item in all the 25 rows. Ten other rows display the results of data sorts averaging the items in smaller sets according to two sets of criteria. One set was the five SD values; the average of the results from each of the five sets of five rows using a common SD were automatically displayed on five rows of the spreadsheet. The other set was selected by five ranges of drop ratio, actual rather than intended values of this being used (the results analysed are selected from a pool having exactly the desired mean and SD but the samples obtained necessarily display small random variations in both mean and SD).

12.6.3
Tuning of systems

The best detection system is not necessarily the one which shows the lowest average number of results to give a detection. Any type of system can be made more sensitive by narrowing its limits, at the cost of experiencing more false alarms. It was not considered sufficient to find that one system was extremely good at detecting changes but gave many false alarms, while another gave few false alarms but was a poor detector. It is certainly of interest to compare the relative severity of different national codes but the author's primary interest is in finding the most efficient way of detecting a change. The exercise was therefore repeated after adjusting the nominal specified strength so that each system gave similar false detection frequencies when assessing the same concrete.

It was found to be important whether the adjustment was in the form of a constant or that of a multiplier of the SD. The various national systems often incorporate a fixed adjustment, e.g. ACI 214 requires not more than 1 in 100 results to be more than 500 psi (3.45 MPa) below the specified strength and BS 5328 requires the running mean of four results to exceed the specified strength by at least 3 N/mm^2 (3 MPa). This investigation has shown that unless such adjustments are expressed in terms of a multiple of the standard deviation, the systems would give a substantially different relative performance according to whether the production was at high or low variability.

Whilst a false alarm would usually (especially using multivariable cusums, section 5.8) be quickly detected as such on further examination, it must represent at least a small waste of time. This would probably not be acceptable more than about once in 50 results on average and should preferably be in the 70 to 100 result range. For the purpose of this exercise, the false alarm occurrence was adjusted to be generally between 70 and 80 results. To ensure the significance of the findings, the adjustment was deliberately slightly skewed so that the best performing systems had **both** a lower frequency of false alarms **and** a faster detection.

Another aspect of system efficiency is the use of multiple criteria. A system can be made to give a better ratio of correct detections to false alarms by composing it of several sub-systems running in parallel. In this case the better performance is obtained at the cost of a more complicated criterion, a larger program and slower operation. With computer assessment, these costs would be negligible compared to increasing physical testing frequency **and it should be realized that a more efficient analysis system has as much value as additional testing**. For example it would be possible to analyse results using a combination of all the systems included in the investigation and to accept that a downturn had occurred when one was detected by any two, or any three, of the nine systems shown. This would no doubt give both a better detection rate and less frequent false alarms. However, the improvement would probably be relatively small since false alarms are frequently due to aberrations in the results, affecting several systems, rather than to aberrations in one of the detection systems. (In this respect it would be of value to persons involved in concrete QC to examine a selection of the data generated for this investigation in order for them to realize the extent to which apparently convincing downturns in a set of results occur as a result of normal statistical variation).

The real reason militating against the multiple criteria approach is that it must still be suitable for the average user. Complication must be avoided as far as possible, both to ensure comprehension by all concerned in its enforcement, and to avoid the much greater effort of examining compliance by manual calculation by persons not having computer knowledge or facilities. It is conceivable that this requirement could be relaxed at some time in the future when it may become reasonable to assume that there would never be a need to analyse results without a computer.

12.6.4 Relative performance of the systems

All the systems, except ACI 214, are nominally directed towards assuring a characteristic strength which 95% of results will exceed. Therefore that characteristic strength is given by mean minus 1.645×standard deviation, i.e. for this exercise, 35–1.645SD.

In the case of ACI 214 the requirement is for only 90% of results to exceed the specified strength. Therefore that strength in this exercise becomes 35–1.28SD. However, in the adjusted limit section the ACI system is still comparable as what is reported is the amount of adjustment required.

It can be seen that both the ACI and the UK systems, in their original forms, give rapid detection of a downturn but also give a high rate of false alarms. The Australian system on the other hand appears unduly lenient. The numerical cusum was adjusted to comply with the 70/80 false alarm frequency during the process of selecting the deduction margin and detection limit. This was done as a separate exercise using the techniques of this investigation in which a large range of margins and limits were tried (in sets of six) to find the most efficient combination.

The basic techniques embodied in the national codes (individual result limit and limits for running means of 3, 4, 5 and 30) were also separately examined. This was necessary because some of the combinations were optional and also to avoid concluding that the code incorporating the largest number of individual criteria (ACI 214) was necessarily the best.

12.6.5 Visual cusum

In the (very lengthy) initial stages of the investigation, hundreds of cusum graphs were examined. It was noted that the basic cusum graph almost invariably showed a quite distinct downturn at the exact point of the artificial downturn, even when the 'drop ratio' was so small that the numerical system detection efficiency was poor.

It should be noted however that this is far from the same thing as concluding that the detection efficiency of the basic cusum is almost perfect. The technique looks better in retrospect than it does in genuine use. Examination of the overall 130 result trial tells nothing of how many of the small false downturns in the cusum graph might have been mistaken for the real downturn, or for how long an operator might have regarded the real downturn as such a false one. So, while the keen and experienced operator using cusum graphing will already have acted before the detection system provides a signal, the less experienced operator will be glad of the confirmation provided by the system and the less keen operator will be prodded into action.

