# Ground bearing concrete slabs

Specification, design, construction and behaviour

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Throughout the book the personal pronoun 'he', 'his', etc. are used when referring to 'the designer', 'the contractor', etc. for reasons of readability. Clearly, it is quite possible these hypothetical characters may be female in 'real-life' situations, so readers should consider these pronouns to be grammatically neuter in gender, rather than masculine.

# Preface

To date, ground bearing concrete slabs have not been perceived as a single type of structural element, rather, they have been dealt with as floors, external hardstandings and highway pavements. Information on each of these three types has been found within the domain of those types of element. This is the first time that the focus has been placed on the matters which are common to all categories of ground bearing slabs. Much of the methodology dealt with in this book is common to all three types of ground bearing slabs. There are some differences, for example external slabs need to incorporate provision for drainage, frost resistance and skidding resistance, whereas floors are invariably required to be flat, smooth, hard and horizontal. Also, the loading regime differs: typically, external hardstandings are required to sustain greater loads but lower pressures than floors. However, once these differences have been quantified, common approaches apply in terms of design, specification and construction.

This book also attempts to integrate the three crucial phases in the development of ground bearing concrete slabs: design, specification and construction. Much existing guidance majors on one of these phases. However, in common with all ground bearing structural elements, concrete slabs have the unusual property that they are subject to a pattern of time-dependent stresses which are initiated at the time of construction and which are dependent upon construction parameters. For example, temperature-dependent stresses, which develop throughout the life of the slab, keep forever as their datum the temperature profile locked into the slab at the precise time when the concrete sets and therefore can sustain strain. This means that concrete placed early in the day may have an entirely different stress pattern as compared with that placed later in the day and this difference will continue throughout the life of the slab. For this reason, the design and specification of a ground bearing slab must take into account construction conditions.

New design methods, which are based upon an ultimate limit state analytical approach, are being introduced into ground floor slab design and these methods are described in this book. These efficient design methods can lead to thinner slabs. More traditional elastic design methods lead to thicker slabs since they do not take advantage of the structural redundancy which is present in all ground bearing slabs. If thinner slabs are to be designed and installed, the designer needs to ensure that stresses caused by restraint to temperature-induced movement and moisture-induced movement are evaluated explicitly: in the past, they were accommodated in the reserve of strength which elastic design methods inevitably provided.

A growing proportion of ground bearing slabs is fully or partially supported on a grid of piles. This is because of a need to use sites where ground conditions are insufficient to support a pure ground bearing slab. This book presents a cost-effective design method for such slabs in which the reinforcing steel is in the form of steel fibres and in which the beams running over the piles are integrated within the slab. At the other extreme, designers are provided with a cost-effective steel-free design, whereby a pattern of closely spaced joints is introduced to control temperature-related stresses and moisture-related stresses. In today's environment-led world, this can have long-term benefits in permitting the re-use of a ground bearing slab as metal-free fill material.

The book is fully illustrated and this reflects the physical nature of the subject matter: the author felt unable to convey the essential content in words alone and so resorted to a mixture of sequential photographs (movies would have been best: maybe this will be possible in the second edition), charts and tables of numeric data.

## Details of the author

John Knapton was Professor of Structural Engineering at the University of Newcastle upon Tyne where he pursued research into industrial floors, heavy duty paving, pavements surfaced with pavers, highway pavements, port pavements and aircraft pavements. He now undertakes consulting work in these areas. Before taking the Newcastle Structures Chair, he ran his own conulting practice and was involved in the design and construction of many industrial floors. His early career included spells as the British Constructional Steelwork Association Research Associate at Newcastle, Research Engineer in the Cement and Concrete Association's Construction Research Department at Wexham Springs and Lecturer in Structural Engineering at Newcastle. He has represented the Institution of Civil Engineers on British Standards Institution (BSI) committees and has published over 100 papers since 1974. His work in developing the Ghanaian rural village of Ekumfi-Atakwa has led to his enstoolment as Nana Odapagyan Ekumfi I (Chief Eagle of Ekumfi I) by the Ghanaian government's House of Chiefs.

His previous publications include *Single pour industrial floors* and *In-situ concrete industrial hardstandings* by the same publisher. He has written the three editions of the British Ports Association heavy duty pavement design manual and has written standards for national and international bodies worldwide in the fields of paving and floors. Additionally, he continues to publish in journals and in conference proceedings. He is Chairman of the Small Element Pavement Technologists (SEPT) Group which holds title to the series of international conferences in concrete block paving.

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# Contents

# Introduction

The author has been concerned for many years that developments in three areas of construction — ground bearing concrete floors, industrial concrete hardstandings and concrete highway pavements — have taken place independently of each other. This is in spite of their obvious commonality in the areas of design, materials, geotechnical appreciation and construction. This book attempts to demolish the walls which have arisen between the three areas while, at the same time, introducing an integrated approach such that a practitioner in one of the three areas can transfer easily into the other two areas.

In case the reader is new to one or more of these three areas, case studies are presented illustrating design, construction, investigation and specification of each type of ground bearing slab. The case studies have been selected to represent those areas likely to be of most relevance to slab designers. For example, a high bay racking warehouse is illustrated and the overlaying of a heavily loaded industrial hardstanding is described. Because concrete is often the preferred construction material for industrial roads where loads may be infrequent but heavy, two such projects are explained.

Data presented are drawn from many international authorities, from industrial practice and from the author's own research. Where it is safe to do so, design short-cuts are presented involving simple charts and tables. However, there is always a design method available which embraces first principles. Both elastic and plastic methods of analysis are presented. The redundancy of a ground bearing slab leads to savings when plastic design is applied. However, the designer should recognise that the stress regime in a ground bearing slab is complex and is rarely calculated in its entirety. The conservatism in traditional elastic design has accommodated those stresses resulting from restraint to slab movement induced by moisture and temperature changes. If the designer really wants to squeeze every last drop out of the strength of his material, he must ensure that he has accounted for all of the factors that might introduce stress. For most ground bearing slabs, this may prove an unattainable goal.

Ground bearing slabs are similar to every other structural engineering element in that we know how strong to build them because we have learned that if we build them weaker they break and if we build them stronger they cost too much. Our mathematical design procedures constitute a valuable resource that allows us to test the sensitivity of design variables but, and this applies to all structural design, there is no substitute for experience.

Structural engineering has been defined as the art of designing structures to withstand loads that we cannot predict using materials whose properties we cannot measure by methods of analysis which we cannot prove and to do so in a manner which ensures that the general public is ignorant of our shortcomings. Ground bearing slab designers fit well within this definition: indeed, I would not allow a designer anywhere near a slab until he signed a statement confirming his understanding of the above.

# 1 Materials

## 1.1 Concrete

### *1.1.1 Introduction*

Concrete is a man made composite material comprising natural aggregate, water and cement to bind the aggregates together to form a hard composite material. For most applications, concrete is defined or specified by its 28 day characteristic crushing strength, cement content and free water/cement ratio. When determining the load bearing capacity of a ground bearing slab, the flexural strength of the specified concrete is needed. It is therefore customary to relate flexural strength to characteristic crushing (or cube) strength of concrete.

### *1.1.2 Specification*

To specify concrete it is necessary to select its characteristic strength together with any limits required on mix proportions, the requirements of fresh concrete and the type of materials that may be used. It is also important to have a good understanding of the methods of transport, placing and compaction procedures that are to be used when specifying a concrete mix as this can considerably change the characteristics and performance of a ground bearing slab. It is current practice to specify a 'designed mix' or special 'prescribed mix' and where applicable compliance testing procedures should be performed. BS 5328: 1981, *Methods of specifying concrete*,[1.1] defines the two types of mixes as follows.

> *A designed mix is specified by its required performance in terms of strength grade, subject to any special requirements for materials, minimum or maximum cement content, maximum free water/cement ratio and any other properties required. Strength testing forms an essential part of compliance.*
>
> *A prescribed mix is specified by its constituent materials and the properties or quantities of those constituents to produce a concrete with the required performance. The assessment of the mix proportions forms an essential part of the compliance requirements. Strength testing is not used to assess compliance.*
>
> *A prescribed mix should be specified only when there is reliable previous evidence or data established from trial mixes, that with the materials and workmanship available the concrete produced will have the required strength, durability and other characteristics. This type of mix*

*may be required to produce concrete having particular properties, e.g. to obtain a special finish.*

### 1.1.3 Types of cementitious material

Commonly specified cementitious materials are listed below with their appropriate British Standard references.

| | |
|---|---|
| BS 12: 1978 | *Ordinary and rapid hardening Portland cement* |
| BS 146: 1973: | Part 2 *Portland-blast furnace cement* |
| BS 1370: 1979 | *Low heat Portland cement* |
| BS 3892: 1982: | Part 1 *Pulverised-fuel ash for use as a cementitious component in structural concrete* |
| BS 4027: 1980 | *Sulphate-resisting Portland cement* |
| BS 6588: 1985 | *Portland pulverised fuel ash cement* |
| BS 6699: 1986 | *Ground granulated blast furnace slag (ggbs) for use with Portland cement* |

Ordinary or rapid hardening Portland cement is the most common cementitious material used but other cements or other combinations of Portland cement with ground granulated blast furnace slag (ggbs) and pulverised fuel ash (pfa) may be used provided that satisfactory data on their suitability, such as previous performance tests, are available. It has been suggested[1.2] that the replacement of up to 35% cement by pfa, or 50% ggbs, could be undertaken without an adverse effect on the wear resistance or flexural strength of the concrete slab, provided thorough curing for at least seven days has been performed. In the case of ggbs, the flexural strength exhibited by the concrete may be enhanced.

### 1.1.4 Aggregates

Aggregates should comply with BS 882: 1992, *Aggregates from natural sources for concrete*.[1.3] The following recommendations should be complied with if a wearing surface is to be achieved. The physical/mechanical properties defined are determined in accordance with BS 812 and stated in BS 882.

- 10% fines value should not be less than 100 kN.
- Aggregate impact value should be not less than 30%.
- Flakiness index should not exceed 35.
- Drying shrinkage of concrete should be less than 0·065%.

*1.1.4.1 Fine aggregate.* The fine aggregate is usually naturally occurring sand. Aggregate passing a 5 mm BS 410 test sieve is termed sand. BS 882: 1992: Section 5.2.2 gives the grading limits of sand used for the construction of concrete slabs. Sands within grading C or M should be used. Sand may be specified as either uncrushed, crushed or blended. Uncrushed sand results from the natural disintegration of rock, whereas crushed sand is the product of crushing processes of gravel or rock. Blended sand is a controlled mixture of two or more of the types described. Very coarse or very fine gradings, as well as gap gradings, should not be used as this can often lead to difficulties in finishing or poor durability of the surface. The sand should be free from soft materials, such as soft sandstone, limestone, coal and lignite and the use of unwashed crushed fines can seriously inhibit the quality of the slab.

*1.1.4.2 Coarse aggregate.* For most slabs 20 mm maximum size aggregate is suitable, but it may be more economic if thicker slabs are to be constructed to use aggregates up to 40 mm maximum size. Gradings as defined in BS 882[1.3] should be used. Soft sandstone or soft limestone should be avoided and crushed igneous or crushed flint gravels of angular shapes are preferred. Other aggregates than those stated in BS 812: Part 1 may be used provided satisfactory data on the exhibited properties of concrete made with them are available. Recent research[1.4] has found that the use of angular or crushed aggregate has resulted in an increase in flexural strength of up to 25% as compared with rounded or irregular aggregates.

### 1.1.5 Admixtures

It is common practice to specify admixtures to aid workability of the fresh concrete without loss of strength or durability. Admixtures are permitted in designed and prescribed mixes and, if specified, they should comply to BS 5075: Part 1[1.5] and Part 3[1.6] as appropriate. The use of an additive will normally be determined by the contractor according to his method of construction. If a construction procedure, such as laser screeding, requires a self-levelling concrete, no water should be added to the mix on site. In this case the use of a super-plasticizer is common. A super-plasticizer is defined in BS 5075: Part 3[1.6] as an 'admixture that, when added to a hydraulic binder concrete, imparts very high workability or allows a large decrease in water content for a given workability'. If two or more admixtures are to be used simultaneously, careful consideration should be taken to assess their interaction and to ensure their compatibility.

### 1.1.6 Concrete quality

Unlike most structural applications of concrete, slab design is based on the flexural strength of the concrete. A relationship between the 28 day characteristic compressive strength and the flexural strength of concrete is required. The 28 day characteristic compressive strength in N/mm$^2$ is defined as the grade of the concrete and is prefixed by the letter C. A relationship between the flexural strength and the 28 day compressive strength is given in Table 1.1,[1.7] although the author's own tests suggest that flexural strength values may be lower than those shown in Table 1.1.

In order to obtain concrete of a particular strength there are a number of mix design limits to comply with. Developments in the technology of cement manufacture in recent times have resulted in the achievement of higher strengths from mixes than those achieved previously. Table 1.2 shows mix design guidance[1.8] for various grades of concrete. To achieve a durable wearing surface, concrete with a minimum cement content of 325 kg/m$^3$ and water/cement ratio not exceeding 0·55 is commonly specified, although stricter limits may be applied if the slab is to be subject to heavy industrial use.

The workability of fresh concrete should be suitable for the conditions of handling and placing, so that after compaction and finishing the concrete surrounds all

*Table 1.1. Relationship between concrete grade, compressive strength, cylinder strength and flexural strength*

| Grade of concrete | Characteristic cube strength, $f_{cu}$: N/mm$^2$ | Characteristic cylinder strength, $f_{ck}$: N/mm$^2$ | Characteristic flexural strength, $f_{ctk}$: N/mm$^2$ |
|---|---|---|---|
| C25 | 25 | 20 | 1·5 |
| C30 | 30 | 25 | 1·8 |
| C35 | 35 | 28·5 | 1·95 |
| C37 | 37 | 30 | 2·0 |
| C40 | 40 | 32 | 2·1 |
| C45 | 45 | 35 | 2·3 |
| C50 | 50 | 40 | 2·5 |
| C55 | 55 | 45 | 2·7 |
| C60 | 60 | 50 | 2·9 |

*Table 1.2. Relationship between free water/cement ratio, cement content and lowest grade of concrete*

| | | | | | |
|---|---|---|---|---|---|
| Maximum free water/cement ratio | 0·65 | 0·60 | 0·55 | 0·50 | 0·45 |
| Minimum cement content: kg/m$^3$ | 275 | 300 | 325 | 350 | 400 |
| Grade of concrete: N/mm$^2$ | C30 | C35 | C40 | C45 | C50 |

reinforcement and completely fills its formwork. The workability of the concrete is normally determined by the contractor to suit his method of working and a slump of 50 mm is the usual maximum. If a slump of greater than 50 mm is allowed there is a tendency for the aggregate particles to segregate. Where the consistency of the mix is such that the concrete is unable to hold all its water, some is gradually displaced and rises to the surface. Separation of water from a mix in this manner is known as bleeding and can lead to dusting and poor wear resistance properties of the hardened concrete. The slump of a mix should therefore be carefully monitored on site during pouring.

The lower the water/cement ratio the higher is the strength of the hardened concrete (see Table 1.2), but this can lead to a loss in workability of the fresh concrete. Although modern equipment is capable of handling less workable mixes, the use of super-plasticizers (see Section 1.1.5) to increase workability is now common. Lowering the free water/cement ratio in a concrete mix also has the advantage of enabling finishing to begin earlier.

### 1.1.7 Micro-silica concrete

Micro-silica concrete may be specified for concrete slab construction where a tough marble-like wear resistant surface is needed. Tests commissioned by a manufacturer of micro-silica concrete produced the results shown in Table 1.3.

The use of micro-silica concrete results in an increase in flexural strength and thinner slabs may be possible so compensating for some of the additional cost of the micro-silica. Recommended design flexural strengths of micro-silica concrete are shown in Table 4.1.

*Table 1.3. Test results showing the increase in 28 day compressive and flexural strength obtained from samples of micro-silica concrete as opposed to conventional C40 concrete*

| | Conventional C40 concrete | Micro-silica concrete |
|---|---|---|
| Cement content: kg/$m^3$ | 330 | 300 |
| Water/cement ratio | 0·55 | 0·45 |
| 28 day compressive strength: N/$mm^2$ | 55 | 83 |
| 28 day flexural strength: N/$mm^2$ | 5·9 | 7·7 |

## 1.2 Subgrade and sub-base

### 1.2.1 Subgrade

Subgrade is the naturally occurring ground or imported fill at formation level. Homogeneity of the subgrade strength is particularly important and avoiding hard and soft spots is a priority in subgrade preparation. Any subgrade fill should be suitable material of such grading that it can be well compacted. Fill containing variable piece sizes often proves difficult to compact, giving rise to settlement and early failure of the slab. In-situ cement stabilisation may prove an economic means of improving a poor subgrade. On very good quality subgrades, such as firm sandy gravel, the sub-base layer may be omitted.

### 1.2.2 Sub-base

The sub-base is the foundation to the concrete slab. For most types of subgrade, a sub-base is essential. This layer usually consists of an inert, well graded granular material (see Section 1.2.5) or cement-treated material such as lean concrete or cement-bound granular material. In the case of wheel and rack loading (see Chapter 3), the sub-base assists in reducing the vertical stress transmitted to the subgrade. Where a distributed load is present, the slab achieves very little load spreading and the bearing capacity of the underlying subgrade may limit the maximum load applied to the floor.

### 1.2.3 Modulus of subgrade reaction

In considering the value of the stresses induced in a slab under loading, the influence of the subgrade is defined by its modulus of subgrade reaction ($K$). Modulus of subgrade reaction is defined as the pressure applied to the subgrade which will cause it to deflect by 1 mm. California Bearing Ratio (CBR) tests and plate bearing tests can be used to establish values (see Section 1.2.4). In many instances, subgrades are variable and results obtained from in-situ tests can often show scatter. CBR and plate bearing tests induce a shallow stress bulb which may not reflect the influence of deeper and possibly weaker material, which might become overstressed beneath a loaded slab. Assumed values of $K$ are shown in Table 1.4. Because the stresses in a concrete slab are insensitive to changes in the strength of its supporting material, the values in Table 1.4 may be assumed if no plate bearing test or CBR test results are available. The values in Table 1.5 are used in the design procedures set out in Chapter 4 (see Section 4.4).

*Table 1.4. Assumed modulus of subgrade reaction (K) for typical British soils*[1.7]

| | Typical soil description | Subgrade classification | Assumed $K$: N/mm$^3$ |
|---|---|---|---|
| Coarse grained soils | Gravels, sands, clayey or silty gravels/sands | Good | 0·054 |
| Fine grained soils | Gravely or sandy silts/clays, clays, silts | Poor<br>Very poor | 0·027<br>0·013 |

*Table 1.5 Enhanced value of K when a sub-base is used*

| $K$ value of subgrade alone: N/mm$^3$ | Enhanced value of $K$ when used in conjunction with: | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Granular sub-base of thickness: mm | | | | Cement-bound sub-base of thickness: mm | | | |
| | 150 | 200 | 250 | 300 | 100 | 150 | 200 | 250 |
| 0·014 | 0·018 | 0·022 | 0·027 | 0·033 | 0·045 | 0·063 | 0·081 | 0·106 |
| 0·027 | 0·034 | 0·038 | 0·044 | 0·051 | 0·075 | 0·104 | 0·137 | — |
| 0·054 | 0·059 | 0·065 | 0·072 | 0·081 | 0·125 | 0·175 | | |
| 0·082 | 0·089 | 0·096 | 0·105 | 0·114 | | | | |

*Table 1.6. The modified thickness of a concrete slab with a C20 lean concrete sub-base*

| Calculated thickness of slab: mm | Modified thickness of slab (mm) when used in conjunction with lean concrete sub-base of thickness: mm | | |
|---|---|---|---|
| | 100 | 130 | 150 |
| 250 | 190 | 180 | — |
| 275 | 215 | 200 | — |
| 300 | 235 | 225 | 210 |

Chandler and Neal[1.7] suggest that the sub-base can be taken into account by enhancing the effective modulus of subgrade reaction ($K$) as in Table 1.5.

When a cement stabilised sub-base is specified, the value of $K$ of the subgrade material is used to calculate the required thickness of the concrete slab. This calculated thickness is then apportioned between the structural slab thickness (the higher strength concrete) and the cement stabilised sub-base thickness. This relationship is shown in Table 1.6 when a C40 concrete is used for the slab and a C20 lean concrete is used for the sub-base.

### 1.2.4 Plate bearing and CBR testing

The plate bearing test procedure is to load the ground through a steel disk, usually mounted on the back of a vehicle, and to record load and corresponding deflexion. The value of $K$ is found by dividing the pressure exerted on the plate by the resulting vertical deflexion and is expressed in units of N/mm$^3$, MN/m$^3$ or kg/cm$^3$. $K$ is established by plate bearing tests with a load plate diameter of 750 mm. A modification is needed if a different plate diameter is used: for a 300 mm diameter plate $K$ is obtained by dividing the result by 2·3 and for a 160 mm diameter plate it is divided by 3·8. Alternatively, the CBR can be measured.

The CBR of a soil is determined by a penetration test that measures the force required to produce a given penetration in the material. This force is compared with the force required to produce the same penetration in a standard crushed limestone. The result is expressed in percentage terms as a ratio of the two penetration forces. Thus a material with a CBR value of 4% offers 4% of the resistance to penetration as compared with that which standard crushed limestone offers. The laboratory test should be carried out in accordance with BS 1377.[1.9] Different subgrade materials will have different CBR values and a conservative value is used for each category of soil. It is unusual for CBR to be measured directly since it can usually be determined with sufficient accuracy from Liquid Limit (LL) and Plasticity Index (PI) values. If the CBR is to be measured directly, it should be done so at the most adverse moisture content which the soil can reasonably be predicted to sustain. BS 1377 includes a 72 hour soaking procedure which will be appropriate in some design situations. Table 1.7[1.10] shows the relationship between CBR and modulus of subgrade reaction ($K$) for a number of common soil types.