What is clear is that, on looking back after concluding that a downturn has occurred, the basic cusum graph will show exactly when that downturn occurred. This is very valuable information because the same logic applies to any other variable for which a cusum graph is drawn, and therefore it is usually easy to match up cause and effect.

System details

ACI 214

Criterion 1—not more than 10% of results below *fc*'.
Criterion 2—not more than 1 in 100 running means of 3 less than f'_c.
Criterion 3—not more than 1 in 100 individual results more than 500 psi (3.45 MPa) below f_c.
Criterion 4—not more than 1 in 100 individual results below 0.85 f'_c.
Control chart recommendation: Running mean of 5 to exceed f'_c

BS 5328

Criterion 1—running mean of 4 to exceed f'_c+3 MPa.
Criterion 2—no individual result more than 3 MPa below f'_c

AS 3600

Production assessment:
Mean strength for last month (last 2 months if less than 30 samples per month) to exceed $f'c$ +1.25 × SD.
Project assessment:
Mean of sets of three not less than f'_c (representing a defined section of concreting, so not really running mean of three as has been used in this comparison)

NUMERICAL CUSUM

The previous mean value is subtracted from each result and if the difference numerically exceeds a selected margin, the difference (less the margin) is accumulated in a register. If the accumulated total exceeds a selected limit, a detection has occurred. In practice positive and negative registers are maintained (because detection of an upturn means that cement can be saved, which is a further reason to prefer numerical cusum), but for the current exercise, only a negative register was maintained.

For any selected margin, a limit can be chosen to give whatever frequency of false alarms is considered acceptable. It is conventional to choose a margin of about half the minimum change it is desired to detect. If this is considered to be 0.5×SD, then SD/4 might be the chosen margin. The investigation reported started with a margin of SD/3 and a limit of 4×SD, but after comparative trials the best results were obtained with a margin of SD/6 and a limit of 5.5×SD.

The use of a numerical cusum in this way is exactly equivalent to using a graphical V-mask technique (Devore, 1990) as is used in the UK.

12.6.6 Assessment of alternatives

On average and after adjustment to a comparable false alarm frequency, the running mean of five gives the quickest detection. However, the numerical cusum follows close behind and is better at detecting very small drops. Numerical cusum is also more directly aimed at detecting change from a previous situation rather than infringement of a specified limit. Since a producer would be ill-advised to work right down to the limit, the latter is likely to be the more useful feature. Numerical cusum is also equally at home in detecting upturns and this is important to the producer. Of course a running mean of five can be adapted to all these purposes but this is not often done.

The national systems are not strictly comparable as they have different intended methods of application. The American ACI 214 publication sets out a range of possibilities together with several pages of excellent advice and information with the objective of allowing specifiers to make their own informed decisions. It also includes a recommendation to maintain control charts and detailed advice on how to do so.

The British Standard BS 5328 condenses its unequivocal requirements into a small table and four carefully chosen sentences. To be fully comparable with the ACI system it would also be necessary to make reference to the requirements of the British Quality Scheme for Ready Mixed Concrete which is an industry based self-regulatory body and recommends cusum control charts or an alternative ‘counting rule’ system involving not more than eight consecutive results below the previous mean.

The Australian system provides a rule by which concrete producers are required (regardless of individual project specifications) to regulate the whole of their production. It then also provides a rule by which individual projects can check the quality of concrete received by that project.

In comparing the requirements it should be remembered that the British code is anticipating approximately double (4 to 6 MPa) the standard deviation normal in Australian capital cities (2 to 3 MPa), with the USA covering a larger and intermediate range. It could also be said that the Australian code is designed to avoid unfair condemnation of the producer and allow full benefit for the attainment of low variability, while the British code is aimed solely at providing near certainty that the supply of sub-standard concrete will be eliminated in all circumstances. It appears that the carrot may be currently showing greater benefit than the stick.

The use of a minimum required strength for any individual specimen has good and bad points. It is reasonable to put a limit to the downward spread of results which could be obtained with very high variability concrete whilst still providing a mathematically acceptable mean. However, test results are subject to error and an individual specimen criterion can require action on the basis of a badly made test if not intelligently administered, and the author's experience is that such matters are often not intelligently administered.

The use of a fixed lower limit for individuals may have its merits but the use of a fixed numerical limit for the running mean of a set of four, as in the UK Code BS 5328, has the unfortunate effect of severely limiting the financial benefit obtainable from good control. As previously noted, any kind of requirement involving a constant produces distortions in performance over a range of SD values.

One final answer to the 'How soon?' question must be 'Before anyone risks their neck'. It is quite possible to assess concrete quality within 24 hours and it is probably legally, and certainly morally, indefensible not to do so prior to prestressing, early demoulding, jump form movement etc.

12.6.7 Other significant considerations

Where cost competition is negligible, it is easy to provide a large safety margin, so totally avoiding failures. In these circumstances a highly-tuned control system may not be essential but is obviously affordable.

Where cost competition is severe, a control system which can detect a shift in mean strength of as little as 1 MPa (150 psi) within 2 or 3 days of its occurrence may be an excellent investment. Where operating conditions and materials are very stable, the additional cost of early age testing may not be justified. Seven-day testing has the advantage that, on detection of a suspected downturn, a reservoir of test specimens from 1 to 6 days age is available and can be immediately brought forward for test to confirm or negate the change. This is providing one is sufficiently knowledgeable (and has done the necessary prior investigations) to correctly interpret results at a range of early ages.

The control process should be considered as a whole, ensuring value for money in several different types of expenditures, e.g.:

1. Batching equipment.
2. Quality of testing.
3. Frequency of testing.
4. Computer equipment.
5. Computer software.

The ability to work to a 1 MPa (150 psi) lower mean strength for *a* given specified strength is worth about 5 kg of cement per cubic metre (8.4 lb/yd^3). This is a sufficient saving (on high volume production) to pay for a very elaborate control system. The ability to detect a downturn in strength a day earlier may avoid a major penalty. It may also justify a lower safety margin.

It should be noted that all criteria relate to the standard deviation of results. Lower variability concrete is easier to control more precisely. As already noted, this is not tautology but a recognition of a multiplier effect of control improvement. A reduction of 1 MPa in standard deviation makes a direct difference of 1.28 or 1.65 MPa to the required target strength (depending on whether the specification is based on 90 or 95% above). It will make at least a further 1 MPa reduction in the strength margin required for the detection of a change. Improved quality control may also be a major sales point. The standard deviation of the concrete strength is obviously affected by the quality and effectiveness of both the batching system and the testing process, as well as by the variability of input materials.

The frequency of testing is an important cost factor to be weighed against the quality of testing, the securing of additional data such as slump, concrete temperature and density, and the cost of result analysis. The cost of elaborate analysis is rapidly reducing compared to that of physical testing **and an increase in one can justify a reduction in the other**.

The ability of a control system to combine results from many different grades of concrete into a single analysis can be equivalent to a several fold increase in testing frequency.

The time between a downturn and its detection and rectification is also affected by the age at test. The days in which mix revisions were based on 28-day test results are hopefully gone, but the choice of test age in the interval of one to seven days is open to consideration. In temperature stable tropical conditions, 3 days is a good choice. Depending on the protection provided to the specimens, and on the time of collection, a 3-day strength may be too variable in other climates. Further options are to use accelerated specimens or to measure thermal maturity in order to obtain a result at 1 to 2 days.

A consideration of the above factors makes it clear that:

1. Except in very low volume situations, there is ample saving in cement cost to offset a high standard of control.
2. The cost of computer analysis with a good class of computer and software is modest compared to other factors in achieving timely control.

13
Troubleshooting

There are several aspects to troubleshooting in concrete technology. One of them, separation of its costing from that of QC, was raised in the first edition and is repeated here.

Another is the examination of existing structures with a view to repair. This is a field in which the author has considerable experience but has for several years done his best to avoid. Some reasons for this attitude are:

1. The field is a very extensive and rapidly developing one and, to provide good professional service, requires that the practitioner keep fully up to date with a myriad of constantly changing techniques and proprietary materials. The author is unwilling to divert enough time and effort to this aspect of concrete technology from his chosen fields of mix design and QC to satisfy his conscience in being such a practitioner.
2. Repairs to concrete structures are very often temporary (unintentionally, that is) and may provide only a short term cosmetic effect at considerable expense. The author does not wish to be involved in such situations.
3. Clients are often unwilling to face up to the very expensive solutions which may be necessary to achieve a degree of permanence.
4. Even the experts have difficulty in establishing which of several competing repair proposals represents best value for money (or whether any proposal offers good value).

However, it should be pointed out to younger readers that this field is likely to absorb something like half the total expenditure on concrete structures in the next few decades. It is also likely to generate distinctly more than half the profits to be made out of concrete technology in this period. This is because typical clients are far more willing to pay for cure than for prevention (even if not enough for reasonably permanent cure). Therefore the author does not encourage others to adopt his own attitude.

The author is from time to time paid significant amounts of money to sort out problems with concrete still in the production stage. Advice on the procedure to follow seems desirable since the kind of action necessary in many (but not all) such situations is reasonably easy to learn (compared to repair), and since even relatively amateur attempts to follow the advice given are likely to be beneficial, even if not necessarily optimum.

The first action must be to establish exactly what the problem is. Some possible problems are:

- Inadequate strength.
- Lack of pumpability.
- Inability to compact.
- Unsatisfactory appearance.

- Excessive segregation or bleeding.
- Inadequate retention of workability.
- Failure to set or stiffen sufficiently rapidly.
- Presetting cracks or later age cracks.
- Excessive cost of imported materials.
- Excessive variability.

Possible problem sources are:

- Unsatisfactory aggregates.
- Unsuitable mix design.
- Poor testing (including sampling, casting and curing of specimens).
- Cement or pozzolan quality.
- Unsuitable admixtures or admixture usage.

Data to request (having relevant past data available on arrival can often shorten the investigation by a day or more):

- Mix details.
- Aggregate gradings.
- Concrete test records (including times, temperatures, and specimen collection details).
- Cement test certificates if available.
- Cores and failed test specimens to inspect.

Of course it is desirable that records go back to a period before occurrence of the problem if possible. Where aggregate testing records seem inadequate, a rapid visit to the stockpiles is desirable before (further) change occurs. Segregation of coarse aggregates, silt content of the sand, and contamination with subgrade material by front-end loader are items to look for.