The standard method of classifying soils in the US for engineering purposes is the Unified System and this system can also be used to assess CBR and $K$. The Unified System classifies soils on the basis of grain size and plasticity. The initial division of soils is based on their separation into coarse (sand and gravel), fine grained soils (clay) and highly organic soils (peat). The distinction between coarse and fine grained is determined by the amount of material retained on the No. 200 (75 micron) sieve. Coarse grained soils are subdivided into sands and gravels on the basis of the amount of material retained on the No. 4 (6 mm) sieve. Gravels and sands are then classed

*Table 1.7. Modulus of subgrade reaction values for a number of common subgrade and sub-base materials*

| | CBR: % | Modulus of subgrade reaction $K$: N/mm$^3$ |
|---|---|---|
| Humus soil or peat | < 2 | unacceptable |
| Recent embankment | 2 | 0·01–0·02 |
| Fine or slightly compacted sand | 3 | 0·015–0·03 |
| Well-compacted sand | 10–25 | 0·05–0·10 |
| Very well-compacted sand | 25–30 | 0·10–0·15 |
| Loam or clay (moist) | 3–15 | 0·03–0·06 |
| Loam or clay (dry | 30–40 | 0·08–0·10 |
| Clay with sand | 30–40 | 0·08–0·10 |
| Crushed stone with sand | 25–50 | 0·10–0·15 |
| Coarse crushed stone | 80–100 | 0·20–0·25 |
| Well-compacted crushed stone | 80–100 | 0·20–0·30 |

according to whether fine material is present. Fine grained soils are subdivided into two groups on the basis of LL and PI. When the soil has been classified in this way, it may be more convenient to use the $K$ values below rather than the ranges in Table 1.2. The US classification system subdivides soil types into different groupings according to the following system.

**GW** Well graded gravels and gravel — sand mixtures, little or no fines, $K > 0{\cdot}082\,\text{N/mm}^3$
**GP** Poorly graded gravels and gravel — sand mixtures, little or no fine, $K > 0{\cdot}082\,\text{N/mm}^3$
**GM** Silty gravels, gravels — sand mixtures, $K > 0{\cdot}082\,\text{N/mm}^3$
**GC** Clayey gravels, gravel — sand — silt mixtures, $K = 0{\cdot}054\,\text{N/mm}^3$
**SW** Well graded sands and gravely sands, little or no fines, $K = 0{\cdot}054\,\text{N/mm}^3$
**SP** Poorly graded sands and gravely sands, little or no fines, $K = 0{\cdot}054\,\text{N/mm}^3$
**SM** Silty sands, sand — silt mixtures, $K = 0{\cdot}054\,\text{N/mm}^3$
**SC** Clayey sands, sand — clay mixtures, $K = 0{\cdot}054\,\text{N/mm}^3$
**ML** Inorganic silts, very fine sands, rock flour, silty or fine sands, $K = 0{\cdot}027\,\text{N/mm}^3$
**CL** Inorganic clays of low to medium plasticity, gravely clays, silty clays, lean clays, $K = 0{\cdot}027\,\text{N/mm}^3$
**OL** Organic silts and organic silty clays of low plasticity, $K = 0{\cdot}027\,\text{N/mm}^3$
**MH** Inorganic silts, micaceous or diatomaceous fine sands or silts, plastic silts, $K = 0{\cdot}027\,\text{N/mm}^3$
**CH** Inorganic clays of medium to high plasticity, $K = 0{\cdot}014\,\text{N/mm}^3$
**PT** Peat, mud and other highly organic soils — unnacceptable

In the above list, G = Gravel, S = Sand, C = Clay, W = Well, P = Poor, M = Medium, H = High plasticity, L = Low plasticity, O = Organic, PT = Peat and $K$ values have been rationalised to the three values shown in Table 1.4 and a fourth higher value which is often appropriate when the enhanced support offered by a sub-base needs to be considered. These four values are used in the design methods presented in Chapter 4 (Section 4.4). The Unified System allows soils to be classified from any geographic location into categories to which engineering properties can be assigned, e.g. particle size distribution, LL and PI. The various groupings of this classification system have been devised to correlate in a general way with the engineering behaviour of soils.

### *1.2.5 Granular sub-base material*

The information provided in this section is based upon *Specification for Highway works, Series 800, Road Pavements — Unbound Materials.*[1.11] Granular sub-base materials are often specified according to Clause 803 ('Type 1') or Clause 804 ('Type 2') of this Department of Transport (DTp) specification. Clause 803 and Clause 804 materials should comprise an approved durable granular material such as gravel, hard clinker, crushed rock or well-burnt colliery shale, blended if necessary with sand or other fine screenings. Figure 5.61 shows Type 1 material spread and compacted on a road project.

Blast furnace slag for use as sub-base materials should comply with BS 1047.[1.12] Steel slag may be used provided it has been weathered and conforms to the requirements of BS 4987: Part 1.[1.13] Materials other than slag when placed within 500 mm of cement bound materials or concrete products should have a water soluble sulphate content not exceeding 1·9 g of sulphate (expressed as weight of $SO_3$ per litre) when tested in accordance to BS 1377: Part 3.

*1.2.5.1 DTp granular sub-base material type 1 (Clause 803 material).* Unless evidence suggests that Type 2 materials will be suitable, all granular sub-bases should be constructed from Type 1 materials which can comprise crushed rock, crushed slag, crushed concrete or well-burnt non-plastic shale. The material must lie within the grading envelope of Table 1.8 and not be gap graded. The sub-base material is transported, laid and compacted without drying out or segregation. The material must have a 10% fines value of 50 kN or more when tested to BS 812: Part 111[1.14] and an Aggregate Crushing Value of less than 30 when tested to BS 812: Part 111. Additionally, the material should have a CBR of 80% or more.

*1.2.5.2 DTp granular sub-base material type 2 (Clause 804 material).* Type 2 granular materials are made up of natural sands, gravels, crushed rock, crushed slag, crushed concrete or well-burnt non-plastic shale. The specification states that the material must lie within the grading envelope of Table 1.8 and not be gap graded. The material is transported, laid and compacted at a moisture content within the range 1% above and 2% below the optimum moisture content and without drying out or segregation. The material must have a 10% fines value of 50 kN or more when tested to BS 812: Part 111. Additionally, the material should have a CBR of 20% or more.

*1.2.5.3 Compaction of granular materials.* Unbound material up to 225 mm compacted thickness is spread and compacted in one layer so that after compaction the total thickness is as specified. The minimum compacted thickness should not be less than 110 mm. Where the layers of unbound material are of unequal thickness, the lowest layer should be the thickest layer. Compaction of unbound materials is carried out by a method shown in Table 1.9. The surface of any one layer of material on completion of

*Table 1.8. Grading requirements for granular materials*

| BS sieve size | Percentage by mass passing | |
|---|---|---|
| | Granular sub-base material Type 1 | Granular sub-base material Type 2 |
| 75 mm | 100 | 100 |
| 37·5 mm | 85–100 | 85–100 |
| 10 mm | 40–70 | 40–100 |
| 5 mm | 40–70 | 40–100 |
| 5 mm | 25–45 | 25–85 |
| 600 micron | 8–22 | 8–45 |
| 75 micron | 0–10 | 0–10 |

*Table 1.9. Compaction requirements for granular sub-base material Types 1 and 2*

| Type of compaction plant | Category | Number of passes for layers not exceeding the following compacted thicknesses: mm[a] | | |
|---|---|---|---|---|
| | | 110 | 150 | 225 |
| Smooth-wheeled roller (or vibratory roller operating without vibration) | Mass per metre width or roll: | | | |
| | over 2700 kg up to 5400 kg | 16 | Unsuitable | Unsuitable |
| | over 5400 kg | | | |
| Pneumatic-tyred roller[b] | Mass per wheel: | | | |
| | over 4000 kg up to 6000 kg | 12 | Unsuitable | Unsuitable |
| | over 6000 kg up to 8000 kg | 12 | Unsuitable | Unsuitable |
| | over 800 kg up to 12 000 kg | 10 | 16 | Unsuitable |
| | over 12 000 kg | 8 | 12 | Unsuitable |
| Vibratory roller[c] | Mass per metre width of vibrating roll: | | | |
| | over 700 kg up to 1300 kg | 16 | Unsuitable | Unsuitable |
| | over 1300 kg up to 1800 kg | 6 | 16 | Unsuitable |
| | over 1800 kg up to 2300 kg | 4 | 6 | 10 |
| | over 2300 kg up to 2900 kg | 3 | 5 | 9 |
| | over 2900 kg up to 3600 kg | 3 | 5 | 8 |
| | over 3600 kg up to 4300 kg | 2 | 4 | 7 |
| | over 4300 kg up to 5000 kg | 2 | 4 | 6 |
| | over 5000 kg | 2 | 3 | 5 |
| Vibrating-plate compactor[d] | Mass per square metre of base plate: | | | |
| | over 1400 kg/m$^2$ -1800 kg/m$^2$ | 8 | Unsuitable | Unsuitable |
| | over 1800 kg/m$^2$–2100 kg/m$^2$ | 5 | 8 | Unsuitable |
| | over 2100 kg/m$^2$ | 3 | 6 | 10 |
| Vibro-tamper[e] | Mass: | | | |
| | over 50 kg up to 65 kg | 4 | 8 | Unsuitable |
| | over 65 kg up to 75 kg | 3 | 6 | 10 |
| | over 75 kg | 2 | 4 | 8 |
| Power rammer[f] | Mass: | | | |
| | 100 kg up to 500 kg | 5 | 8 | Unsuitable |
| | over 500 kg | 5 | 8 | 12 |

[a] The number of passes is the number of times that each point on the surface of the layer being compacted is traversed by the item of compaction plant in its operating mode (or struck, in the case of power rammers). The compaction plant in Table 1.9 is categorised in terms of static mass. The mass per metre width of roll is the total mass on the roll divided by the total roll width. Where a smooth-wheeled roller has more than one axle, the category of the machine is determined on the basis of the axle giving the highest value of mass per metre width.

[b] For pneumatic-tyred rollers the mass per wheel is the total mass of the roller divided by the number of wheels. In assessing the number of passes of pneumatic-tyred rollers, the effective width is the sum of the widths of the individual wheel tracks together with the sum of the spacings between the wheel tracks providing that each spacing

*Table 1.9. continued*

does not exceed 230 mm. Where the spacings exceed 230 mm the effective width is taken as the sum of the widths of the individual wheel tracks only.

[c] Vibratory rollers are self-propelled or towed smooth-wheeled rollers having means of applying mechanical vibration to one or more rolls. The requirements for vibratory rollers are based on the use of the lowest gear on a self-propelled machine with mechanical transmission and a speed of 1·5–2·5 km/h for a towed machine. Vibratory rollers operating without vibration are classified as smooth-wheeled rollers.

[d] Vibrating-plate compactors are machines having a base plate to which is attached a source of vibration consisting of one or two eccentrically-weighed shafts. They normally travel at speeds of less than 1 km/h.

[e] Vibro-tampers are machines in which an engine driven reciprocating mechanism acts on a spring system, through which oscillations are set up in a base-plate.

[f] Power Rammers are machines which are actuated by explosions in an internal combustion cylinder; each explosion being controlled manually by the operator. One pass of a power rammer is considered to have been made when the compacting shoe has made one strike on the area in question.

compaction and immediately before overlaying should be well closed, free from movement under compaction plant and from ridges, cracks, loose material, pot holes, ruts or other defects. All loose, segregated or otherwise defective areas should be removed to the full thickness of the layer, and new material laid and compacted.

### *1.2.6 Cement stabilised sub-bases*

The information provided in this section is based upon *Specification for Highway works, Series 1000, Road Pavements — Concrete and Cement Bound Materials.*[1.15]

*1.2.6.1 Constituents.* The cement in cement-bound materials must comply with the materials in Table 1.10 or the combinations in Table 1.11.

The maximum proportions of ground granulated blast furnace slag (ggbs) with Portland cement should not be greater than 65% of the total cement content for cement-bound materials. The water content should be the minimum amount required to provide suitable workability to give full compaction and the required density.

*1.2.6.2 Cement Bound Material Category 1 (CBM1).* CBM1 is typically made from a material that has a grading finer than the limits in Table 1.12.

*1.2.6.3 Cement Bound Material Category 2 (CBM2).* CBM2 is typically made from gravel-sand, a washed or processed granular material, crushed rock, all-in aggregate, blast furnace slag or any combination of these. The constituents of the material must fall within the grading limits shown in Table 1.12. The material must have a 10% fines value of 50 kN or more when tested in accordance with BS 812: Part 111[1.14] with samples in a soaked condition.

*1.2.6.4 Cement Bound Material Category 3 (CBM3).* CBM3 is made from natural aggregate material complying with BS 882.[1.3]

*1.2.6.5 Cement Bound Material Category 4 (CBM4).* CBM4 is made from natural aggregate material complying with BS 882.

*Table 1.10. Cementitious material specifications*

| Cement | Complying with |
|---|---|
| Portland cement (PC) | BS 12 |
| Portland blast furnace cement (PBC) | BS 146 |
| Portland pulverised-fuel ash (PFA) cement | BS 6588 |
| Pozzolanic cement (Grades C20 or below) | BS 6610 |

*Table 1.11. Cementitious material combination specifications*

| Combination | Complying with |
|---|---|
| Portland cement with ground granulated blast furnace slag | BS 12 |
| Portland cement with pulverised-fuel ash (PFA) for use as a cementious component | BS 3892: Part 1 |
| Portland cement with micro-silica having a current BBA certificate | BS 12 |

*Table 1.12. Grading of aggregate materials used in the four categories of cement bound materials.*

| BS sieve size | Percentage by mass passing nominal maximum size: | | | |
|---|---|---|---|---|
| | — | — | 40 mm | 20 mm |
| | CBM1 | CBM2 | CBM3 & CBM4 | |
| 50 mm | 100 | 100 | 100 | — |
| 37·5 mm | 95 | 95–100 | 95–100 | 100 |
| 20 mm | 45 | 45–100 | 95-100 | 100 |
| 10 mm | 35 | 35–100 | 45–80 | 95–100 |
| 5 mm | 25 | 25–100 | 35–50 | 35–55 |
| 2·36 mm | N/A | 15–90 | N/A | N/A |
| 600 micron | 8 | 8–65 | 8–30 | 10–35 |
| 300 micron | 5 | 5–40 | 0–8 | 0–8 |
| 75 micron | 0 | 0–10 | 0–5 | 0–5 |

If blast furnace slag aggregate is to be used, it must comply with BS 1047: 1983.[1.15] Cement for use in all cement-bound material and aggregate for use in CBM3 and CBM4 should be kept dry and used in the order in which it is delivered to the site. Different types of cementitious material must be stored separately.

*1.2.6.6 Drylean concrete.* Drylean concrete is a lean concrete with a low water content. The maximum aggregate to cement ratio is 15 to 1. The water content should be between 5 and 7% by weight of dry materials, the final value being selected to give the maximum dry density. The material should be rolled to give the maximum possible density.

*Table 1.13. Batching and mixing of cement-bound materials*

| | Site requirements | | | | Specimen requirements | |
|---|---|---|---|---|---|---|
| Category | Mixing Plant | Methods of batching | Moisture content | Minimum compaction | Minimum 7 day cube compressive strength: N/mm$^2$ | |
| | | | | | Average | Individual |
| **CBM1** | Mix-in-place or mix-in-plant | Volume or mass | To suit requirements for strength, surface level, regularity and finish | 95% of cube density | 4·5 | 2·5 |
| **CBM2** | " | " | " | " | 7·0 | 4·5 |
| **CBM3** | Mix-in-plant | Mass | " | " | 10·0 | 6·5 |
| **CMB4** | " | " | " | " | 15·0 | 10·0 |
| **Drylean Concrete** | " | " | Between 5% and 7% of dry weight | Maximum possible | 15·0 (Maximum) (No single cube below 12) | |

*1.2.6.7 Batching and mixing.* Cement bound materials should be made and constructed as in Table 1.13.

Batching and mixing should be carried out in the appropriate manner described in Table 1.13. Where the mix-in-plant method is used and materials are batched by mass, materials should be batched and mixed in compliance with BS 5328: Part 3.[1.1]

*1.2.6.8 Transporting.* Plant-mixed cement-bound material when mixed should be removed from the mixer immediately and transported directly to the point in consideration.

*1.2.6.9 Laying.* All cement-bound material should be placed and spread evenly in such a manner as to prevent segregation and drying. Spreading the material is undertaken concurrently with placing or without delay. Base cement-bound material is often spread using a paving machine or a spreader box and operated with a mechanism that levels off the cement-bound material to an even depth. Cement-bound material is always spread in one layer so that after compaction the total thickness is as specified. Compaction is carried out immediately or within two hours of the addition of the cement. The surface of any layer of cement-bound material on completion of compaction and immediately before overlaying should be well closed, free from movement under compaction plant and from ridges, cracks, loose material, pot holes, ruts or other defects.

*1.2.6.10 Compaction.* Compaction should be carried out immediately after the cement-bound material has been spread and in such a manner to prevent segregation. Compaction must be completed within two hours of the addition of the cement. The surface of any one layer of cement-bound material on completion of compaction and before overlaying should be well closed, free from movement, compaction plant and from ridges, cracks, loose material, pot holes, ruts or other defects.

*1.2.6.11 Curing.* Immediately on completion of compaction, the surface of the cement-bound sub-base is cured for a minimum period of seven days.

### *1.2.7 Settlement*

A site investigation can provide the necessary information to enable an estimate of long-term settlement to be made. Where slabs are supported on subgrade such as organic soils, heavy clays and loose sands, or where land has been reclaimed, anticipated long-term settlements may be significant. Plate bearing tests, as described in Section 1.2.4, enable long-term settlements to be predicted. Soil stabilisation, drainage or compaction, or the use of piled foundations may be used to reduce or eliminate settlement.

## 1.3 Slip membranes

A slip membrane consists of polyethylene sheeting laid beneath the slab with overlaps of at least 200 mm and is usually placed immediately prior to concrete pouring. Wrinkles and folds should be completely removed as they can result in weakening of the slab in later life as they may form crack inducers. It is advisable to anchor the polyethylene sheeting with small heaps of concrete, especially on the overlaps. A minimum of 125 micron (500 gauge) polyethelene sheeting should be used and 250 micron (1000 gauge) or 300 micron (1200 gauge) sheets are common.

A slip membrane is used to reduce friction between a concrete slab and its sub-base. The coefficient of friction with the use of a membrane is in the region of 0·2, compared with values of up to 0·7 when the concrete slab and sub-base are in direct contact. Prevention of loss of moisture and fines from the fresh concrete into the sub-base does occur, although a slip membrane is not intended or required to serve as a damp-proof membrane. When damp-proofing is to be provided, thicker sheets or more elaborate measures may be required.

If an impermeable membrane is used then drying can take place only from the upper surface of the slab which may result in curling. The perforation of the slip membrane, or complete omission with the use of a blinding material, may therefore need to be considered, although this would result in an increased loss of water and fines from the underside of the slab.

## 1.4 Polypropylene fibres

### *1.4.1 Introduction*

Polypropylene fibres for concrete can be in fibrillated or monofilament form manufactured in a continuous process by extrusion of polypropylene homopolymer resin. They are usually coated to improve wetting and dispersibility within the cement paste and to increase the extent of contact and bond between the fibres and the concrete matrix in the hardened state. Polypropylene fibres are not a substitute for conventional structural reinforcement or normal good curing procedures, but they may be used as an alternative to non-structural mesh (see Section 1.6) for crack control purposes acting as a secondary reinforcement. The design of polypropylene fibre-based concrete floors proceeds as for unreinforced slabs. The main purpose of polypropylene fibres is to provide crack control by distributing and absorbing tensile stresses which may occur as a result of shrinkage and temperature movements, particularly in the early life of the slab when the concrete has yet to reach sufficient tensile strength. They do not eliminate cracks and are not considered to contribute to the strength of the slab although they do enhance impact resistance and abrasion resistance.

### 1.4.2 Monofilament fibres

Monofilament fibres are manufactured from extruded sheet/film material that is subject to molecular alignment, coated and cut to the appropriate length. This type of fibre is usually much finer than the fibrillated fibre (see Section 1.4.3) and the properties of a concrete resulting from the addition of monofilament fibres depend on the large number of fibres present. A smoother surface finish may be achieved from the use of the monofilament fibre as opposed to the fibrillated type. Monofilaments do not provide any mechanical bond to the cement paste, but rely on their greater number per metre cube of concrete and their chemical bond in order to achieve their proven qualities in the plastic and hardened state.

### 1.4.3 Fibrillated fibres

Fibrillated fibres are manufactured from extruded sheet/film material that is subject to molecular alignment, fibrillated, coated and cut to the appropriate length. Clustering of fibres is overcome by the mixing of aggregates in the concrete mix. Basic properties of fibrillated fibre[1.17] are:

- density = 900 kg/m$^3$
- tensile strength range = 560–770 N/mm$^2$
- elastic modulus = 3·5 kN/mm$^2$
- melt point = 160–170 °C.

Fibrillated fibres have a rough surface texture which gives each fibre a high degree of mechanical bond to the concrete. Monofilament fibres achieve enhanced plastic shrinkage control and trowel workability, while fibrillated fibres impart a higher degree of abrasion resistance to the resulting concrete.