13.1 INADEQUATE STRENGTH

The typical steps taken by the author when called in to investigate problems may be of interest. The steps are:

1. Restore strength to a safe level so work can continue while investigating. Cement content adjustments should always overshoot when increasing and undershoot when reducing. Use 8 to 10 kg/MPa to adjust upwards, 4 kg/MPa to adjust downwards. If adjustment gives cement content over 500 kg use 500 kg plus 2 kg of fly ash for each 1 kg of cement not added; similarly with 0.5 kg silica fume, or 100 ml superplasticizer.
2. Start casting at least 4, perhaps 6, test specimens per sample. Test at 2, 3, 7, 28 and perhaps 56 days. Assume the gain in MPa will remain the same with the revised mix. In default of prior data, conservatively assume that strength will increase 33% from 2 to 3 days, another 33% from 3 to 7 days and 10 MPa from 7 to 28 days. Substitute actual figures as soon as available.

3. Draw cusum graphs of strength (at all available ages), density, concrete temperature, slump, 7 to 28-day gain (for example). If data is available, cusum graphs of sand silt content and/or specific surface should also be drawn on the same presentation. A cusum of average pair difference between pairs of specimens from the same sample will show whether there has been a deterioration in quality of testing (an average pair difference in excess of 1.5 MPa is an indication of poor testing quality). Such graphs will usually show when the problem started and what caused it.
4. Examine batching records (assuming a computer operated plant which records actual batch quantities) before and after the downturn for signs of cement shortfall or aggregate, especially sand, over-batching.
5. Calculate MSF using the formulas in Chapter 3. The MSF is a measure of the sandiness of the mix taking into account sand grading, sand %, cementitious material content, and entrained air. Calculate water content using formula in Chapter 3. Is actual water content really known? An MSF in excess of 30 represents over-sanding and high water requirement unless for flowing, superplasticized concrete.
6. Calculate strength according to one or more of formulas in Chapter 3. If this agrees with strength obtained/being investigated, then high water content is the explanation and the reason and cure are obvious (may be any combination of high MSF, silt in sand, concrete temperature, high slump).
7. If calculated water or strength does not agree with actual, re-check sand silt percentage and grading. Check concrete density as this will confirm water and/or air content and/or compaction of test specimens. **The water content is the major separating factor between alternative directions of investigation.** If water is the end cause, then the basic cause is likely to be in the area of dirty or finer sand, high sand content, high slump, or high concrete temperature. If water is not the cause, then the basic cause is likely to be in the area of poor testing (including sampling, compaction, curing, capping (if cylinders), defective or badly cleaned/assembled moulds (if cubes), centering, load rate, etc.), or of cement quality or quantity.

13.2 POOR WORKABILITY/PUMPABILITY

Generally the causes are an excess or deficiency of fine material, a gap in the grading, or an excess or deficiency of fluidity.

1. Does the concrete bleed? If so there is either a gap in the grading, a deficiency of fine material, or excessive fluidity. If the concrete pumps reasonably at the start, but will not re-start after a delay, this is often due to bleeding.
2. Using the author's MSF, the value of this must be at least 24 to 25 for pumping to be possible. The higher the desired fluidity, the higher the MSF value will have to be. However, values in excess of 32 will exhibit excessive friction unless superplasticized to high slump.
3. Draw a graph or produce a table of individual percentage retained on each standard sieve. Ideally all sieves below the largest will have similar percentages of around 7 to 10%. One size missing may not be fatal if those either side are normal. Any two consecutive sieves with a combined total retained of less than 7% would be a potential problem. More than 20% on a single sieve finer than 4.76 mm might also create a problem in pumping.
4. Is there at least 300 kg/m^3 of material passing the 0.15 mm sieve (including cement)? If not additional fines may be needed as either fine sand, crusher fines, fly ash, or cement.

5. If the (single) sand is so coarse that more than 55% (perhaps 50%) of it is necessary to provide an MSF of 25 there is likely to be a problem with bleeding, segregation and pumpability. Additional fines as in (4) above are necessary.
6. Air entrainment, fly ash, and silica fume (in increasing order of effectiveness) are effective suppressors of bleeding and so assist pumpability. The author has witnessed a huge foundation 4.5 m deep filled with concrete of more than 200 mm slump and containing 40 kg/m^3 of silica fume, which exhibited no bleeding whatever.
7. Although nothing to do with mix design, it should be borne in mind that it is pressure which causes a problem in pumping and faster pumping requires higher pressure. Also a delay caused by a gap in deliveries is an aggravating factor. Therefore, if pumping problems are being experienced, pumping more slowly and ensuring that one truck is not emptied before a replacement arrives may assist.

13.3 UNSATISFACTORY APPEARANCE

This may be due to inept placing, poor formwork or many other things which are beyond the scope of this book. However, it is also often due to bleeding, the remedies for which have been covered above. If bleeding happens at all, it often results in a flow of water up the face of formwork, leaving clearly visible signs. A slight formwork leak (just of water) can cause a surface flow over an area of more than a square metre and result in a large black stain, known as a hydration stain.

13.3.1 Presetting cracks

There are two kinds of presetting cracks with diametrically opposite causes: settlement cracks and evaporation cracks.

13.3.2 Settlement cracks

These result from settlement of the concrete due to loss of bleedwater. In settling, the concrete ‘breaks its back’ over anything resisting settlement in one location and not another, e.g. reinforcing bars, cast-in plumbing, sharp changes in depth of section. Measures to avoid bleeding have been dealt with above.

13.3.3 Evaporation cracks

These result from evaporation of water from the surface layer of concrete. If a concrete has very low bleeding e.g. silica fume concrete, it is susceptible to such cracks and measures must be taken to avoid evaporation e.g. use of an aliphatic alcohol evaporation retardant such as ‘Confilm’, a sheet material such as polythene, or a mist spray of water drifting across the surface.

13.3.4
Thermal stresses

Another frequent cause of early age cracking is thermal stress. This can be reduced by substituting pozzolanic material for cement in the mix design. However, action other than mix change may be needed, such as avoiding restraint to thermal shortening (in the case of long slabs), maintaining more uniform temperatures by insulating the exterior surfaces of large masses of concrete.

13.3.5
Adiabatic shrinkage

This should not be forgotten as a cause of early cracking in cement-rich mixes. This removes free water from the concrete by chemical combination. It can produce similar results to drying shrinkage but much more rapidly, and in spite of any measures taken to reduce or prevent evaporation.

Excessive variability

The first thing is to establish whether the variability is in the concrete or in the testing. Two places to look are the average pair difference in the 28-day results and the range of densities of test specimens from the same sample of concrete. The average pair difference should desirably be below 1.0 MPa and densities should not have an average range exceeding 50 kg/m^3. However, calculated densities may vary through inaccurate measurement of specimens rather than variable compaction or segregation and this would have no effect on strength variability.

A second place to look is at multivariable cusum graphs of strength and other variables. If slope change points in strength correlate with those of other variables, the cause will be clear. Direct plots of multiple variables will show whether individual high or low results have an explanation. If there is no explanation, and especially if 7 and 28-day results do not correlate, testing would be suspect.

Having established that the variability is actually in the concrete and not just the testing, batch quantity records should be available if batching is by computer-operated plant. It should not be overlooked that the correct quantities may be weighed out but may be insufficiently mixed to give uniformity. There have also been examples of short central mixing times (prior to further mixing by agitator trucks) which have not permitted time for all the metered admixture to enter the mixer. Similarly part of a particularly critical ingredient such as silica fume may 'hang up' in the batching skip from time to time and finish up in the next load.

13.4
CAUSES OF CRACKING IN CONCRETE SLABS

The causes of cracking in concrete are sufficiently well known to permit their automatic diagnosis in most cases. The author has in fact written an expert computer system for this purpose, which unfortunately used a now superseded shell and is therefore not currently operative. An expert system is a computer program which asks questions of a user in order to be able to diagnose the cause of the user's problem; the better ones are also able to explain why the particular question is being asked, on request by the user.

The first question to be asked is the age of the concrete at cracking. If the age was less than 10 hours, the crack would be classified as a presetting crack caused by either excessive evaporation from the surface or by restrained or differential bleeding settlement. If the age was more than 10 hours but less than 48 hours (and especially if the crack occurred in the early morning following pouring) the crack would probably be a thermal contraction crack. If the age exceeded two days (and was after termination of moist curing if any) it may be due to drying shrinkage.

To determine whether presetting cracks are caused by evaporation or settlement, questions are asked about the shape, size and location of the crack and about whether the concrete bled substantially or was subjected to drying winds and low humidity. Evaporation cracks may be quite wide on occasions but they are usually short and randomly orientated. However, they can sometimes be concentrated in an area of the slab which is more exposed to wind and can form parallel lines. In the latter case they may be more difficult to distinguish from settlement cracks occurring over a steel mesh, except that it would not be likely that evaporation cracks would be parallel to the direction of the mesh, or at the same spacing. As already noted the settlement cracks can occur over reinforcing bars, installed plumbing or the like. They can also occur at lines where the section deepens, such as dropped capitals for columns, haunched beams, or the edge of thickened areas of a slab.

A classic situation for thermal cracking exists when a thin concrete wall is poured between restraints. The restraints may be a heavy foundation beam with starter bars or substantial columns with projecting reinforcement. When a wall in such a situation is poured on a warm afternoon using a mix rich in a high heat generating cement (e.g. a white cement) the width of the crack to be anticipated on stripping next morning can be calculated if a maximum reading thermometer is inserted. Such cracks are often widest at the base, next to the restraining foundation beam, and taper away to nothing two or three metres up the wall.

A commonly encountered situation is where a crack runs parallel to, and often close to, a sawn control joint. It is easy to see that either the joint was not deep enough to be effective or, more likely, it was actually cut after the slab had already cracked, although perhaps before it had opened sufficiently to be noticeable.

Another useful distinguishing test is to place a straightedge at right angles across a crack. If the straightedge will rock, this indicates that the slab has deflected and therefore that the crack was probably caused by subgrade or formwork movement, or structural inadequacy in the case of suspended slabs.

Where cracks are three pointed, they are usually caused by a swelling or settlement resisting rock immediately below the junction of the cracks, e.g. a 'floater' in a soft subgrade subject to moisture movement.

In the case of suspected thermal cracks, it is useful also to check whether the concrete had a high cement content, making it likely to generate more heat, whether it was poured on a hot afternoon followed by a cold morning, and whether there was a delay in pouring, which could have allowed the concrete to heat up whilst kept waiting in the truck.