### 1.4.4 Addition and mixing

The addition of polypropylene fibres is at a recommended dosage of approximately 0·90 kg/m$^3$ (0·1% by volume or 1 l/m$^3$). They are compatible with all cementitious products and admixtures, and generally require no change in mix design or water/ cement ratio. The fibres may be added at either a conventional batching/mixing plant or by hand to the ready mix truck on site. An even distribution throughout the concrete can be achieved in a 6 $m^3$ truck mixer in five minutes at full mixing speed.

### 1.4.5 Placing, curing and finishing

Concrete mixes containing polypropylene fibres can be transported by normal methods and flow easily from the hopper outlet. No special precautions are necessary when pouring and fibre-dosed concrete will flow around an obstruction, such as reinforcement, in the same manner as a conventional concrete mix of similar proportions. Conventional means of tamping or vibration to provide the necessary compaction can be used.

Curing procedures similar to those specified for conventional concrete should be strictly undertaken. If steam curing at a temperature in excess of 140 °C is to be used, polypropylene fibres should not be used. The fibres do not affect the hydration rate or stiffening time of the concrete.

Placed fibre-dosed mixes may be floated and trowelled using all normal hand or power tools. Occasional fibres protruding through the surface will quickly wear away. This is more common with fibrillated fibres than with monofilament fibres. Workmanship should comply with the relevant requirements of BS 8000 Part 2: Sections 2.1 and 2.2.[1.18] Anti-wear products and other toppings may be used.

### 1.4.6 Controlling plastic shrinkage cracks in concrete

Plastic cracking may occur in the plastic concrete as a result of drying shrinkage. Plastic cracks are formed within the first 24 hours after the concrete has been placed when the evaporation rate is high and the surface of the concrete dries out rapidly. Plastic shrinkage cracks generally pass through the entire slab and form weakness, permanently lowering the integrity of the slab before the concrete has had the opportunity to gain its design strength. Plastic cracks may occur through the whole depth of a slab and cannot be remedied by surface treatment.

Polypropylene fibres inhibit plastic cracking by holding water at or near the surface of the concrete, delaying evaporation and increasing cement hydration. Therefore, bleeding is inhibited. As concrete hardens and shrinks, micro-cracks develop. When the micro-cracks intersect a fibre strand, they are blocked and prevented from developing into macro-cracks and hence plastic cracking. This reduction of micro-cracks in the plastic state enables the concrete to better develop its optimum integrity. A number of research programmes studying the effect on plastic shrinkage cracking of concrete with the use of polypropylene fibres are referenced in Chapter 5.14.

### 1.4.7 Effect on workability

Polypropylene fibres act mechanically. They impart a cohesive effect which is due to surface tension and breaks down under vibration and compaction. The slump of a fibre-dosed concrete will be lower as a result of the thixotropic effect caused by the fibres but the mobility or placeability of the concrete is generally unaffected. Water should not be added to compensate for this thixotropic effect. Vebe and other compaction factor test results are not significantly affected by the addition of polypropylene fibres. The improved cohesiveness also proves to be beneficial in pumped concrete owing to the reduction in rebound when placing.

### 1.4.8 Polypropylene fibre reinforced concrete

The use of polypropylene fibres in the construction of a ground floor concrete slab is not considered to contribute to the strength of the slab. The addition of polypropylene fibres at the usual recommended amount (0·9 kg/m$^3$) will not significantly affect the ultimate compressive, tensile or flexural strength of the concrete matrix. Before ultimate stress is reached, the performance of a fibre-enhanced concrete is improved in a number of ways. These improvements are due to concrete being an inherently variable material with a wide range of stress concentrations and the addition of the fibres favourably reduces this variability. If a fibre is aligned across a crack there is a small increase in stress required for crack propagation to occur.

### 1.4.9 Strength characteristics

Tests from a manufacturer of polypropylene fibres[1.17] revealed the strength characteristics shown in Table 1.14.

Compressive strength tests conducted in accordance with BS 1881 indicated that the fibres, when used at the recommended dosage rate of 0·90 kg/m$^3$, slightly increase the early strength gain of concrete. The fibres have no significant effect on the 28 day compressive strength of concrete cubes nor do they have any substantial effect on the flexural strength of concrete.

*Table 1.14. Test results comparing the strength of polypropylene fibre-dosed concrete and conventional plain concrete*

| | Strength of fibre-reinforced concrete: N/mm$^2$ | Strength of unreinforced concrete: N/mm$^2$ |
|---|---|---|
| Compressive strength (equivalent cube method) | | |
| 1 day | 16·5 | 16·0 |
| 3 days | 28·5 | 24·5 |
| 7 days | 34·0 | 35·0 |
| 28 days | 43·5 | 39·5 |
| Cube compressive strength | | |
| 1 day | 16·0 | 14·5 |
| 3 days | 28·0 | 27·5 |
| 7 days | 34·0 | 36·0 |
| 28 days | 48·5 | 44·5 |
| Flexural strength | | |
| 1 day | 2·3 | 2·1 |
| 3 days | 4·0 | 3·7 |
| 7 days | 4·2 | 4·8 |
| 28 days | 4·6 | 6·2 |

### 1.4.10 Shatter resistance

Typically, when concrete test cylinders fail in compression at ultimate load, there is an initial crack. Continued loading with plain concrete specimens causes the cylinders to fragment and fall apart. With polypropylene fibres present the concrete specimen holds together after maximum load without falling apart or shattering. Tests[1.17] show the ability of polypropylene fibre-dosed concrete to remain intact and not to shatter after more than 10% additional compression as compared to plain concrete which shattered completely shortly after the first crack developed. This characteristic of polypropylene fibre-dosed concrete is important in applications where there are impact or seismic concerns.

### 1.4.11 Impact resistance

The addition of polypropylene fibres increases energy absorption/impact of a concrete slab. The fibres bridge the cracks that develop and thereby inhibit further crack growth. Therefore, whereas the ultimate tensile strength of fibre-dosed concrete does not increase appreciably, the tensile strain at rupture does. Where steel reinforcement is used in concrete, the addition of fibres enhances the bond between the concrete and the reinforcing bars by inhibiting cracking on the concrete under bearing stress. Chapter 5 shows the results from the study on impact resistance of fibre-dosed concrete.

### 1.4.12 Abrasion resistance

The introduction of polypropylene fibres into concrete results in a greater surface abrasion resistance compared to that of conventional concrete. Tests[1.17] have shown that the presence of fibres in a concrete mix reduces the amount of bleeding and assists in holding aggregate near the surface of fresh concrete so resulting in better surface integrity. Chapter 5 shows the results of abrasion tests of fibrous concrete.

### 1.4.13 Permeability

Permeability is defined as the ease with which a fluid can flow through a solid. The addition of polypropylene fibres to a concrete slab reduces its water permeability owing to fibres interfering with the normal bleed channels and capillaries that are initially formed in the plastic state. With the reduction of cracking of the concrete resulting from the inclusion of fibres, the penetration of water has been laboratory tested[1.17] to reduce by at least 50%. Figure 1.1 shows how increasing polypropylene fibre dosage decreases the permeability of the concrete.

### 1.4.14 Resistance to freeze/thaw

Fibre-dosed concrete has a significantly enhanced resistance to frost attack. It is now common for polypropylene fibres to be specified in external slabs to provide frost resistance instead of the more traditional air entrainment.

### 1.4.15 Chemical resistance

The presence of fibres does not alter the chemical resistance of concrete. Polypropylene is an inert and alkali resistant material and will not degrade in concrete.

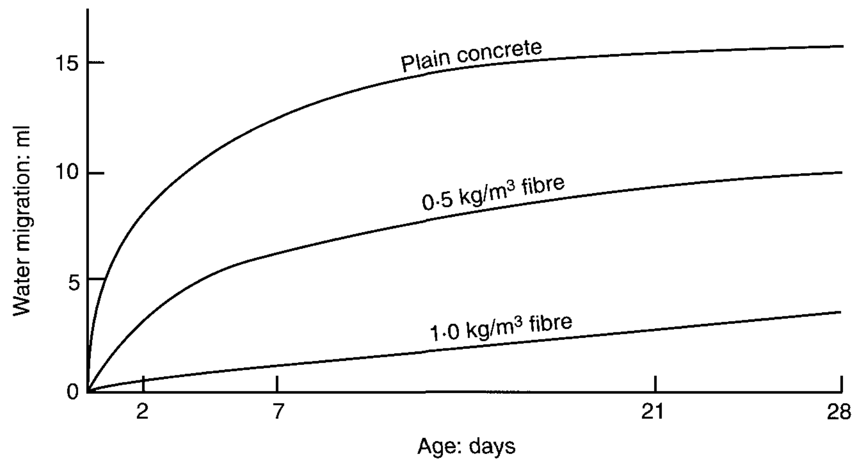



*Fig. 1.1. Effect of polypropylene fibres on concrete permeability*

### *1.4.16 Reducing corrosion of steel reinforcement*

The addition of polypropylene fibres into a concrete slab significantly increases the protection of the steel reinforcement, within the slab, against corrosion. The reduction in permeability of the concrete is an attribute of prime importance with regard to the protection against corrosion. The high toughness index, which is the ability to sustain a load after initial crack, is also important in reducing the corrosion of reinforcing steel. This is due to the reduction of spalling of the concrete and the continued bond to the steel reinforcement.

## 1.5 Steel wire fibres

### *1.5.1 Introduction*

Steel fibres may be used in place of mesh reinforcement. The stresses occurring in a concrete slab are complex and, depending on the type of load, tensile stresses can occur at the top and at the bottom of the slab. There are, in addition, stresses that are difficult to quantify, arising from a number of causes such as sharp turns from fork lift trucks, shrinkage and thermal effects, and impact loads. The addition of steel wire fibres to a concrete slab results in a homogenously reinforced slab achieving a considerable increase in flexural strength and enhanced resistance to shock and fatigue.

### *1.5.2 Concrete composition and quality*

In order to obtain steel fibre reinforced concrete that is easy to pump and to work, with minimum shrinkage, a steel wire fibre manufacturer[1.10] specifies the following.

- Quantity of cement (commonly Ordinary Portland Cement) should be between 320 and 350 kg/m$^3$.
- 750–850 kg/m$^3$ good quality zero to 4 mm well graded sharp sand should be used.
- Use a continuous aggregate grading with a maximum size of 28 mm for rounded gravel and 32 mm for crushed stone. Limit the fraction larger than 14 mm to 15–20%.

- Characteristic compressive strength of at least 25 N/mm$^2$ should be used.
- Water/cement ratio should be about 0·50, and should not exceed 0·55.
- The use of a super-plasticizer is permitted to obtain the necessary workability.
- Admixtures of chloride or chloride containing concrete additives are not permitted.

### 1.5.3 Addition and mixing

The recommended dosage rate of steel fibres is usually between 20 and 40 kg/m$^3$. The greater the dosage rate the greater is the flexural strength of the slab for a particular grade of concrete. Fibres can be added at the mixing plant or on site directly into the mixing truck. At the mixing plant the steel fibres are usually added into the mixer at the same time as the aggregates. On site the concrete must first achieve the correct workability by the addition of a super-plasticizer before the fibres are added (see Section 1.1.5). The fibres should then be added at the manufacturer's specified rate resulting in a uniform distribution. For example, one manufacturer[1.10] recommends addition at a rate of two 30 kg bags per minute with the truck rotating at full mixing speed and mixing continuing for a further two minutes after the addition of the full dose. Visual inspection during pouring is necessary to check fibre distribution is satisfactory. All fibre bundles must separate into individual fibres, otherwise mixing is insufficient.

### 1.5.4 Placing, curing and finishing

When placing a slab the concrete should be compacted as effectively as possible. Conventional means of tamping or vibration can be used. The usual techniques of floating and trowelling can be applied for finishing. After compaction and levelling, anti-wear products and cement are often spread on top of the concrete surface. Brushing of the concrete surface can be undertaken. In the case of an internal floor slab, the fresh concrete should be protected during curing periods by closing all openings to the building with the exception of ventilation holes. Immediately after finishing, a curing compound should be applied to combat rapid drying, forming an unbroken film on the surface of the concrete. A second curing layer may be applied if environmental conditions might cause rapid drying. Thin slabs, with thicknesses of 120 mm or less, should be provided with a double curing layer to prevent the risk of curling at the edges resulting from overfast drying. It should be noted, however, that if a wear-resistant topping is to be included the curing compound must not be applied, in which case the concrete can be kept moist by wet spraying or by overlaying plastic sheeting. Plastic sheets must not be applied if there is a risk that the temperature will become too high and result in the concrete setting too quickly.

### 1.5.5 Types of steel fibre available

The most commonly used steel fibre is the 60 mm long hooked or crimped fibre. Hooked and crimped fibres are usually glued together (collated) into 'clips' with a special water-soluble glue to form fibre plates that readily disperse in the concrete mixer. The hooks or crimping help to ensure optimum fibre anchorage (or adhesion) in

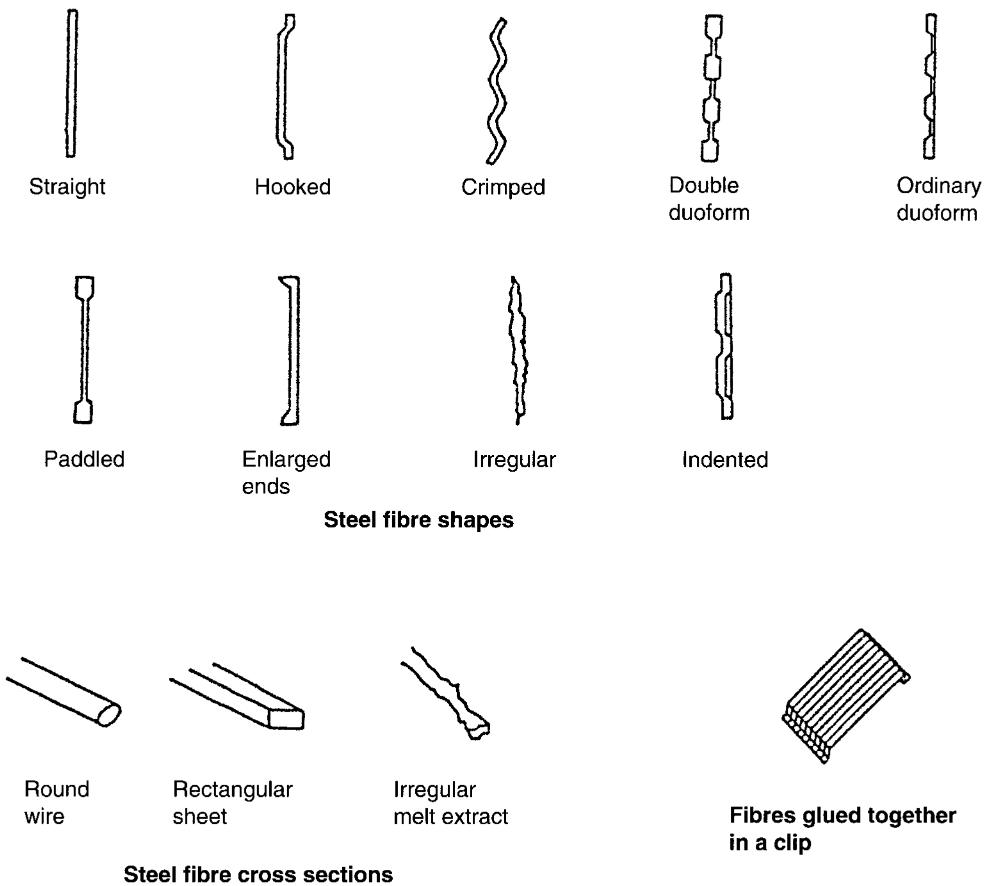



*Fig. 1.2. Different steel fibre types*

the hardened concrete. Enhanced adhesion can be achieved by either anchorage points at the ends of fibres (e.g. a pedal or hook) or in the case of a crimped fibre by adhesion along the whole length of the fibre. It is usual to consider only fibres with enhanced adhesion for reinforcement in concrete slabs. Various types of steel fibres are illustrated in Fig. 1.2.

Breaking or premature deformation of the fibres is prevented by the very high tensile strength of the drawn wire (usually greater than 1100 N/mm$^2$). The aspect ratio, which is the fibre length to fibre diameter ratio, is also an important factor in fibre specification with common values of 60 and 75.

### 1.5.6 Controlling cracking

Steel wire fibres effectively limit the extension of micro-cracks always present in concrete (see Fig. 1.3). In concrete without fibres, tension cannot be transmitted across the crack, that is, once the tensile capacity of the plain concrete is exceeded, the micro-crack will extend rapidly resulting in brittle failure. The action of the steel

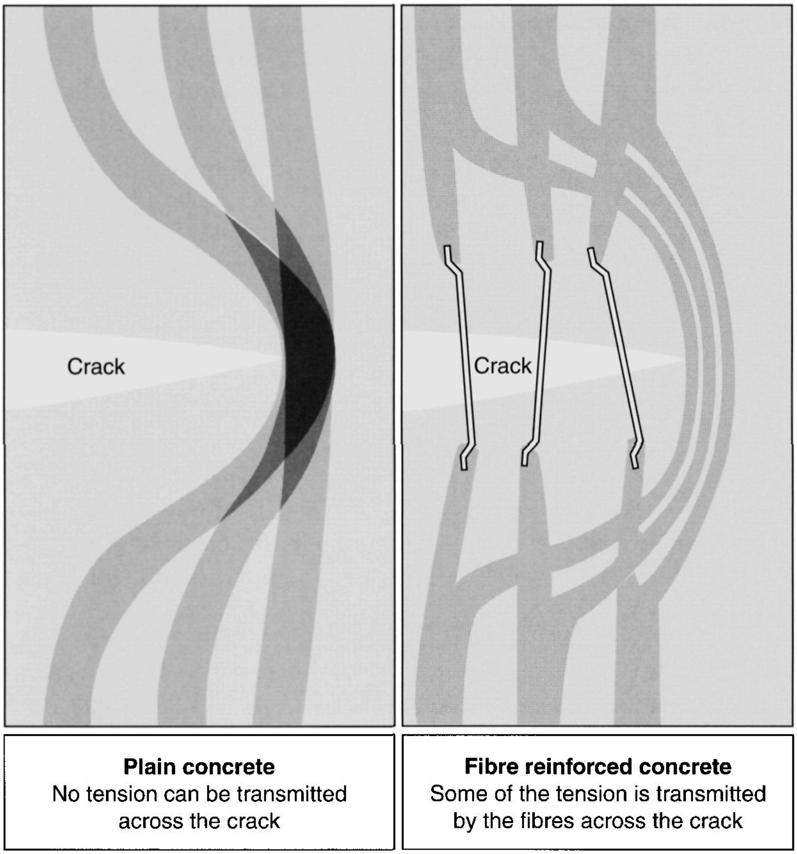



*Fig. 1.3. Stress lines in concrete under tension (Tatnall and Kuitenbrauer)*[1.19]

wire fibres in a concrete slab is to reduce the concentration of stresses near the micro-cracks by:

- fibres bridging the crack and therefore transmitting some of the load across the crack
- fibres near the crack tip resisting more load owing to their higher modulus of elasticity compared to that of the surrounding concrete.

A crack is formed where the ultimate stress in the floor slab is exceeded locally. Steel fibres cause the crack to behave as a hinge, resulting in a redistribution of stresses. Unlike a broken zone in a brittle material, this hinge can still resist stresses (depending on the type and dosage used) and thus increases the load bearing capacity of the floor. Chapter 5 shows the results of a research programme on shrinkage cracking of steel fibre reinforced concrete. Steel wire fibres should not be specified to prevent micro-cracking. Micro-cracking in steel fibre reinforced concrete will occur at a similar rate to that expected in plain concrete. Steel fibres prevent micro-cracks from developing into macro-cracks.

### 1.5.7 Flexural strength properties

The primary function of introducing steel fibres into a concrete mix is to increase the load capacity of the slab. Unlike most structural applications of concrete, when designing a floor, we rely upon the flexural strength or modulus of rupture of the concrete, to which we assign a design value. In the US, flexural strength is defined as the stress corresponding with the occurrence of the first crack in the test specimen. This is the point at which the load-deflection curve deviates from a straight line relationship and can be seen in Fig. 1.4. Flexural strength is calculated from the load at first crack and the dimensions of the test specimen.

In the Japanese standard, flexural strength is defined in terms of the maximum load and specimen dimensions as the modulus of rupture. This can be seen in Fig. 1.5. The flexural strength of concrete is determined from loading tests on concrete prisms (usually 150 × 150 × 450 mm) at 28 days. The test is run at a constant deflection rate of 0·5 mm/min. The actual deflection is recorded as a function of load. The test is continued until the deflection is at least 3 mm (1/150th of the span). The surface below the curve up to 3 mm deflection is the flexural toughness $D_b$, expressed in N/mm. The flexural toughness factor or equivalent flexural strength $f_e$ is defined as:

$$f_e = D_b I/(dbh_t^2) = 1/(150)^2 D_b$$

where $D_b$ is the flexural toughness, $I$ is the second moment of area, $d$ is the deflection of slab, $b$ is the distance of applied load from support, and $h_t$ is the height of slab (all in millimetres).