Surface crazing occurs when the surface layer shrinks relative to the body of concrete below it. This can be caused by allowing the surface to dry or cool quickly and is more likely when a high shrinkage surface layer, rich in cement paste and fine sand and of high w/c ratio, is present.

There is an almost universal tendency to use quality control personnel for trouble shooting of the above nature. This may be a reasonable use of any spare time, but it is important to ensure firstly that it does not disrupt the QC routine and secondly that such work is separately costed from QC. This is because the economic justification of QC should be clearly established as it otherwise tends to be regarded as a luxury item, first in line for cutting in hard times. Troubleshooting in general is not QC, indeed it may be the result of inadequate QC, and it is rarely cost saving or revenue generating. Many QC departments (not only in the concrete industry) have been axed or decimated through a failure to recognize this.

Summary and Conclusions

This second edition of the book has the same objectives as the first edition. These are:

- To promote a general understanding of the process by which low variability concrete can be produced to a given requirement, and especially of the misconceptions which hamper this process.
- To present the simplest possible process of effective mix design and quality control.
- To try to provide common ground, co-operation and mutual understanding between those who produce concrete and those who specify, supervise, test, or use it.

Since the first edition there has been a major change in the software program described, but it is a change in presentation, ease of use, and data capacity, rather than conception and philosophy. Its adoption and use by major concrete producers now attest the effectiveness of the program.

Others have shown that a packing density basis gives more precise answers to mix design problems over a wider range than the author's specific surface basis. They involve much more complex mathematics but computer programs take care of this. However, such methods require additional test data and do not necessarily result in a different solution to that using specific surface in most practical circumstances. Therefore the very simple specific surface concept has been retained in the author's system. It remains to be seen whether the packing density methods are as effective as specific surface in the day to day regulation of production concrete. An important factor may be whether the industry can be persuaded to institute regular sand flow testing, and if so which of the alternative systems is able to make best use of such data.

A program to produce and regulate mix tables (from 200 to 500 kg of cement) rather than individual mixes is now part of the Conad program. This may be a valuable adjunct to current adjustment methods but it is still only a step along the way to the author's preferred solution. The eventual solution is still seen as the abandonment of such tables in favour of a single formula from which all the hundreds of mixes in use can be generated by entering suitable parameter values. The formula would include terms for slump, temperature, transit time, air content, etc., and so would hopefully result in lower variability concrete. There is little difficulty in producing the formula, the challenge lies in the subsequent analysis of test data and the day-to-day adjustment of the formula, when no two mixes are identical. It is a challenge with which the author still hopes to be presented. The solution may be in the form of graphs of actual over-calculated strength plotted against each variable in the formula.

There is no comparable situation in quality control as opposed to mix design. The multigrade, multivariable, cusum technique has neither simpler nor more effective competitors. If and when the infinitely variable mix situation envisaged in the preceding paragraph arrives, it will still be possible to cusum actual minus calculated (= target) strength.

The development of truck mounted workability control devices has been reported. It is not very new but it is not yet in established large-scale use. It is possible that this development will finally close the last loophole in the quality of premixed concrete and result in significantly reduced variability. However, some plants are already able to operate at standard deviations as low as 1.8 MPa without such devices and some testing laboratories experience within sample SDs as high as 1.5 MPa. It would seem that the heat should certainly be put on testing laboratories to stay under 1.0 MPa if further reduction in overall variability is to be achieved.

Some old heresies remain and some are even getting worse. Progress remains slow in some parts of the world at getting rid of minimum cement content specifications and permitting rapid mix adjustment. Real retrogression is seen in the recent adoption of coefficient of variation rather than standard deviation by most if not all ACI committees. These anachronisms surely cannot long survive the examples of better control from other parts of the world.

One anachronism which does now seem to be dead is that of prohibiting the use of cement replacement materials such as fly ash and slag. The penny has finally dropped that these materials, properly used, actually improve concrete.

One of the author's pet hobby horses, cash penalties for minor strength (or several other) shortfalls, is not on anyone else's agenda and seems unlikely to be attainable in the near future. However, it remains the only fully satisfactory solution to the problem in the author's opinion, so the book continues to advocate it.

Durability has rightly become a hot topic but it is not intended to be one of the stronger points of this book. It was instructive to be asked to write a specification to ensure 120 year life for structures on a major free-way shortly before taking a holiday to see 2000 year old structures in Turkey. The durability of concrete is a different problem to the durability of reinforced concrete.

The attainable strength of concrete continues to rise until it is no longer relevant to the normal concrete industry. As much as 800 MPa has been achieved in semi-laboratory conditions involving pressure and heat and with no aggregate coarser than a fine sand (called RPC, 'reactive powder concrete'). The limit for anything which could be considered to be real concrete is probably around 200 MPa, with 150 MPa probably a more useful figure to bear in mind. Certainly 100 to 120 MPa must be regarded as relatively easily attained using silica fume and superplasticizers. It becomes important to ensure that test strengths are not unrealistic compared to the same concrete in the structure, especially in regard to water curing.

It is hoped that the CD-ROM which accompanies this book will enhance readers' comprehension of the Conad system. It was a matter of regret that colour illustrations were not possible in the first edition and would still be too expensive in this edition. This will hopefully be more than overcome by the full colour screencam videos on the CD. If you have sound on your computer, you will also be able to hear audio presentations. Should you have problems with the CD you should address your questions to the author rather than the publisher. He can be reached by Email as: *kenday@concreteadvice.com.au.* He would of course also be pleased to hear from readers who are not having problems with the CD but have suggestions to make or can provide feedback on entering their own data in the Conad Demo program provided. Initial testers of this program have been so enthusiastic that a similar but more advanced version, Conad Mini, has been produced for sale at modest cost (see the CD for details).