The equivalent flexural strength is the representative value for the reinforcing effect of steel fibres. In Japan, this test is already standard, while in the Netherlands it is

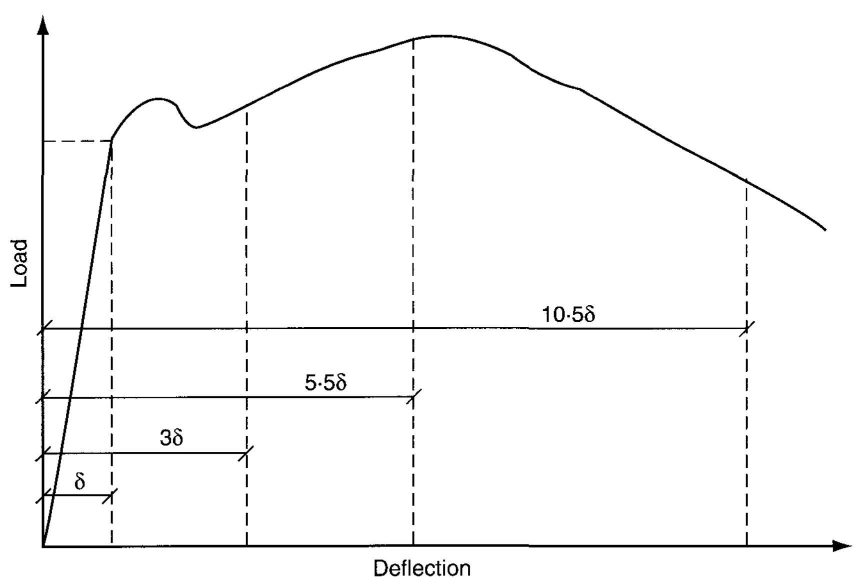



*Fig. 1.4. Flexural strength test method used in the US (δ = deflexion occurring at elastic limit)*

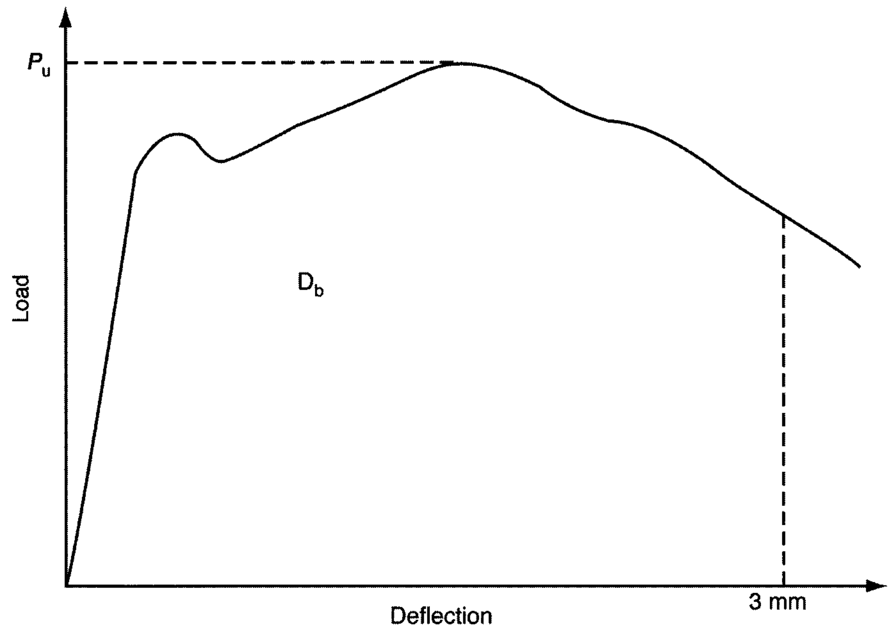



*Fig. 1.5. Japanese test method ($P_u$ = ultimate load)*

included in CUR Recommendation 10[1.20] as a basis for determining the design value of Steel Fibre Reinforced Concrete (SFRC). The Dutch design method assumes the mean value of the equivalent flexural strength to be the flexural strength design value $f_f$.

A manufacturer of anchored steel fibres commissioned TNO, Delft,[1.21] to undertake flexural strength tests using fibres embedded in C30 concrete. These tests have resulted in values of mean flexural strength of up to 4·2 N/mm$^2$, depending on dosage, type and size of fibre. Partly from these results, partly from work undertaken at the UK Cement and Concrete Association, and partly from the results reported in Section 5.14. of this book, the flexural strength values shown in Table 1.15 have been developed. The values are repeated in Table 4.2. The values in Table 1.15 apply to anchored bright wire steel fibres of length 60 mm and wire diameter 1·0 mm.

*Table 1.15. Concrete design flexural strengths with steel fibres present*

| Concrete grade and dosage | Flexural strength: N/mm$^2$ |
|---|---|
| Plain C30 concrete | 2·0 |
| 20 kg/m$^3$ steel fibre C30 concrete | 2·8 |
| 30 kg/m$^3$ steel fibre C30 concrete | 3·2 |
| 40 kg/m$^3$ steel fibre C30 concrete | 3·8 |
| Plain C40 concrete | 2·4 |
| 20 kg/m$^3$ steel fibre C40 concrete | 3·1 |
| 30 kg/m$^3$ steel fibre C40 concrete | 3·6 |
| 40 kg/m$^3$ steel fibre C40 concrete | 4·2 |

### 1.5.8 Post-cracking behaviour

The addition of steel wire fibres to a concrete floor slab ensures that it has load bearing capacity following the appearance of the first cracks, as illustrated in Section 5.14. Laboratory tests have resulted in the following theory of the behaviour of steel fibre reinforced slabs. Before the first peak load, a concrete slab exhibits elastic behaviour with the modulus of elasticity being similar to that of non-fibrous concrete. Following the first peak load and with increasing slab deflection, there is a redistribution of the bending moments (stresses) which leads to higher ultimate load capacities and thus enhanced performance.

Calculation methods used to determine the thickness of concrete floor slabs are based upon elastic theory and do not take into account the specific properties of SFRC. This has led to some debate concerning the way in which SFRC floors should be designed. The Netherlands CUR Commission has suggested the adoption of a lower elastic modulus for SFRC to account for the redistribution of stresses and has suggested the use of Japanese flexural strength specifications to account for the additional toughness inherent in steel fibre reinforced concrete. Section 5.14 shows the results and conclusions from research programmes studying the post-cracking performance of SFRC.

### 1.5.9 Resistance to impact, fatigue and corrosion

The increased resistance to fatigue provided by SFRC is of particular importance to slabs subject to heavy and intensive traffic often found on industrial floors and pavements. Such slabs must be able to resist the frequent and sudden heavy loads common in industrial areas and therefore the increased impact resistance gained by the use of fibres is an important attribute. Spalling of concrete resulting from the corrosion of steel reinforcement is greatly inhibited when SFRC is specified because of the small diameter of the steel fibres.

### 1.5.10 Economy

Steel wire manufacturers claim that substantial labour and materials savings can be achieved by the specification of steel wire fibres. Some of the economic advantages are:

- elimination of labour needed for cutting and fixing traditional mesh reinforcement
- quicker levelling of the floor owing to the absence of top reinforcement
- reduction in slab thickness compared with floors designed with plain and mesh reinforced concrete owing to increased flexural strengths
- greater joint spacing.

### 1.5.11 Specification

For any SFRC application it is recommended that the following be included in the specification.[1.20]

- A description of the desired sub-base (work floor or sheet).
- The required strength class of the concrete.
- The required rate of consistency of the SFRC.
- The method of compaction.

- Usage of a plasticizer if required and which type.
- The method of checking homogeneity of the mix.

## 1.6 Welded steel wire fabric (mesh)

Fabric should comply with the requirements of BS 4483: 1985.[1.22] Information has been used from BS 4483 to produce this section.

### *1.6.1 Introduction*

Steel wire fabric comprises an orthogonal arrangement of longitudinal wires and cross wires welded together at some or all of the cross-over points in a shear resistant manner. The fabric is usually manufactured by machine with the intersection joints formed by electrical resistance welding. Butt welded wires are also often used. The shearing load required to produce failure of a welded intersection should not be less than $0.25Af_y$, where $A$ is the nominal cross-sectional area of the smaller wire at the welded intersection and $f_y$ is the wire's characteristic yield strength. The mesh is usually supplied in bundles bound together in a flat, rolled or folded form.

### *1.6.2 Quality control*

Manufacturers specify wire of grade 460 complying with the relevant British Standards (BS 4449,[1.23] BS 4461,[1.24] BS 4482[1.25]) to produce the fabric. The number of broken welds must not exceed 4% of the total and must not exceed half the number of cross welded joints along any one wire.

### *1.6.3 Dimensioning*

Steel wire fabric is available in the wire diameter and spacing arrangements shown in Table 1.16.

### *1.6.4 General*

Steel wire fabric is assumed to carry the tensile force developed in the concrete owing to restraint to shrinkage caused by loss of moisture or temperature. Consequently, mesh allows greater joint spacings. Steel wire fabric is often used in long strip floor construction as it can be placed conveniently without the need for cutting. Where necessary, fabric sheets should be lapped at their edges and ends by 450 mm. Overlapping can result in an unacceptable build up of thickness of reinforcement. When using wire guided vehicles, interference with control signals needs to be considered, i.e. careful placing at a specific depth within the slab may be needed.

### *1.6.5 Polymer grid reinforcement*

Although they do not contribute to the flexural strength of concrete, polymer grids are a recent development that may provide an economic alternative to steel fabric. The inclusion of a polymer grid aids the early age crack control of concrete in a similar manner to fibre reinforced concrete (see Section 1.4.6). Their lightweight, non-rusting and non-magnetic nature may prove beneficial. Additional research is needed before polymer grid reinforcement can be specified commonly.

*Table 1.16. Preferred range of designated fabric types and stock sheet size*[1.22]

| Fabric reference[a] | Longitudinal wires | | | Cross wires | | | |
|---|---|---|---|---|---|---|---|
| | Nominal size: mm | Pitch: mm[b] | Area: $mm^2/m$ | Nominal wire size mm | Pitch: mm | Area: $mm^2/m$ | Mass: $kg/m^2$ |
| **Square mesh** | | | | | | | |
| A393 | 10 | 200 | 393 | 10 | 200 | 393 | 6·16 |
| A252 | 8 | 200 | 252 | 8 | 200 | 252 | 3·95 |
| A193 | 7 | 200 | 193 | 7 | 200 | 193 | 3·02 |
| A142 | 6 | 200 | 142 | 6 | 200 | 146 | 2·22 |
| A98 | 5 | 200 | 98 | 5 | 200 | 98 | 1·54 |
| **Structural mesh** | | | | | | | |
| B1131 | 12 | 100 | 1131 | 8 | 200 | 252 | 10·9 |
| B785 | 10 | 100 | 785 | 8 | 200 | 252 | 8·14 |
| B503 | 8 | 100 | 503 | 8 | 200 | 252 | 5·93 |
| B385 | 7 | 100 | 385 | 7 | 200 | 193 | 4·53 |
| B283 | 6 | 100 | 283 | 7 | 200 | 193 | 3·73 |
| B196 | 5 | 100 | 196 | 7 | 200 | 193 | 3·05 |
| **Long mesh** | | | | | | | |
| C785 | 10 | 100 | 785 | 6 | 400 | 70·8 | 6·72 |
| C636 | 9 | 100 | 636 | 6 | 400 | 70·8 | 5·55 |
| C503 | 8 | 100 | 503 | 5 | 400 | 49 | 4·34 |
| C385 | 7 | 100 | 385 | 5 | 400 | 49 | 3·41 |
| C283 | 6 | 100 | 283 | 5 | 400 | 49 | 2·61 |
| **Wrapping mesh**[c] | | | | | | | |
| D98 | 5 | 200 | 98 | 5 | 200 | 98 | 1·54 |
| D49 | 2·5 | 100 | 49 | 2·5 | 100 | 49 | 0·77 |
| **Stock sheet size**[d] | Longitudinal wires | | | Cross wires | | | Sheet area |
| | Length 4·8 m | | | Width[e] 2·4 m | | | 11·52 $m^2$ |

[a] When specifying a steel wire, fabric reference codes should be used. Reference letters A, B, C and D represent square, structural, long and wrapping mesh respectively. The numbers in the reference represent the area of steel of the longitudinal wires per metre width of fabric.
[b] The centre to centre spacing of wires in a sheet of fabric.
[c] Wire usually of grade 250 for use in wrapping fabric.
[d] Stock sheet size: Fabric types 'A' and 'B' are delivered in standard sheets of 4·8 × 2·4 m, or in scheduled size sheets. Fabric type 'C' is available in sheets or rolls.
[e] When specifying fabric sheets the width is the overall dimension measured in the direction of the cross wires.

## 1.7 Reinforcing bars

Steel reinforcing bars should comply with BS 4461: 1978 (1984).[1.24] Information has been taken from BS 4461 to produce this section. Steel reinforcing bars (rebar) are delivered to site either in stock lengths, scheduled lengths or cut and bent to the specified shape. Stock lengths are usually 6 m or 12 m. Bars delivered to site should be tagged with an identification number or code relating to the floor's reinforcing schedule or design drawings. The preferred diameters are shown in Table 1.17.

High yield grade 460 steel is use for concrete slabs (characteristic tensile strength is 460 N/mm$^2$). Bars should be free from defects and to ensure correct bonding to the concrete, there should be no loose rust, scale, grease or dirt present when the concrete is cast. Also if wire guided vehicles are to be used, bars should be fixed at a sufficient depth to avoid interference with control signals. The inclusion of steel reinforcement in concrete floors permits thinner slabs to be designed.

Reinforcing bars are often used to provide additional strength where obstructions might otherwise weaken the slab. For example, Fig. 1.6 shows the reinforcement of a

*Table 1.17. Preferred sizes of rebar*[1.24]

| Nominal size: mm | 8 | 10 | 12 | 16 | 20 | 25 | 32 | 40 |
|---|---|---|---|---|---|---|---|---|

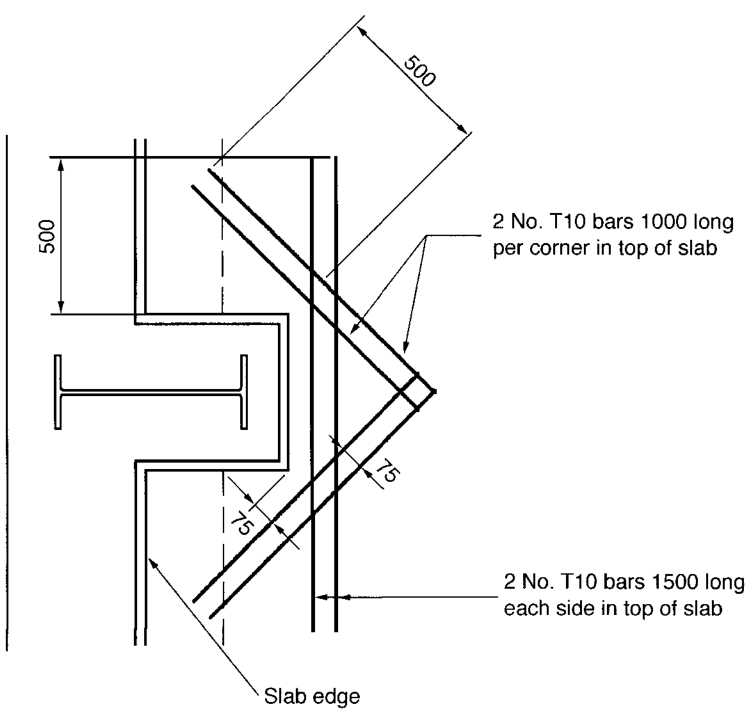



*Fig. 1.6. A floor slab is to be reinforced near a perimeter column where the edge of the floor has been diverted around the column, so creating a stress magnification point*

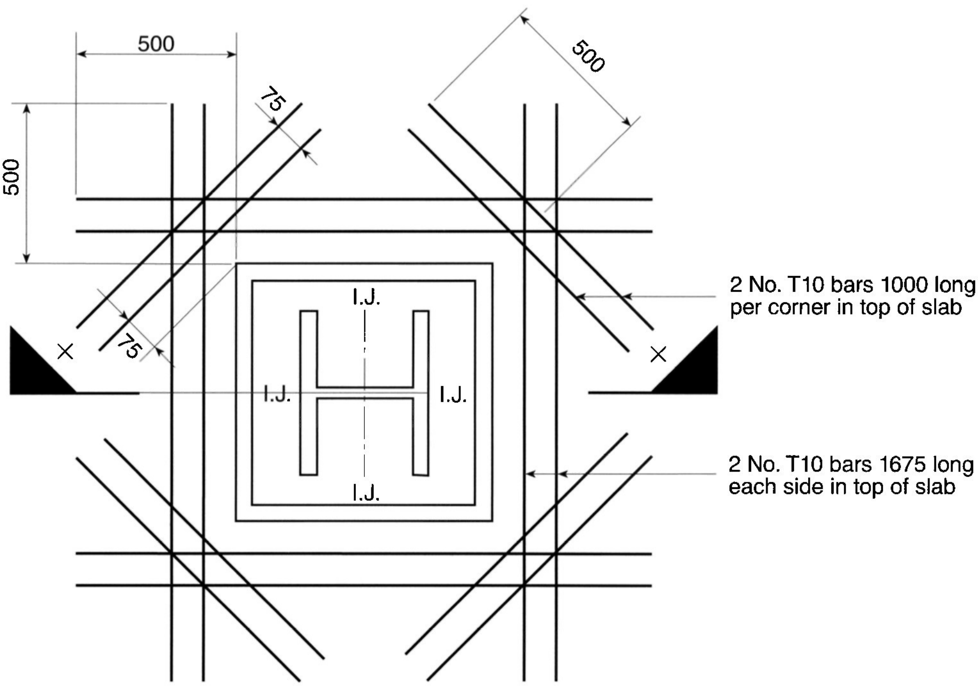



*Fig. 1.7. A column in the interior of a floor slab might cause cracking. A pattern of 10 mm diameter bars will prevent cracks from developing*

floor slab in the vicinity of a perimeter column and Fig. 1.7 shows reinforcement around an internal column.

## 1.8 Surface finishes for floors

### 1.8.1 Introduction

When selecting a specific type and method of finish, a number of factors need to be considered. These factors include the type of traffic and loading the floor will encounter, and the need for abrasion, impact and chemical resistance. With the development of many specialist toppings and surface treatments, designers are able to specify lower strength concrete for the floor slab and provide the surfacing requirements in the topping material. A number of materials are available which will improve the wear resistance, chemical resistance and the general appearance of a concrete floor as well as reduce its slipperiness and susceptibility to dusting.

### 1.8.2 Finishing techniques

To ensure good performance from a concrete floor surface, the concrete mix must undergo full compaction. Surface water often results from the vibration action performed on the concrete surface, and should be removed if a durable, wear resistant surface is to be achieved.

### 1.8.3 Cement based toppings

It can be very expensive to construct a floor with very high strength concrete. If a floor is susceptible to particularly abrasive conditions, it may be more cost effective to use a lower strength concrete with a high strength topping. Toppings are thin layers of cement-rich, fine aggregate concrete with a high shrinkage potential. High strength toppings should have an aggregate/cement ratio of about 3:1 and consist of about 30% good quality concreting sand with 10 mm single-sized coarse aggregate (often crushed rock). Debonding, curling and cracking are problems that have to be minimised with careful consideration to be taken in the design and construction of these floors. Toppings are applied in one of the following ways.[1.26]

- *Monolithic construction.* The topping is applied when the concrete is still in a plastic state. This allows the topping to become structurally integral with the slab. High strength toppings are usually between 12 and 20 mm thick. This form of construction eliminates the risk of the topping debonding from the slab.
- *Bonded construction.* The topping is bonded to the slab after it has hardened. It has little structural value and is therefore not included in the structural depth of the slab. To apply the topping, the slab's coarse aggregate must be exposed by the use of mechanised plant, such as the pneumatic scabbler or shot blaster. The surface should be rigorously cleaned of dust before the topping is applied. Bonding agents or admixtures are often used.
- *Unbonded construction.* Requires increased topping thickness and is constructed above a damp-proof or isolating membrane interposed between the topping and the slab. This method is often used when re-surfacing is needed.

### 1.8.4 Curing compounds

Curing is a vital operation in the production of a hard wearing concrete slab surface. The main objective is to prevent early drying out of the surface and therefore to allow full hydration to take place, resulting in a greater final strength and abrasion resistant concrete. Taking good care in curing reduces the risk of plastic cracking, dusting and drying shrinkage. Although traditional curing methods, such as the application of wet hessian sacks and polythene sheets, are still often used it is common to specify a curing agent, often in the form of a acrylic polymer solution which impregnates the concrete surface forming a membrane. Application of the curing membrane is usually in the form of a spray onto the newly laid concrete after the moisture sheen has evaporated, or after final powerfloating. Many curing compounds combine the two functions of curing and sealing, thereby resulting in increased concrete surface protection.

Tests undertaken by a leading UK authority on the abrasion resistance of concrete floors in relation to curing concluded that abrasion resistance can be greatly enhanced by the application of a resin based sprayed-on membrane. Polyethylene sheet proved to be the next most effective, followed by the wet hessian sack method and finally air alone. It was reported that the need for a good curing technique was particularly important when a high water/cement ratio was used.

*Table 1.18. Increasing abrasion resistance with dry shake hardeners*

| | Abrasion depth: mm |
|---|---|
| 40 $N/mm^2$ repeat powerfloated concrete[a] | 0.40 |
| Non-metallic dry shake floor hardener | 0.05 |
| Metallic dry shake floor hardener | 0.02 |

[a] The control specimen displayed the best performance which can be realistically achieved from an untreated concrete surface. A curing agent and sealing agent is often applied after finishing with the dry shake topping.

### 1.8.5 Dry shake floor hardener (sprinkle finishes)

The surface of a concrete floor slab may be enhanced by the application of a metallic, non-metallic or natural (quartz) dry shake floor hardener. Ready to use pre-blended materials consist of selected aggregates mixed with cement. When sprinkled and trowelled into the fresh wet surface of concrete floors (monolithic construction) they form a dense, toughened, wear resistant surface. The use of a dry shake topping can also prove beneficial in providing a smooth, anti-slip and non-dusting surface with increased resistance to the penetration of oils and greases. It is becoming common to apply the topping with an approved automatic spreader used in conjunction with a laser screeder, with the full specified quantity applied evenly onto the concrete immediately following screeding. Tests[1.27] using an accelerated wear abrasion machine have demonstrated how dry shake floor hardeners increase resistance to abrasion. The test results are summarised in Table 1.18.

### 1.8.6 Liquid surface treatments

Several surface treatments are available to improve the properties of the surface of a floor. Prior to application the floor surface quality should be good, otherwise the benefit may be temporary. Chemical hardeners, such as sodium silicate and magnesium fluorosilicate, are often used in solution to increase wear resistance and prevent dusting. The surface to which the treatment is to be applied should be dry and clean to allow the hardener to react chemically with the concrete to produce a case hardening effect. Most hardeners are spray applied approximately two weeks after concreting.

There are many coatings in the form of sealers and paints available. Solvent based resin sealers have been proven to improve significantly the abrasion and chemical resistance of a concrete surface so improving its performance and maintenance costs. The sealer will also provide the floor with a high resistance to the penetration of oils, cleaning detergents and many other harmful chemicals. When pigmented, a sealer can also improve the appearance of a concrete floor.

Tests[1.27] have been performed to estimate the significance of liquid surface treatments with the abrasion apparatus described in Section 1.8.5. It was concluded that penetrative resin seals were found to produce a significant increase in the abrasion resistance of a concrete surface and chemical hardeners had much less effect.

### 1.8.7 Basic recommendations

The following recommendations (reproduced from TR34[1.28]) should be considered when specifying finishing treatments:

(i) *In the majority of industrial environments a satisfactory degree of abrasion resistance can be achieved with repeated power trowelling and effective curing without modifying the surfaces of the floor in any other way.*

(ii) *In a light industrial environment abrasion resistance can be achieved with a single pass of the power trowel combined with effective curing.*

(iii) *In a heavy industrial environment some modification of the surface is required prior to or after power trowelling and curing, although the most effective way by which it can be achieved is still to be ascertained. This may include the application of a 'sprinkle' finish, high strength topping, or penetrating in surface seal.*

# 2 Construction

## 2.1 Introduction

A correctly designed and constructed ground bearing concrete slab combines the advantages of hard wear, long life and the ability to carry heavy loads at low costs. The purpose of a ground bearing slab will vary according to application and each project requires its own individual characteristics, including strength, abrasion resistance, flatness and aesthetics. An important factor commonly taken into consideration is speed of construction and the savings that accrue from fast-track construction.

While this chapter deals with some general construction matters, Chapter 5 presents ten case studies that illustrate project-specific information in more detail. They are:

| | |
|---|---|
| Section 5.3 | A single pour industrial floor |
| Section 5.4 | A long strip floor |
| Section 5.5 | A long strip external hardstanding |
| Section 5.6 | An unreinforced concrete road |
| Section 5.7 | Differential settlement of a single pour floor |
| Section 5.8 | Repairs to a cracked external hardstanding |
| Section 5.9 | Construction of a floor perimeter beam |
| Section 5.10 | Traditional industrial floor installation |
| Section 5.11 | Construction defects in an industrial road |
| Section 5.12 | Overlaying a cracked external hardstanding |

Note that Section 5.12 includes a specification for the overlay of a cracked concrete external hardstanding using concrete block paving as the new surfacing material.

## 2.2 Principal issues

The important issues in ground bearing concrete are:

- level tolerances
- cracking — load induced, moisture loss induced and temperature profile induced
- joint performance/load transfer
- skidding and/or abrasion resistance
- foundation

- ride quality
- surface texture and drainage
- loading regime.

One of the unique features in ground bearing slabs is the relationship between design, construction and performance. All of the above issues are influenced by both design and construction.

## 2.3 Traditional construction methods[2.1]

### 2.3.1 *Long strip construction*

For both internal and external concrete slabs, long strip construction has been established as the conventional way of construction since the early 1970s. The concrete is laid in a series of strips up to 6 m wide using timber or steel formwork. Every second strip is concreted initially, leaving infill unconcreted strips. The infill strips are concreted several days later using the originally laid strips as the formwork. Strips can be up to 60 m or more in length. Slabs are often thicker than 150 mm and steel mesh reinforcement may be provided. The concrete may be placed and compacted in two layers, the upper layer being placed while the lower is still plastic. Compaction takes place using either internal poker vibrators or twin-beam vibrating compactors running on the formwork or on the previously cast slabs. Care has to be exercised in the management of concrete deliveries — if there is a delay between deliveries, the earlier concrete may begin to set prior to the next delivery being placed, resulting in a 'cold joint'. This might have structural implications. Sections 5.4 and 5.5 illustrate long strip construction for a floor and an external hardstanding, respectively.

### 2.3.2 *Wide strip construction*

Wide strip construction was introduced in the UK following the development of a wide span compacting beam known as the 'Razorback' (a space frame compacting beam shown in Fig. 2.1) in the US. Razorbacks enable slabs to be laid in strips of width up to 25 m although it is more common to work in the range 9–15 m. Laying the concrete follows the same principles as for the long strip method. The concrete is placed between formwork, levelled, screeded, compacted and left to cure. In the case of roads and external hardstandings, a curing compound is applied, but in the case of floors, finishing techniques preclude this and protection from sun and rain is achieved by the building in which the floor is cast. An industrial floor should be installed when the building's cladding and doors are installed.

Often, the use of two-layer construction to incorporate mesh reinforcement is uneconomical so stools or large diameter circular fabric supports are used to enable the mesh to be positioned prior to the placing of the concrete.

The advantage of wide span construction over long strip construction is that greater daily output can be achieved with a similar sized labour force. As larger areas can be constructed, more skilled finishers are required, and in the case of floor construction, multi-headed powerfloats are commonly used, so enabling one layer compaction.

*Fig. 2.1. Use of 'Razorback' enables greater widths of concrete to be placed*

## 2.4 Large bay construction[2.1,2.2]

Frequently, concrete slabs, particularly floors, are laid using large bay or large pour methods so as to increase the daily output and to speed up the project. The first UK method was developed by A. Monk & Co. Limited working with Silidur SA of Belgium and differed from conventional ground floor construction in that no side forms were required, except to contain a day's pour. High workability concrete has a slump in excess of 150 mm which is made possible by the addition of super-plasticizers. The high slump value allows concrete to be poured directly from a truck mixer and to be spread manually. The concrete almost self-levels such that a satisfactory level can be achieved by undertaking final adjustments based upon levels provided by laser transmitters. Any discrepancies from true level can be corrected using timber screed boards to bring the surface to the correct level. Compaction is achieved by lightweight screed beams or vibrating pokers. The principal disadvantages of the method are in the segregation of the aggregate, with the larger particles sometimes sinking to the bottom of the slab and the poor surface regularity often achieved. Segregation can lead to a high concentration of fines at the surface and consequent loss of abrasion resistance. For this reason, such floors are often treated with abrasion enhancing toppings, for example Amorex.[2.3] This technique allows the use of steel mesh reinforcement, which is usually laid out a day ahead of the placing of the concrete.

Laser guided screeding machines can be configured to install concrete to a fall so as to allow external slabs to be installed. They have been used to install external concrete slabs and cement stabilised roadbases.

## 2.5 Laser screed slab installation[2.2,2.3]

Hughes Group and John Kelly (Lasers) Ltd[2.3] introduced laser guided screeding machines to Europe and Scandinavia in the 1980s. Precision Concrete Floors Ltd[2.3] were the first UK company to adopt laser screeding. By 2002, there were over 25 laser guided screeding machines constructing concrete floors in the UK. Section 5.3 illustrates the installation of a floor using a laser guided screeding machine.

### 2.5.1 The Somero S240 Laser Screed[2.2]

The machine described in this section is one of several types of laser guided screeding machines. Laser guided screeding machines combine state-of-the-art laser control systems with conventional mechanical screed mechanisms. The machines have four wheel drive, four wheel steer, including 'crab' steer for awkward areas, and are operated by one person seated at a point of maximum visibility. See Fig. 2.2.

Mounted on the twin axles, a circular fully slewing turntable carries a counter balanced telescopic boom, typically having a 6 m reach on the end of which is attached a 3–4 m wide screed carriage assembly which comprises a plough, an auger to spread the concrete accurately and a vibrating beam for compaction. Test results have shown that such machines can compact concrete to depths in excess of 300 mm.

A self-levelling laser transmitter is fixed at a visible point close to the work so as to project a 360° rotating beam across the working area. Depending on the type of transmitter, various inclinations of floors can be achieved, including level, single and dual grades. The level of the laser screeding machine is controlled by a laser beam which activates receivers mounted on the screed carriage assembly. During concreting the signals are relayed continuously to an on-board control box that automatically

*Fig. 2.2. Laser guided screeding machine. The carriage to the left is pulled towards the remainder of the machine and thereby places, compacts and levels the concrete in one operation. The levels are controlled automatically so the operator can focus upon moving the concrete. 2000 m² can be installed daily using this technique*

controls the level of the working screed head by direct intervention on the machine's hydraulic system. The laser transmitter rotates at 300 rpm so that the height of the screed carriage is adjusted five times per second.

### 2.5.2 *Laser screed operation*

Laser guided screeding machines are used in conjunction with mixer trucks which place concrete 25–35 mm above the finished floor level. The positioning of the laser screeding machine, mixer trucks, slip membrane, joints and reinforcement requires careful organisation so as to attain maximum output. Once the concrete has been deposited from the mixer, the boom of the screeding machine is extended over the freshly poured concrete and the screed carriage is lowered until the receivers lock onto the signal generated by the laser transmitter. The boom is then retracted, drawing the screed carriage towards the operator across the concrete, placing, compacting and screeding simultaneously.

In one pass a laser screeding machine can place, compact and screed 20 $m^3$ concrete in under two minutes. Because of the geometry of the horizontal auger, screeding takes place from left to right with an overlap between sequential screeding runs of 300 mm so as to ensure optimum level and surface regularity across the entire floor slab. As the pour is contained by the perimeter of a building, or by kerbs, no formwork is required except to allow for doorways and drains, or to contain a day's pour for slabs which cannot be finished in one day.

Output depends on site-specific details, type of reinforcement used (fabric can pose several problems as the laser screeding machine tends to lift the mesh out of the concrete) and the speed at which the concrete supplier can deliver the concrete to site. Outputs of between 2000 $m^2$/day and 3000 $m^2$/day are normal, and 5000 $m^2$/day has been reported.

### 2.5.3 *Laser screeding level control*

The screeding level of the machine's screed carriage is maintained by an automatic laser controlled system. Laser receivers mounted at each end of the screed carriage detect the reference datum plane emitted by a laser level transmitter situated near the work area. The on-board control box checks and adjusts the screed carriage level in relation to the laser plane five times per second.

### 2.5.4 *Advantages associated with laser screeding machines*

Those contractors using laser screeding machines have reported the following benefits as compared with traditional long strip construction.

- Higher strength, denser and more durable floors.
- Wide bay construction with maintained tolerances.
- Flatter floors (see Fig. 2.3).
- Working method ensures high productivity as it eliminates manual screeding.
- Ensures construction programmes are kept to time, enabling possibilities of earlier use of facilities.
- Damage-prone construction joints are kept to a minimum.

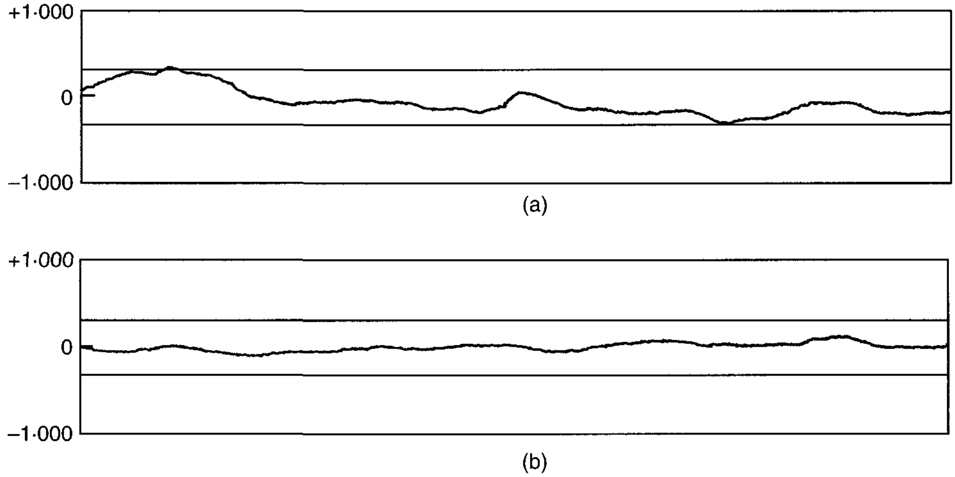



*Fig. 2.3. Comparison of floor profiles achieved with: (a) manual screeding; and (b) laser screeding*

- Choice of mesh or fibre reinforcement.
- Larger areas of floor can be placed, screeded, vibrated, compacted and left to cure in a single day. 5000 $m^2$/day has been reported but outputs of 2000–3000 $m^2$/day are more common.

## 2.6 Finishing and curing processes

Once the slab has been laid and screeded the final stages of finishing and curing are important to achieve a flat smooth durable floor or external hardstanding.

### *2.6.1 Floor finishing*

Finishing is a critical and skilful operation that takes place after the slab has been screeded. Preliminary finishing involves the use of a lightweight alloy straight edge which is drawn across a freshly laid surface to produce a flatter surface than that screeded. In the case of a floor slab, once the concrete has gained sufficient strength to support a person, powerfloating takes place using walk-behind mono powerfloats or multi-headed ride-on powertrowels in order to achieve a hard dense and flat surface, as shown in Section 5.3. On floors where high surface tolerances are required, such as in warehouse racking buildings, a combination of powertrowels and straight-edges may be used. Once finishing has taken place the concrete is left to cure.

In the case of external hardstandings, drainage and skid resitance need to be achieved. A brushed finish is often applied, as shown in Section 5.5. Once the correct surface texture is achieved, a curing compound is sprayed over the surface to prevent the rapid evaporation of the concrete's free water — see Section 2.6.2.

### *2.6.2 Curing*

Curing is an essential part of the construction process. Some consider that the labour intensive traditional method of covering the slab with wet hessian for at least seven days is the most effective curing method. If water escapes too early the upper part of the slab will dry out quickly with slower rates of curing occurring below. This can lead to low abrasion resistance and slab curling. Curing time, and hence concrete strength, is affected by temperature, wind, rain and sun. Shading may be sufficient to allow the concrete to cure. Alternatively, a spray cure membrane such as Proseal, manufactured by Armorex,[2.3] may be applied immediately following powerfloating.

## 2.7 Slip membranes[2.1,2.2]

### 2.7.1 *Purpose of slip membranes*

The purpose of slip membranes is to reduce the coefficient of friction between a concrete slab and its sub-base. They are not intended to act as a damp-proof membrane (DPM) although the material does allow the retention of some moisture. If a DPM were to be used then no moisture would escape downwards from the slab. Perforations cannot be entirely avoided and so an amount of moisture may escape into the sub-base. The slip layer reduces the coefficient of friction so allowing the concrete to move more easily as it shrinks during the curing process, hence reducing stresses in the slab, decreasing the possibility of cracking and assisting in the development of induced joints.

### 2.7.2 *Slip membrane materials*

At present there is no British Standard governing the use and type of slip membrane although polyethylene sheets are commonly used with thicknesses of 1000 gauge (250 microns) and 1200 gauge (300 microns).

### 2.7.3 *Importance of sub-base with regard to slip membranes*

Slip membranes are installed between the slab and sub-base immediately before the concrete is placed. They have to be strong enough to withstand construction traffic, such as truck mixers. It is important to overlap the sheets by at least 200 mm and use tape to ensure adjacent sheets do not move or separate. Care must be taken to avoid wrinkles and rips, which may induce cracks in the strengthening slab.

If ruts are present in the sub-base surface and the slip membrane sheets are placed over them, voids may form beneath the slab when the concrete is placed. Weak spots will develop leading to a possibility of cracking when load is applied. If the cross-section of the sub-base consists of rises and falls or peaks and troughs, the slip membrane might fail to do its task as the shape of the sub-base may cause interlock and therefore limit the horizontal movement of the slab as it shrinks. Stresses could increase and cracking ensue. It is important to ensure that the sub-base is as flat as possible and that care is taken in laying the slip membrane sheets.

### 2.7.4 *Use of slip membranes in post-tensioned slabs*

A post-tensioned slab has tendons running through its length. Once the concrete has gained enough strength, the tendons are tensioned so compressing the slab and increasing the strength of the floor by preventing tensile stress from developing in the concrete. Two layers of slip membrane can be used to reduce the coefficient of friction between the slab and the sub-base. This may lead to a reduction in the amount of reinforcement required within the slab and also to a reduction in the number of contraction joints. When two layers of slip membrane are used for their slip characteristics, as opposed to their strength, thickness may be reduced. Typically, a sheet used in two layer construction might be of 500 gauge (125 microns). Greater care must be taken in the handling and placing of thin membrane sheets.

### *2.7.5 Considerations in the design and use of slip membranes*

There are no specific guidelines regarding the use of slip membranes and the designer has the choice of whether or not to use a slip membrane. The choice will be based on considerations of shrinkage, strength, curling and joint details. The purpose of the slip membrane is to allow the concrete slab to shrink freely and so reduce the levels of stress developed by restraint to movement. Allowing movement reduces the number of contraction joints required, but the joints which are provided may be more active. A consequence of eliminating the slip membrane is the development of interlock between the sub-base and the slab. Slab/sub-base friction is enhanced and the coefficient of friction increases from 0·2 to 0·7 so inhibiting the movement of the slab and requiring more contraction joints, each moving by less. For this reason, omitting the slip membrane might be beneficial in warehouse racking buildings where large joint movements could render a floor unsuitable for pallet and fork lift trucks owing to the possibility of such equipment becoming unsteady when carrying load across joints.

Curling is caused by differential moisture loss between the surface and base of a concrete slab. When a slip membrane is used, the majority of moisture escapes from the slab through its upper surface which then shrinks and cures faster resulting in the slab attempting to curl up at its edges. Eliminating a slip membrane allows moisture to escape through the base of the slab, which leads to more even curing of the slab and so reduces curling. If too much water escapes through the base of the slab, instead of curling upwards, the slab may develop hogging with the edges of a slab attempting to curl downwards. Blinding (i.e. providing fine material at the surface) the sub-base can reduce hogging by allowing moisture to escape from the slab while at the same time preventing it from flowing away through the sub-base. The ideal slip membrane would:

- allow the slab to move relative to the sub-base as it shrinks during curing, enabling fewer joints to be constructed
- be easy to handle and strong enough not to tear
- be perforated so as to allow a controlled amount of moisture to escape and so eliminate curling.

## 2.8 Joint details

In the early life of a concrete slab, it will contract and is in danger of cracking and this is controlled by an arrangements of joints. In slabs where operating conditions can permit the presence of cracks, for example in slabs where hygiene and dust control are of

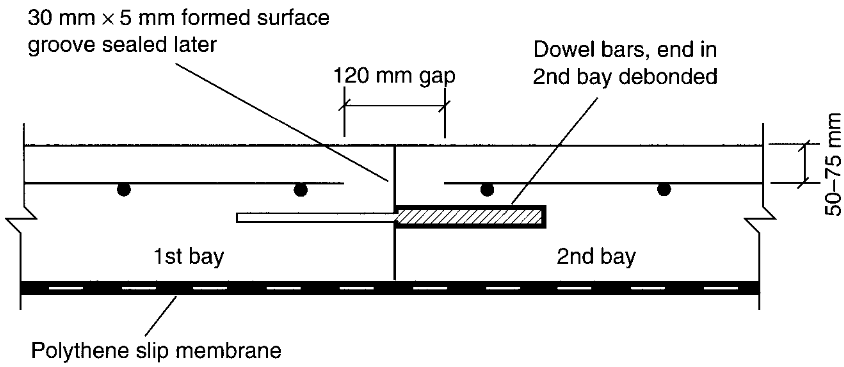



*Fig. 2.4. A Formed Doweled Contraction (FDC) joint*

secondary importance, the number of joints can be reduced and additional reinforcement provided. Particular attention should be paid to the alignment, setting and compaction of concrete at joints.

## 2.8.1 Movement joints

Movement joints are provided to ensure minimum restraint to movement caused by moisture and thermal changes in the slab. Movement joints are designed to allow the slab to contract. Expansion joints are used in regions where temperature changes can be substantial in a short period of time and are rare in the UK.

*2.8.1.1 Formed Doweled Contraction (FDC) joint.* This type of joint (Fig. 2.4) is provided at the ends and sides of a construction bay. It includes debonded dowel bars. To ensure the dowel bar and sleeve arrangement can move, the bars are debonded using one of three techniques illustrated in Fig. 2.5.