Readers may also like to visit our web site at *http://www.concreteadvice.com.au* where updates of various items will become available from time to time.

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Concrete Advice Pty Ltd

Concrete Advice is a small but internationally well known Australian company. It was founded in 1973 by Ken Day.

Our main product is the CONAD integrated concrete mix design and quality control computer program. Our principal, Ken Day, began the developments leading to the system in 1952 when he originated his own form of specific surface mix design and first started multi-variable control graphing.

The original business of Concrete Advice was as a general consultant and trouble-shooter in concrete technology. The area of activity included Singapore, Indonesia, Malaysia, the Philippines and Hong Kong, as well as Australia. In the early 1980s a subsidiary company (Concrete Advice (S) Pte Ltd) was formed in Singapore and employed over 30 people on concrete quality control on major projects in the area before being taken over by a major French laboratory organization (CEBTP). During this period the QC system was used on all projects but as a non-computerized spreadsheet and on behalf of consultants and contractors rather than concrete suppliers.

In the mid 1980s Ken put Concrete Advice on hold and spent two years with the Australian Government airfield construction branch. This gave him access to huge quantities of data, good computers and good computer expertize and enabled transformation of the system into a Lotus spreadsheet format (having most of the current features but in a much less powerful form).

In 1987/88 Ken resumed operation of Concrete Advice, providing QC services on major concrete projects in Melbourne using the new computer system.

The system went through a transformation into a compiled DOS system. The system has been in use by Pioneer in Singapore since the early 1990s and a first major licence was purchased by CSR Readymix in 1993. Subsequently it was further transformed into a Windows program written in Delphi and a licence was also purchased by Boral in 1997 and by Holderbank in 1999. This confirmed the program as the world leader in the field of major ready mix company technical control software.

Over 100 technical papers have been written on mix design and QC by Ken Day over a period of more than 40 years. This has culminated in, but will not end with, the publication the second edition of *Concrete Mix Design, Quality Control and Specification*.

About the CD-ROM

This book is supplemented by a free CD-ROM, which will operate under Windows 95, 98 and Windows NT, prepared by Concrete Advice Pty Ltd. The CD contains several screencams (see below) which enable the author to provide a guided tour of the system described in the book. You will see the system being used and, if your computer has a compatible sound card, you will hear the author describing it to you.

It also contains free programs which enable you to enter your own data and see whether they support the author's views on many things. Of particular importance are the method of predicting 28-day strength from 7-day results, the use of the specific surface/Mix Suitability Factor, and multigrade, multivariable cusum analysis graphs.

The free programs enable you to test the applicability of the technology, but they are not designed to be efficient working tools. A low-cost program, Conad Mini, is also demonstrated (although a working version is not provided). This is similar to the demonstration programs, but with features added to make it into a reasonable working tool for small scale use. It has also been used as a trial horse for features currently being added to the main system. These include a multi-language facility and a facility to nominate any series of up to 13 sieve sizes.

The screencams of the full Conad system enable the system to be explained and demonstrated far more fully than would be possible even with extensive still coloured photographs.

The publisher cannot enter into any discussion about the CD-ROM, but the author would be very pleased to receive feedback, positive or negative, on the contents of the CD or the book. It is possible that assistance can be provided in the case of difficulty. It is also likely that the author will make updated versions of the CD from time to time and these will be available direct from him at low cost (say US$10, of US$20 including airmail postage) to purchasers of the book. All queries concerning updates of the CD should be addressed direct to the author.

It will not be practicable to contact the author by telephone, but he may be reached by email at **kenday@concreteadvice.com.au.** Email contact will permit you to attach data files if you wish to present material for discussion. Alternatively, faxes may be sent to (613) 9723 7773.

Readers may also wish to visit the author's website: **http://www.concreteadvice.com.au.** Concrete Advice Pty Ltd is an Australian company founded by the author in 1973 as a vehicle for his services. Further details of the company are also given in the CD, with a brief cv of the author.

SCREENCAM PRESENTATIONS

Screencams are essentially videos of a person using a computer. They can be with or without sound commentary. They can be stopped, paused, and/or re-run at any time, but (unfortunately) they cannot be slowed down.

You do not need to have Lotus on your own computer to be able to view a screencam.

Real data have been used to demonstrate the system but, to avoid disclosing our clients' commercially sensitive information, an effort has been made to use very old data, to use mixed data from several sources, and to omit titles and other items which may identify the source of the data. This detracts somewhat from the presentation quality, but nevertheless provides a very good appreciation of the system. Using old data also enables the illustration of defects which may subsequently have been reduced or eliminated.

DEMO PROGRAMS

These programs are provided to enable readers to enter their own data in order to see whether many of the points made by the author are valid in their own circumstances.

QCDemo enables normal concrete test data to be entered. Readers can see for themselves whether change points are more clearly revealed through the cusum technique and whether strength change points correlate with changes in concrete temperature, slump and density. A particular feature has been made of examining the 7- to 28-day relationship. Predictions of 28-day strength are made both by the author's preferred technique of adding average gain and by the more usual assumption that a percentage gain will be experienced. The program analyses the predicition errors resulting from each method. The author would be most interested to receive reports from readers on their findings. Anyone who provides such a report will be informed of the results of the survey in due course.