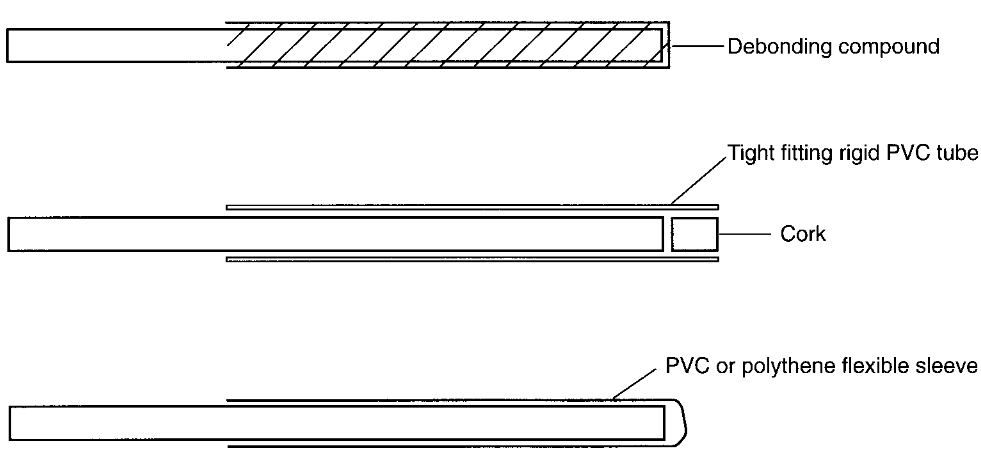



*Fig. 2.5. Three methods of debonding doweled joints*

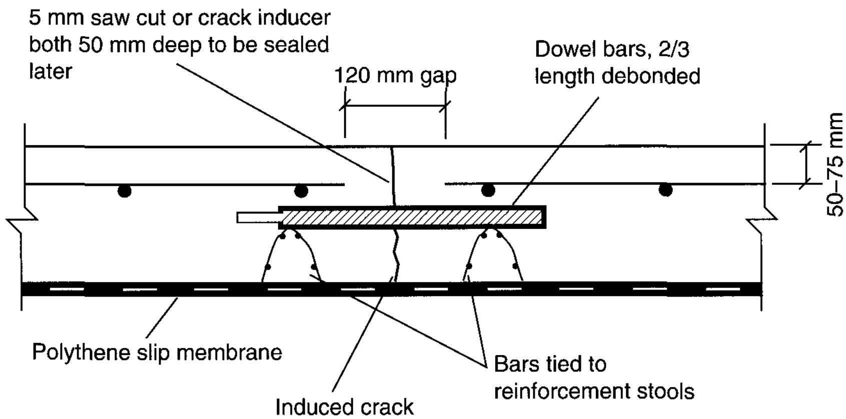



*Fig. 2.6. Induced Doweled Contraction (IDC) joint*

*2.8.1.2 Induced Doweled Contraction (IDC) joint.* This type of joint is designed to allow movements as the concrete shrinks. It is used in construction bays which are designated large (any area greater than 1000 $m^2$) and which has been poured in a continuous operation. This type of joint (Fig. 2.6) is formed by crack induction (Section 2.8.3). The dowel bars enable vertical load transfer and are fixed prior to the placing of the concrete.

*2.8.1.3 Induced Contraction (IC) joint.* Figure 2.7 shows the type of induced contraction joint used in large construction bays. It reduces the cost of joints by eliminating dowel bars and reduces the risk of joint breakdown owing to poor workmanship in the placing of the dowel bars before concreting. Load transfer depends on aggregate interlock across the induction groove. Floors incorporating this type of joint are suitable for lightly loaded applications where reinforcement requirements are minimal.

*2.8.1.4 Isolation joints.* Isolation joints or full movement joints are used to permit movements around the fixed parts of buildings such as recesses, walls (Fig. 2.8), drains

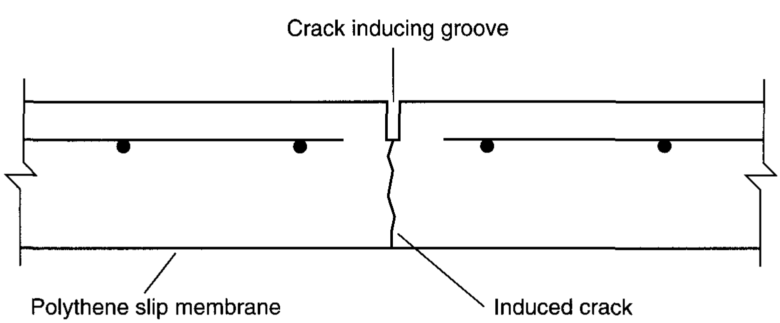



*Fig. 2.7. Induced Contraction (IC) joint*

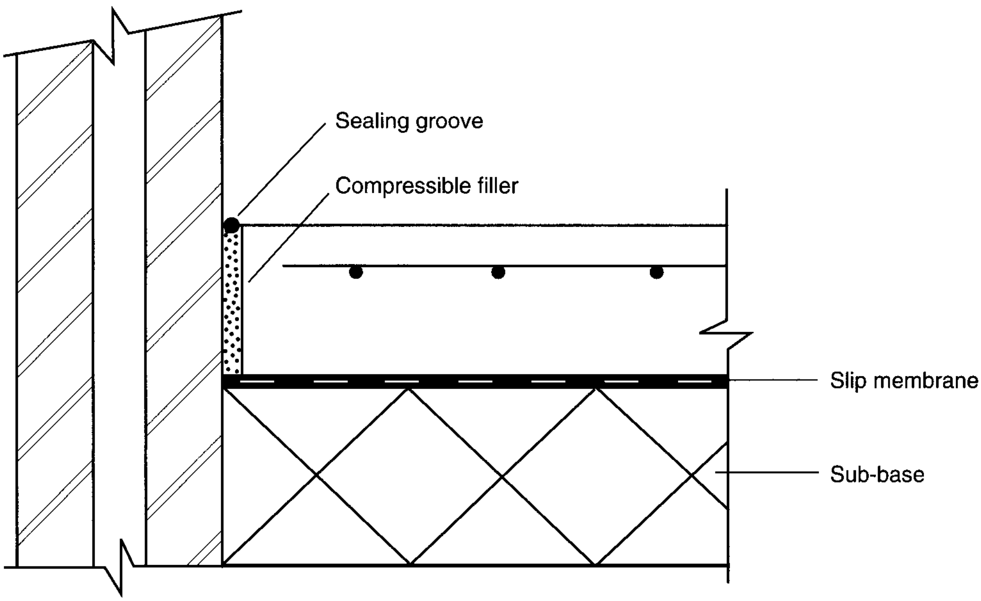



*Fig. 2.8. Isolation joint at a wall*

and columns. They keep the concrete slab separate from fixed elements. At columns there are two different methods of isolation — diamond or circular (Fig. 2.9). The circular joint is often preferred as it uses less expensive formers and has no stress inducing sharp corners.

## 2.8.2 Tied joints

Tied joints hold two construction bays together using tie bars and are primarily used to restrain movement and contraction in the horizontal plane. The tie bars also act as a form of stress relief as they are bonded within the concrete and provide a large amount of strength across the joints. Tie bars are inserted in pre-drilled holes in the formwork as the concrete is placed.

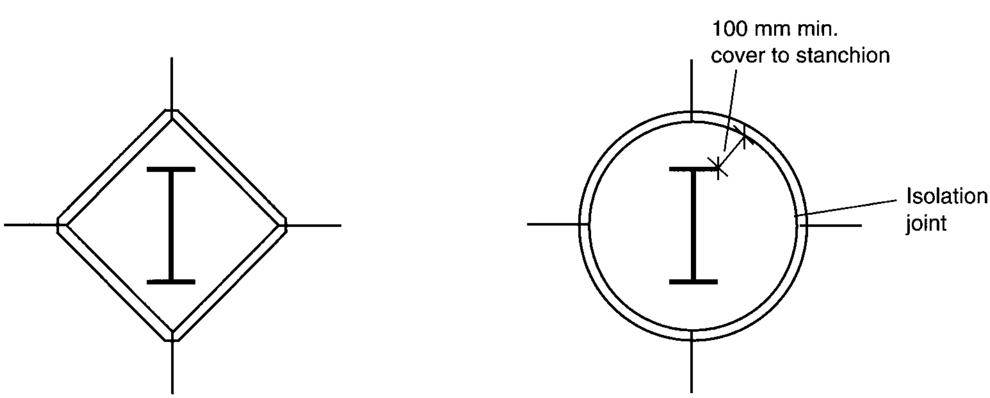



*Fig. 2.9. Circular and diamond isolation joints at stanchions (plan view)*

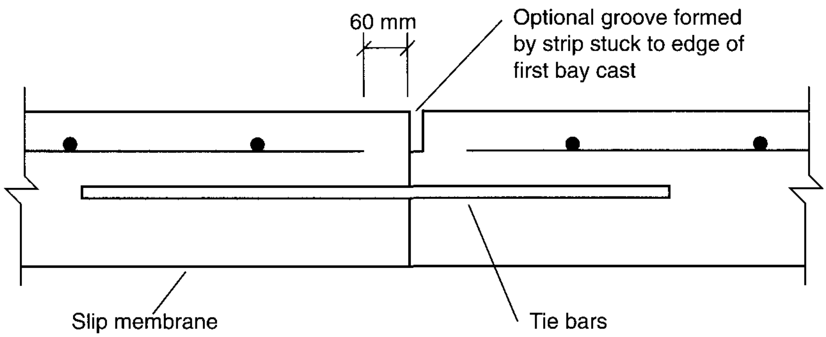



*Fig. 2.10. Formed Tied (FT) joint*

*2.8.2.1 Formed Tied (FT) joint.* This type of tied joint (Fig. 2.10) is provided around the edges of a construction bay or at a stop-end commonly when constructing concrete floors using the long strip method and has proved successful in controlling movement at joints. A groove is provided to allow the joint to be sealed (Section 2.8.5) and is formed by a strip placed on the edge of the first bay cast.

*2.8.2.2 Induced Tied (IT) joint.* Long strip and large bay pours are prone to cracking. Using induced tied joints (Fig. 2.11), construction bays are reduced in size to reduce the possibility of cracking. As the floor attempts to shrink it is restrained by the ties. Cracking may occur and a crack inducer is placed in the surface of the slab encouraging the slab to crack at the joint.

## *2.8.3 Crack induction methods*

*2.8.3.1 Sawn joints.* The saw cut acts as a line of weakness which is incorporated into the slab at the position of the joint such that the slab will crack at that point owing

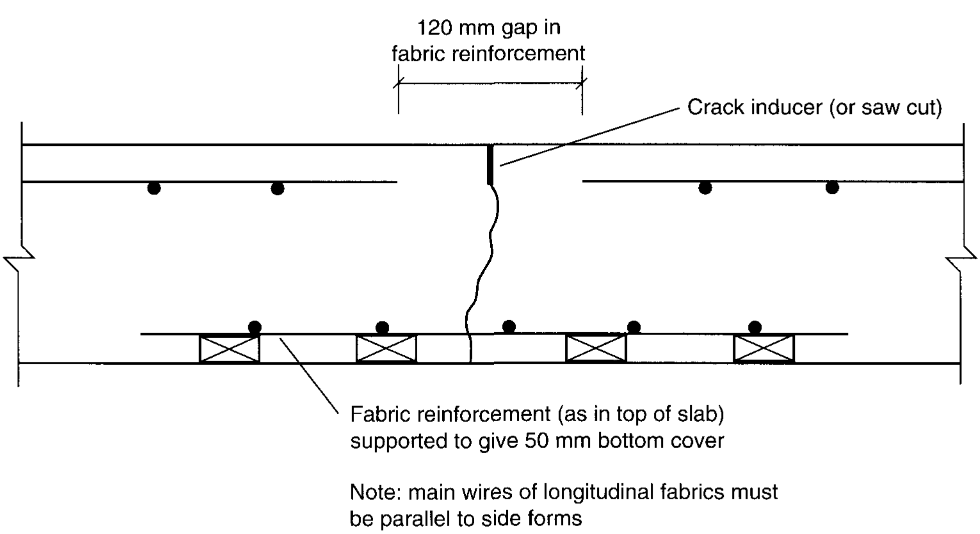



*Fig. 2.11. Induced Tied (IT) joint*

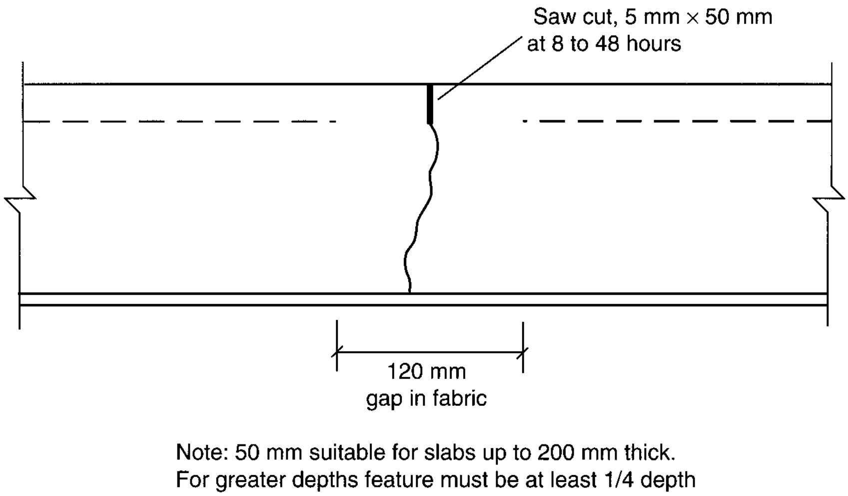



*Fig. 2.12. Crack induction joints using the saw cut technique*

to an increase in the tensile stress in the remaining depth of slab. Saw cuts are formed when the concrete has gained sufficient strength to withstand the effects of a concrete saw but not so much that the effect of sawing would damage the floor. Sawn joints are particularly durable. They are expensive and can cost as much as ten times as much as wet formed joints. The saw cut breaks the upper layer of reinforcement and the groove has a depth of 40 mm or 50 mm (Fig. 2.12). As well as the joint being the most durable crack induction form, a sawn joint is also very serviceable with no difference in level at each side of the cut. This aids flatness, which is important on heavily trafficked floors.

*2.8.3.2 The timing of forming sawn joints.* Joints should be cut when the concrete has gained sufficient strength to support the weight of the cutting equipment but before it has gained sufficient strength that sawing might loosen or pull out aggregates and fibre reinforcement. A suggested sawing time-scale is between 24 and 48 hours after initial concrete set. This leaves a time window as shown in Fig. 2.13 assuming the following.

- Concrete is mixed 1 hour before placing.
- First concrete is mixed at 7.00 a.m.
- Last concrete is mixed at 4.00 p.m.
- Concrete is placed between 8.00 a.m. and 5.00 p.m. on day 1.
- Assume concrete takes 6 hours to reach initial set.
- Therefore, earliest initial set = 1.00 p.m.; latest initial set = 10.00 p.m.

If the concrete is to be sawn between 24 and 48 hours after initial set, the first cut can be performed 24 hours after the latest initial set so as to ensure that all of the concrete has gained sufficient strength. All saw cutting must be finished 48 hours after the earliest initial set. In this example, saw cutting can commence at 10.00 p.m. on day 2 and must be finished by 1.00 p.m. on day 3. This gives 15 hours sawing time.

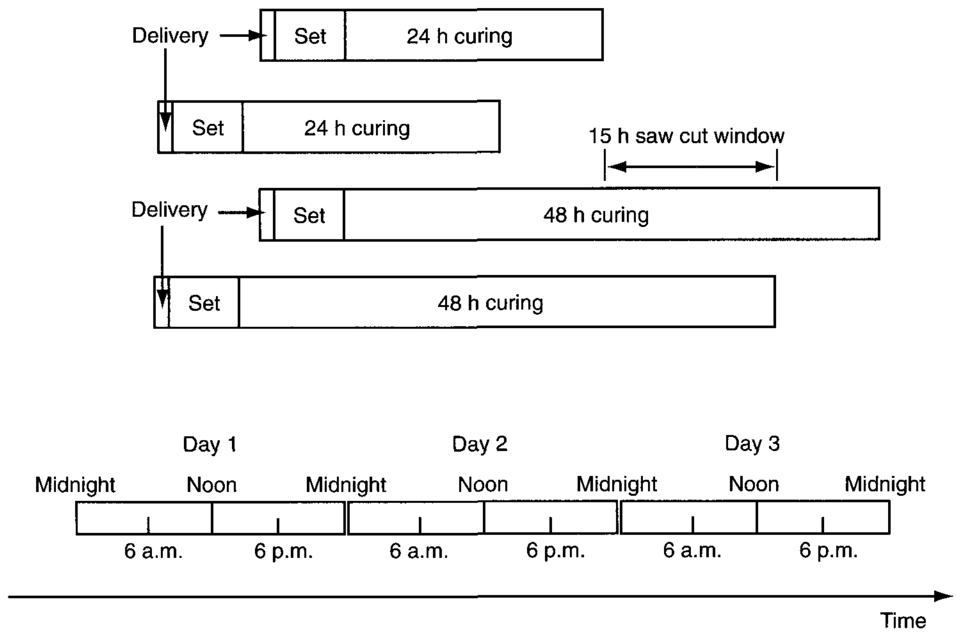



*Fig. 2.13. The timing of forming sawn joints*

*2.8.3.3 'Soff-cut' sawn joints.*[2.4] Soff-cut is a way of forming crack induction joints within a slab prior to the concrete developing significant strength. The saw cut is made immediately the powerfloats or powertrowels have finished, saving time and forming an economical joint. The Soff-cut system is used on US highway pavements and a Soff-cut joint serves the same purpose as the more conventional sawn joint.

*2.8.3.4 Other methods of crack induction.* Figure 2.14 shows traditional crack inducers involving the use of metal or plastic strips which are inserted into the wet concrete after placing. The upper section of the strip is removed once the concrete has hardened leaving the lower section in the slab to form the crack inducer. The metal insert (Fig. 2.14(b)) is shaped so as to not disrupt the concrete surface when inserted. It is strong enough to be able to be pushed into the concrete vertically. The zip-strip is used more commonly in the US and comprises a plastic extrusion made from two identical parts to form a T-shape. The rigidity of the plastic section allows the vertical section to be pushed into the concrete and the top section is removed for reuse prior to powerfloating. Immediately following concreting, a groove is formed using a bricklayers' trowel against a string line to ensure accuracy. Once the inducing strip is in place further compaction and vibration is necessary to ensure that no air has been introduced into the concrete.

## 2.8.4 Position of joints[2.5]

Table 2.1 shows joint spacings for various types of concrete. The joint spacings in the table have been used successfully for many years in the UK. Some consider that spacings can be greater than 12 m for steel fibre reinforced floors. While 14 m joint

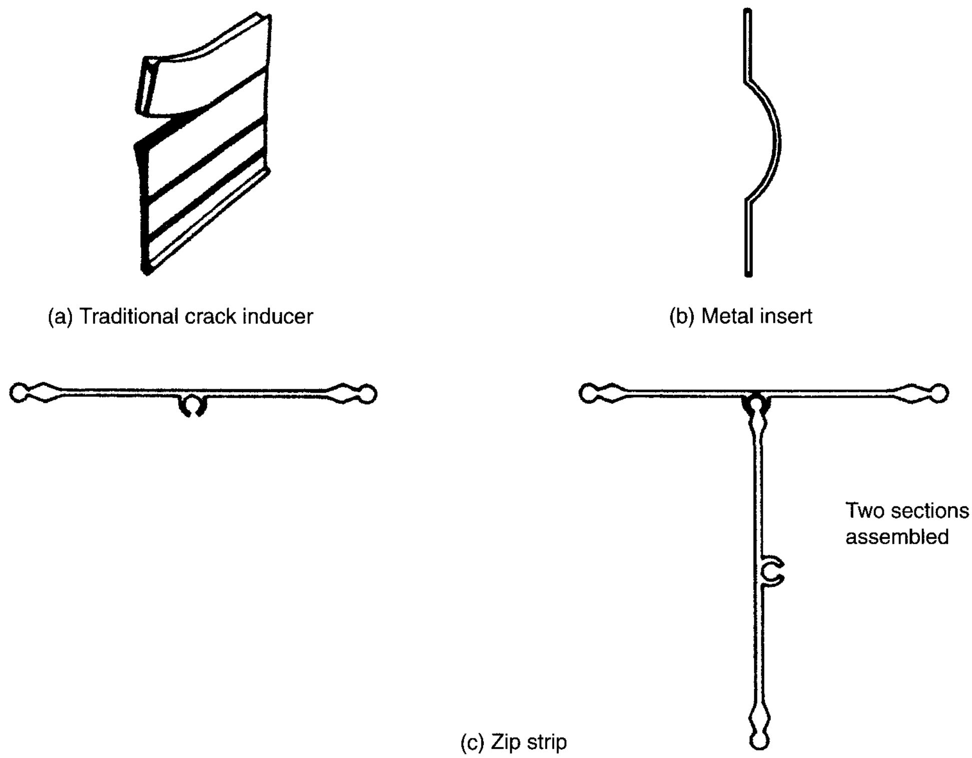



*Fig. 2.14. Traditional crack inducers*

spacings will probably be acceptable for most applications, the additional movement that would occur at joints might lead to loss of aggregate interlock and joint degradation. Joints spacings are frequently designed to coincide with column spacings or other features such as recesses or floor width changes. In any project, it is necessary to develop a joint layout identifying all practical issues prior to the placing of concrete.

*Table 2.1. Common concretes and suggested joint spacings*

| Concrete type | Joint spacing: m |
|---|---|
| Plain C30 concrete | 6 |
| Micro-silica C30 concrete | 6 |
| 20 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C30 concrete | 6 |
| 30 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C30 concrete | 8 |
| 40 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C30 concrete | 10 |
| Plain C40 concrete | 6 |
| Micro-silica C40 concrete | 6 |
| 20 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C40 concrete | 6 |
| 30 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C40 concrete | 10 |
| 40 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C40 concrete | 12 |

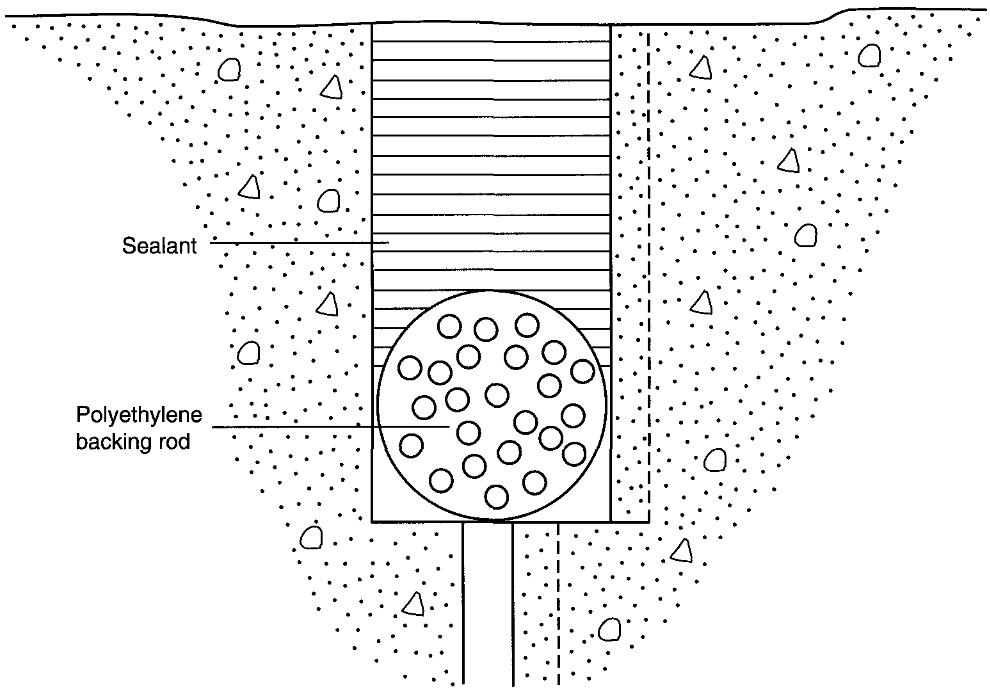



*Fig. 2.15. Details of joint sealing groove*

### 2.8.5 Joint sealing

The purpose of sealing joints is to prevent dust, water and other debris entering the joint. The sealant must be able to withstand the strain of the opening of the joint resulting from contraction of the slab and remain fixed to the faces of the groove sides (Fig. 2.15). For floors which are to be trafficked by rigid-tyred vehicles and where joint widths are greater than 5 mm, it may be necessary to use a strong semi-rigid sealant, such as pouring grade epoxy or grout, in order to provide support to the edges of the joint. The low elasticity of such sealants might cause them to fail when applied to an active joint. For this reason, they are applied at a later date when shrinkage movements have occurred. Alternatively, more flexible mastic based materials can be used. In slabs where joint spacings exceed 10 m, it is particularly important to carry out the joint sealing. With deep crack inducing grooves, a polyethylene backing rod may be placed in the lower part of the sealing groove to reduce the amount of sealant required.

Durability can be improved with polysulphide sealants and when appearance is of concern, gun-grade materials can be used, especially for narrow grooves. Sealants should be inspected and maintained regularly.

## 2.9 Floor construction case study

### 2.9.1 Details of case study

Figure 2.16 shows a plan of an industrial building which is to include a single pour floor that will be subjected to patch loads from vehicles, uniformly distributed loads from

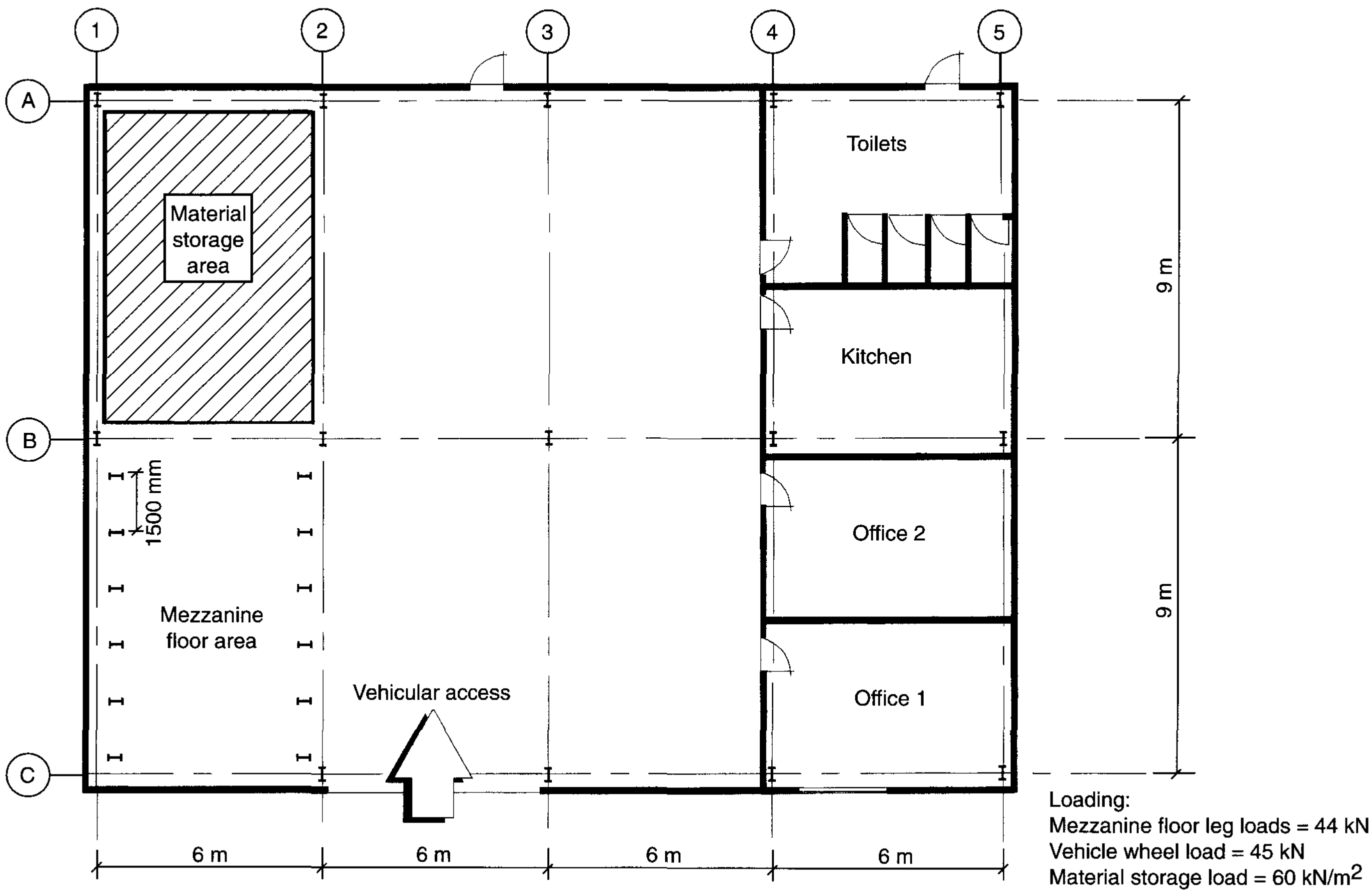



*Fig. 2.16. Industrial unit design example*

materials storage and point loads from the supporting columns of a mezzanine floor. The detailed calculations for the floor are presented in Chapter 5. The building includes several rooms which are to be constructed directly off the floor. Chapter 5 shows that a 225 mm thick floor on 250 mm thickness of granular sub-base material is suitable. A C40 concrete reinforced with 20 kg/m$^3$ of steel fibre is to be used for the floor slab. Figure 2.16 shows the grid lines which align with structural column centres. The building upon which this example is based was constructed in the UK in 1994.

Figure 2.17 shows the proposed arrangement of joints. The joint spacings are determined in Chapter 5. A perimeter beam circumscribes the floor so as to facilitate the use of a laser screeding machine. The perimeter beam is 700 mm wide and is widened at external column locations. The perimeter beam is seperated from the floor by an isolation joint which allows the floor and the perimeter beam to move independently of each other. Each of the three internal columns is surrounded by a diamond shaped pad that is separated from the floor by an isolation joint. The mezzanine floor supporting columns are to be bolted onto the floor and do not have independent foundations. The main structural columns and the building perimeter wall are to have independent foundations.

### 2.9.2 Setting out

Figure 2.18 shows the grid lines that are established on the site prior to any construction work. Pegs should be located in the ground using accurate surveying instruments at each end of each grid line. Temporary bench marks should also be established. At this stage, it may become evident that the proposed levels of the building are inappropriate. For example, it may be realised that the proposed levels will lead to excessive cut or fill. It is often the case that a revision in levels is possible at this stage, although care needs to be exercised in relation to drainage falls and highway gradients. Note that, at this stage, no attempt is made to locate the position of the floor.

### 2.9.3 Site strip

Figure 2.19 shows how the topsoil and underlying material is removed down to the level of the underside of the sub-base. Usually, this involves removing material approximately 1 m beyond the extreme grid lines. Care needs to be exercised in the protection of the underlying material from weather and construction traffic, especially as the exposed surface will be flat and therefore subject to ponding. Many types of soil in the UK will be significantly weakened if trafficked in a saturated condition and once weakened may never recover their undisturbed strength. The floor designer will have relied upon a soil strength in his structural calculations and failure to ensure that the strength is maintained may lead to the floor failing. Should the excavation become waterlogged, it may be necessary to suspend construction.

### 2.9.4 Excavate and construct pad footings for steel columns

Pad footings of plan dimensions 1·5 m × 1·5 m are to be constructed by excavating below the site strip level as shown in Fig. 2.20. If the soil remains stable with a vertical edge, no formwork is required and the pad footings can be constructed directly against the soil. Wet