MixDemo enables readers to enter mixes in a spreadsheet, having previously entered gradings and specific gravities of the aggregates to be included. This will provide values of Mix Suitability Factor (MSF) and predicted strength. Readers will be able to see whether their mixes have a consistent MSF for similar type mixes of differing strength and whether their mixes would be considered under- or over-sanded by the author's criteria. Strength predictions can vary substantially from obtained values, but should vary by a consistent amount over the strength range covered.

Readers are welcome to use the free programs, but even small producers may find it worthwhile to progress at least to the Conad Mini programs. These can be purchased from the author at low cost and are similar in operation and even easier to use than the Demo programs.

The CD contains screencams of Demo, Mini and full Conad programs.

MINI CONAD SCREENCAMS

Mini Conad evolved from **Conad Demo**. In producing the latter for free demonstration purposes it became obvious that this type of program could be of real continuing use if provided with a few additional features (mainly the ability to save and print results).

The Mix and QC parts of the programs provide much of the technical power of the full system. What they do not provide is the facility to establish and operate extensive databases of materials, customers, delivery and test details etc., the ability to generate and print automatic reports, or the ability for the user to extensively customize the system. The main numbers which actually drive Conad (strength, density, cement and water contents, slump and temperature) can be entered and analysed, but there is no facility to store descriptions or associated information such as customers, projects, testing officers, truck numbers, times etc.

The programs may be of interest and continued value to a small concrete producer or a site engineer. They may also serve as a trial horse for those who may later buy the full system once they are convinced of the value of the analysis tools.

Having originated Mini Conad, we then became aware of two further possibilities:

Mini Batch. A number of batch plant equipment manufacturers have expressed interest in the batch analysis capabilities of Conad. The difficulty is that batch data inclusion generally requires more assistance in establishment than other parts of Conad and it also interacts with other parts of the system, especially material control files. Concrete Advice personnel do, of course, provide such assistance in any major installation, but not everyone who is interested in monitoring their batching wants the full system. The batch plant manufacturers could themselves provide such assistance with minimal training. However, a great deal of training would be necessary for them to provide effective support for full Conad. Mini Conad is the ideal answer to this. It enables batch plant manufacturers to sell and support the batch analysis program. Some clients will later wish to progress to full Conad.

Mini Equivalent Age. Precasters and prestressers in particular have a considerable financial interest in establishing the exact age at which *in situ* concrete (perhaps even specific areas of a precast unit) will reach a certain strength. This is of particular importance as it is often earlier than indicated by test specimens nominally 'cured under the same conditions', but sometimes having as little as half the actual maturity.

Ease of use. A low purchase price is only part of the justification for Mini Conad. Of similar importance is its simplicity. You can hire the full Conad system on a monthly basis prior to purchase (or permanently), but there is still a cost in staff time of learning how to use it. If you are dubious that cusum analysis and specific surface will work for you, you can find out by using Mini Conad for a while. However, only a trial of full Conad can show you its very substantial administrative benefits and long term savings of staff time.

We ourselves started with the assumption that the financial justification for Conad would be in terms of cement saving through better control and more accurate strength targeting. However, it has become quite apparent from our clients' experience that such a saving, whilst important, is outweighed by improved staff efficiency, better record keeping and simply more knowledge and control of the situation.

FULL CONAD SCREENCAMS

The series starts with an SCM entitled 'Overview'. The main menu of the system is displayed. This is an almost blank screen showing a row of seven key words, each of which generates a pull-down menu. They are:

1. **File**
   Enables selection of the directories that the program uses. Also assists in archiving of data and combination of data from different sources.
2. **Batch**
   Recovery and processing of data from a computerized batching plant. Use of this data for various types of production control.
3. **Test result entry**
   Entry of normal field and laboratory test data, including automatic prediction of 28-day strength from 7 days and earlier.
4. **Analysis**
   Analysis, graphing and reporting of previously entered data, including incorporation of batch and material data.
5. **Mix control**
   Design and adjustment of mixes, either individually or as ranges of similar mixes. Currently includes assessment of relative performance of mixes (this may soon be transferred to the Analysis section).
6. **Material control**
   Entry and analysis of data on all materials, including aggregates, cements, pozzolans and admixtures.
7. **Early age**
   Recovery of temperature data from DT5 (single channel) and DT50 (5 channel) recorders and more detailed analysis of strength gain of both test specimens and *in situ* concrete.

A series of 14 separate screencams covers each of the above in more detail. Another screencam explains cusum graphing, including the difference between UK cusum analysis and the multigrade, multivariable cusum analysis used by Conad. A final screencam covers the transmission of data by email.

TIPS ON USING SCREENCAMS

The screen shows a conventional cursor being used, but this is not under control of the observer. Stop and pause buttons in the bottom right-hand corner are also not accessible to the observer.

Note the location of the control panel: this can be dragged and dropped in a different location if desired. It shows the progress of the screencam as an expanding green line and carries stop and pause buttons and a resume playing arrow. A second cursor, which is controlled by the observer, looks like a roll of film when over the displayed screen, but turns into a pointer over the control panel.

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