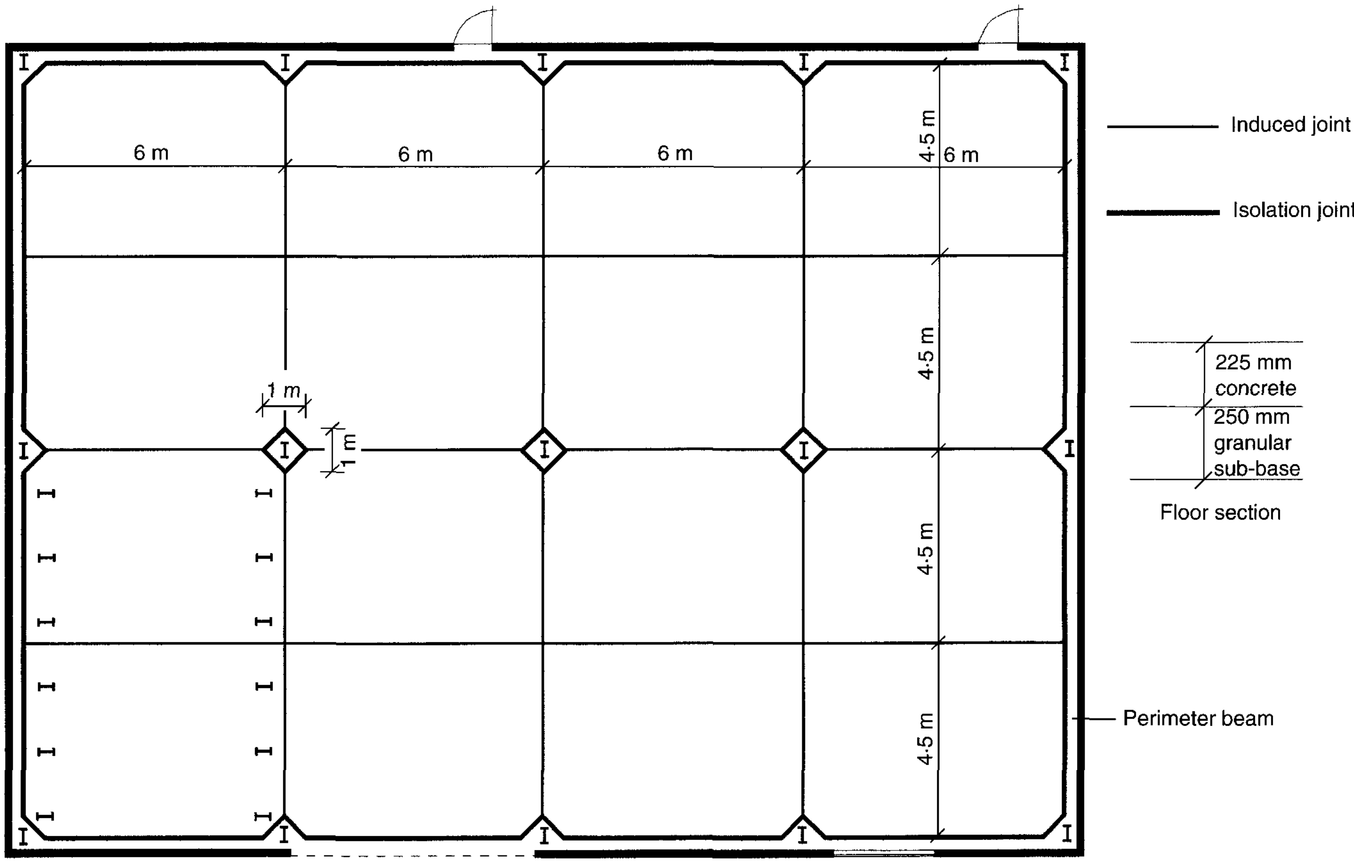



*Fig. 2.17. Arrangement of joints in design example*

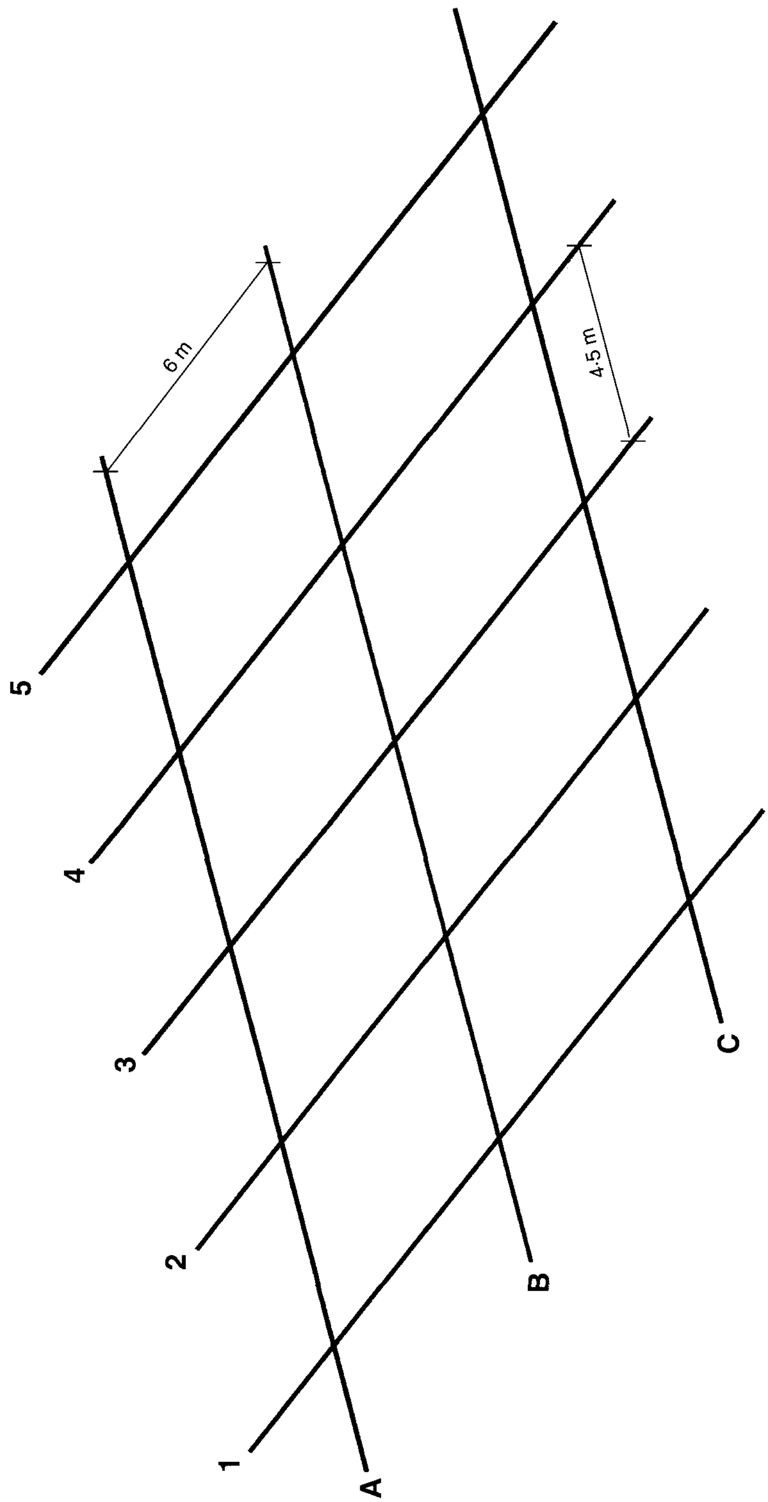



*Fig. 2.18. Establish grid lines on site*

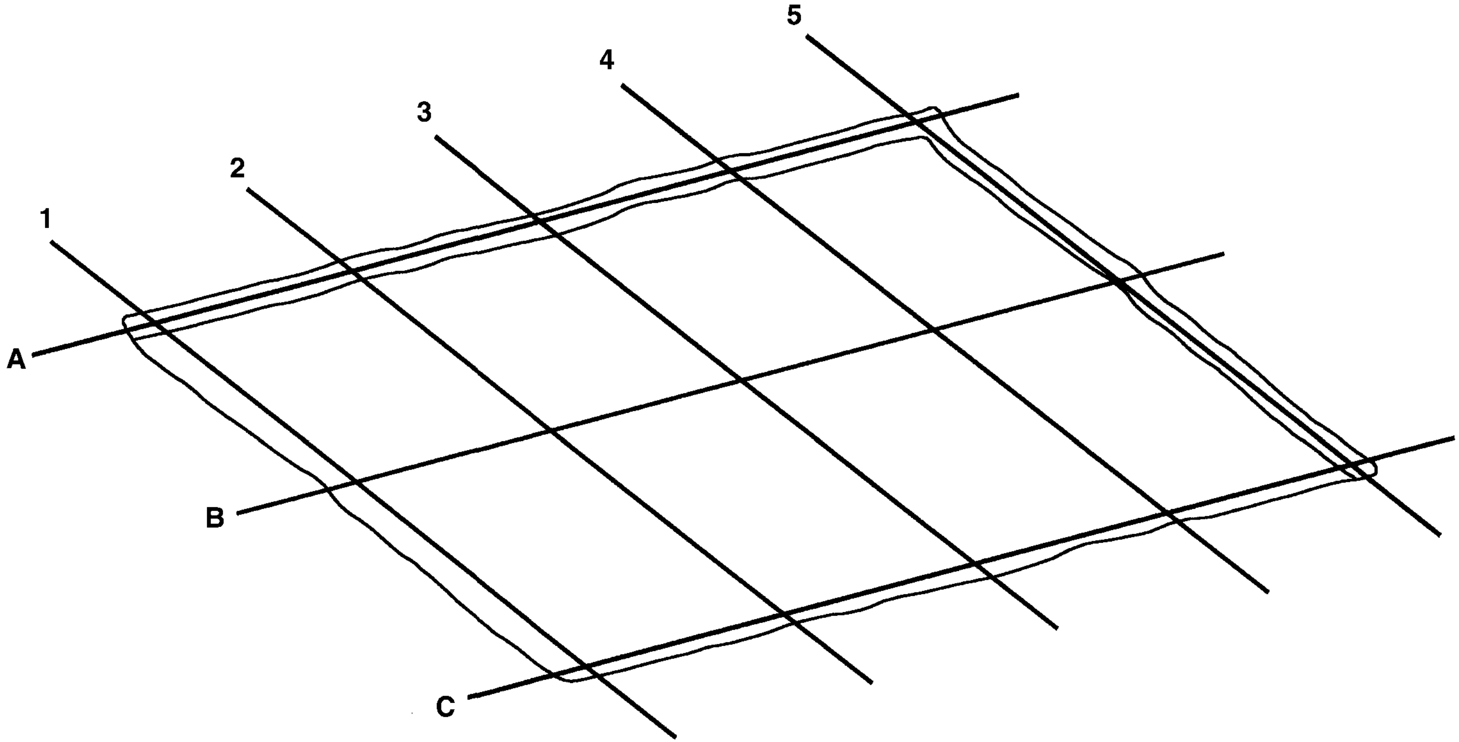



*Fig. 2.19. Topsoil strip — excavate to level of underside of sub-base*

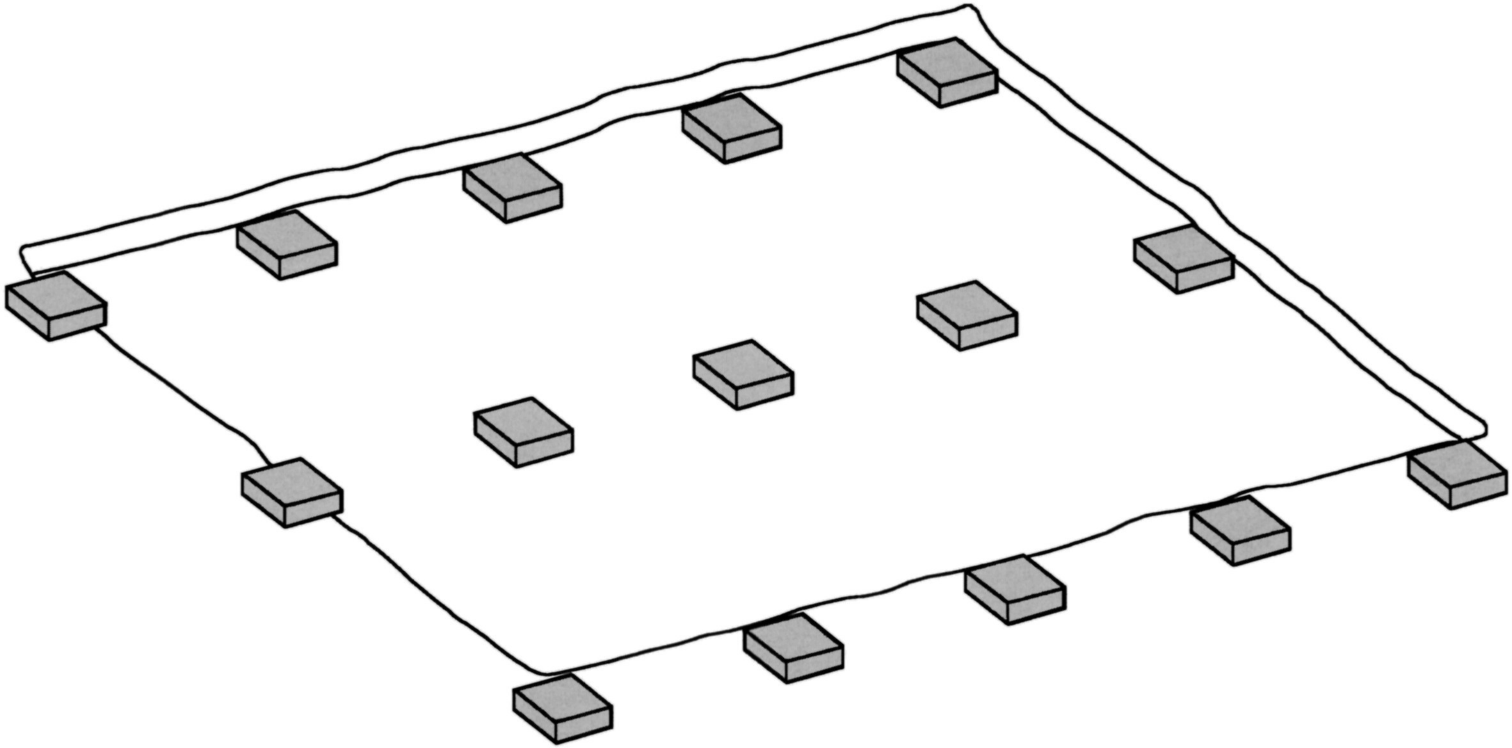


*Fig. 2.20. Excavate and construct pad footings for steel columns*

blinding concrete is first poured into each excavation, steel mesh and/or bars are then fixed and the concrete is poured. Sockets are left in the surface of the pad footings to allow the fixing of the steel columns. The upper level of the foundations has to be sufficiently deep to permit both the floor and the foundation to the building perimeter wall to be constructed. In this case, the foundations are 750 mm thick and their upper surface is 300 mm beneath the lower surface of the floor sub-base. After casting, a check should be made that each base aligns accurately with the grid. In this case, each base requires $1{\cdot}68\,m^3$ of concrete so one concrete delivery of $5\,m^3$ will be sufficient for three bases.

### 2.9.5 Install underground services

Services need to be installed and taken through the subgrade, then vertically upwards to the level of the floor. Backfilling of all trenches must be undertaken diligently to avoid later settlement which would induce stress in the floor slab. All water and gas pipe runs should be pressure tested prior to any trench reinstatement. The position of all services should be recorded to minimise disruption in case of future repairs — see Fig. 2.21.

### 2.9.6 Erect structural steelwork

The structural steelwork and the cladding must be installed before the floor can be constructed — see Fig. 2.22. Usually, the structure comprises hot rolled mild steel portal and gable frames supporting cold rolled galvanised steel purlins and side rails. Wind bracing may be installed in one or more bays and the column base plates are fixed permanently to their pad footing foundation bases.

### 2.9.7 Fix roof and side cladding

It is common for industrial buildings to include corrugated steel sheets as the roof cladding and as cladding for the upper parts of the sides, as shown in Fig. 2.23. The cladding may include an inner lining material to provide thermal insulation and roof lights, and windows may be included. More sophisticated cladding systems may be installed where end usage is predominantly office accomodation or high-tech workshop facilities. The cladding must prevent sunshine and wind from interfering with the floor construction operation.

### 2.9.8 Excavate and construct external wall foundations, then external walls

Most industrial buildings include a height of masonry or similar durable cladding material as the lower part of the building walls. This is partly to improve durability and partly to improve appearance. Two leaves are usually provided, an inner leaf of concrete blocks constructed in discrete bays between the flanges of the columns (sometimes including a box around the columns) and an external leaf of brickwork sailing past the columns externally to create an uninterrupted effect. The masonry is constructed on its own foundation comprising a strip footing constructed above the column pad footings. The concrete block leaf is 100 mm thick, the brickwork leaf is 105 mm thick and the cavity between them is 50 mm thick, so the total wall thickness is 255 mm. The inner face may be plastered or dry lined with plaster board, adding 10 mm — see Fig. 2.24.

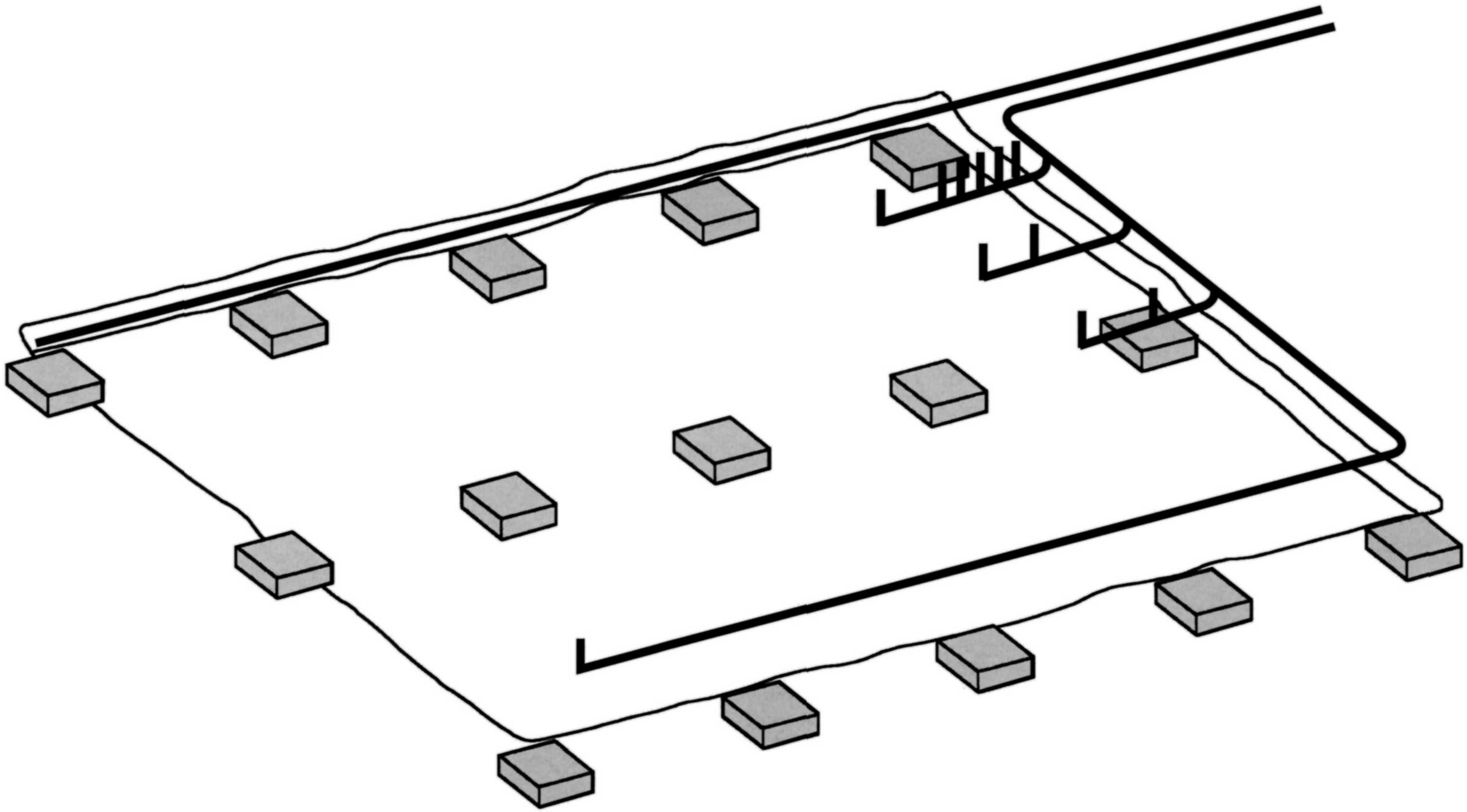


*Fig. 2.21. Install ducts and pipes for gas, electricity, water, foul drainage, surface water runoff and communications*

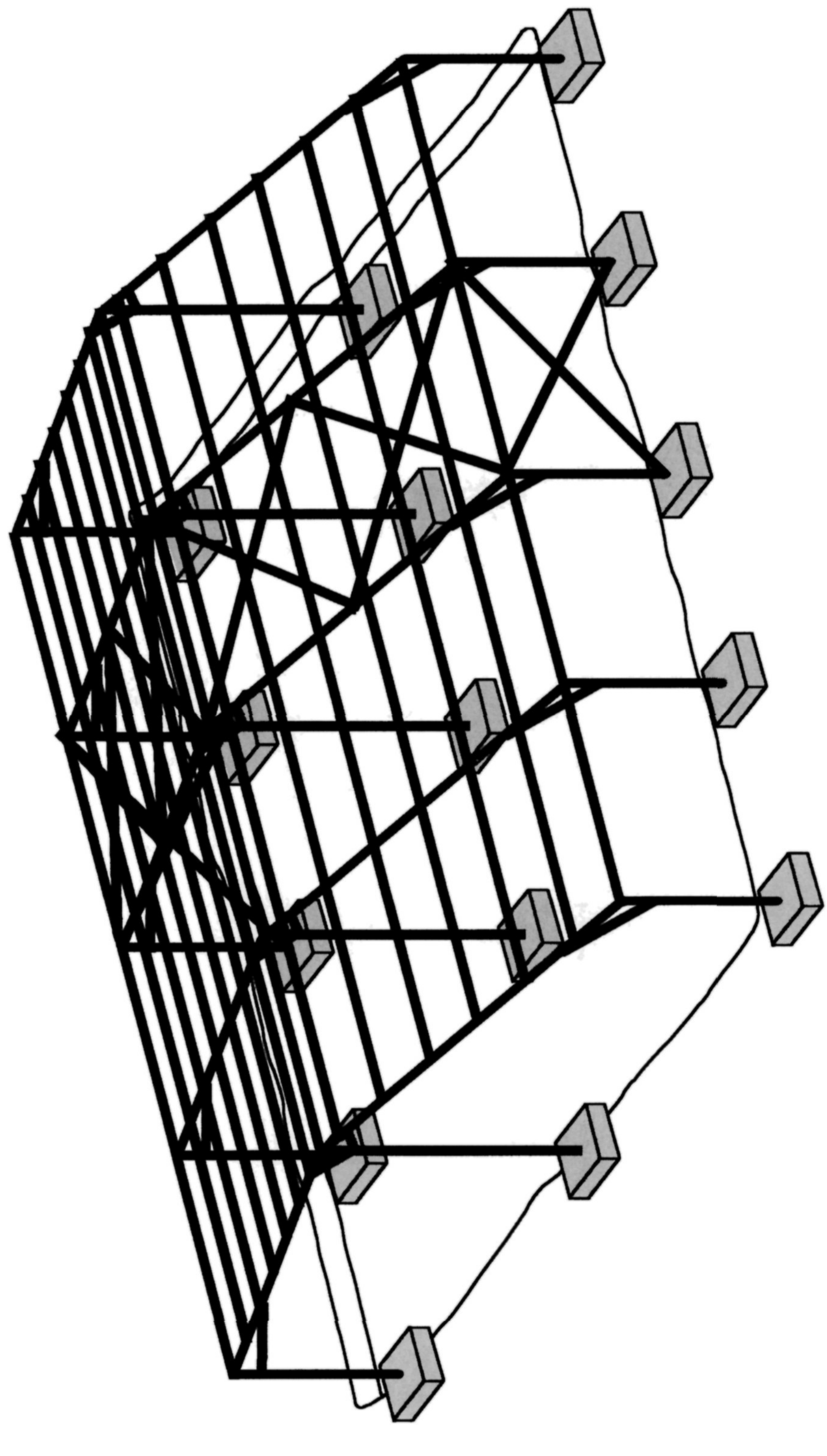


*Fig. 2.22. Erect structural steelwork*

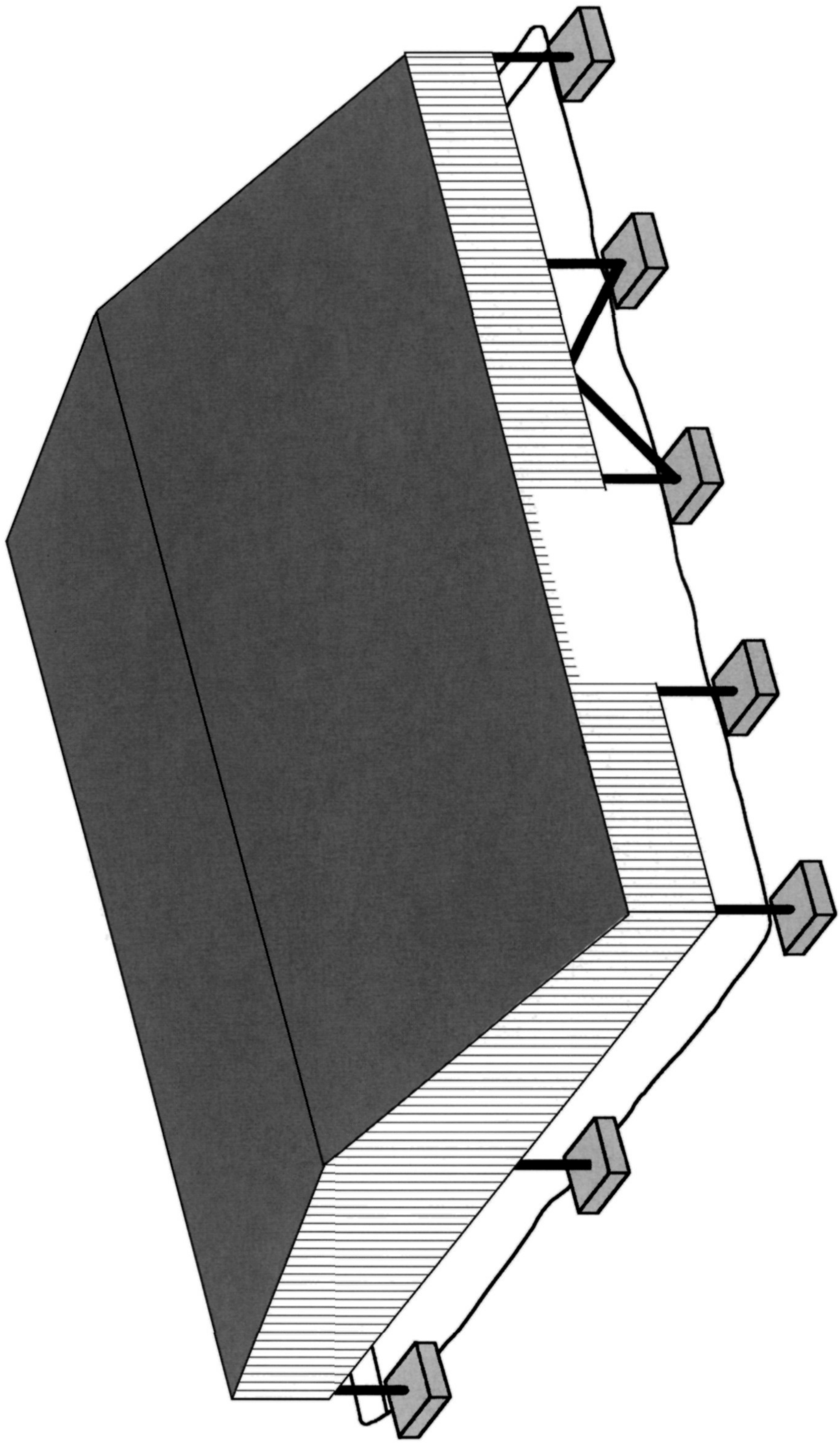


*Fig. 2.23. Fix roof and side cladding*

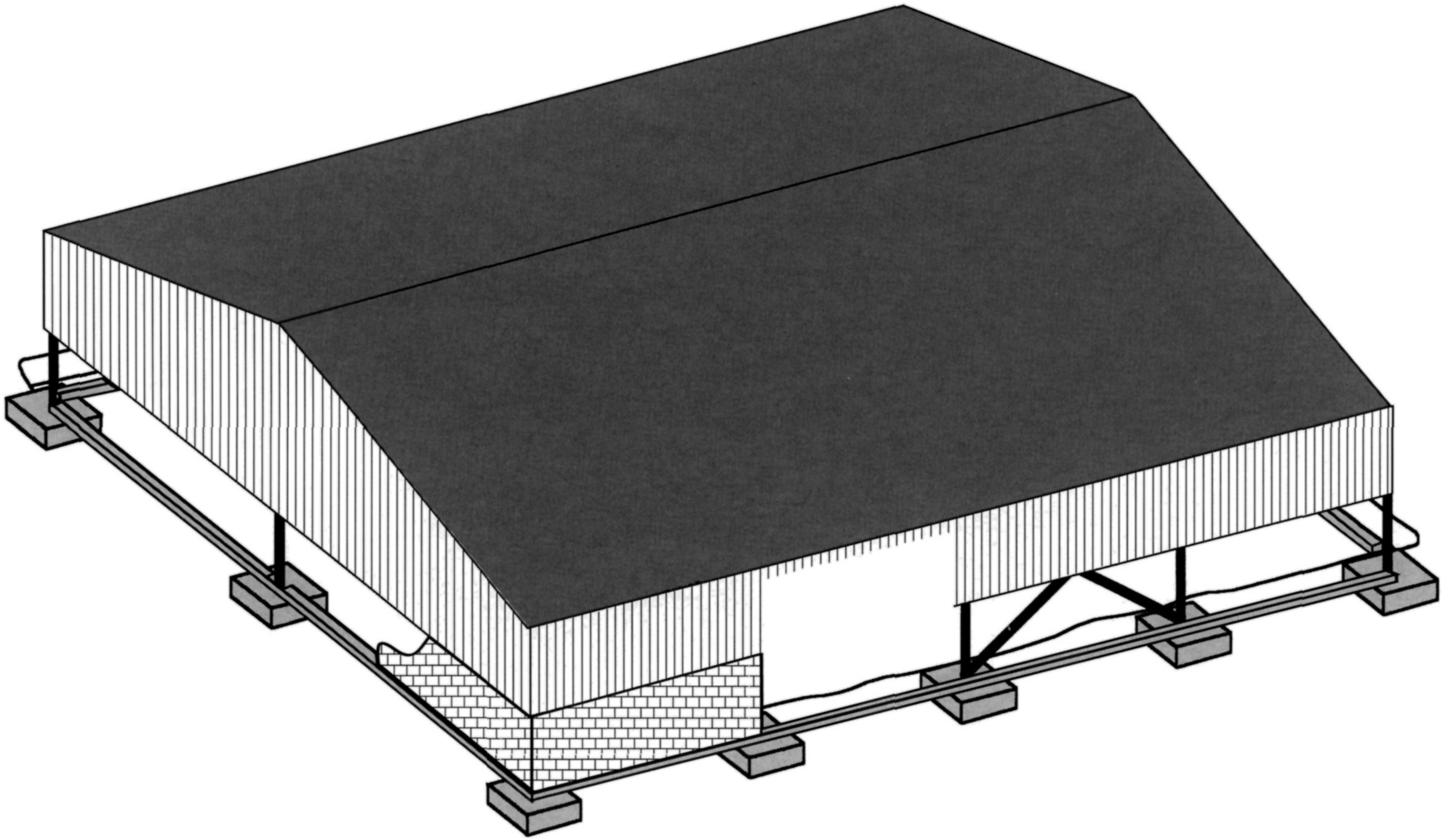


*Fig. 2.24. Excavate and construct external wall foundations, then external walls*

### 2.9.9 Construct sub-base

The crushed rock or cement stabilised sub-base is now installed by spreading the material and compacting it with a vibrating roller — see Fig. 2.25. The material is installed right up to the masonry walls and a plate vibrator is needed to compact adjacent to the walls to avoid damage. The plate vibrator is also required around service ducts. Compacted crushed rock materials have a variety of surface textures. In the case of harder or coarser grained materials, the surface may be open textured or 'hungry', in which case it should be blinded with limestone dust to create a closed surface. In the case of cement stabilised sub-bases, curing will be required comprising an impermeable spray. Although the sub-base surface can be used by construction traffic, care must be taken to avoid rutting the surface of the material. This can occur in the case of some softer limestone crushed rock materials, for example dolomite won from some Durham quarries.

### 2.9.10 Construct perimeter strip and isolation pads around internal columns

The perimeter strip is approximately 700 mm wide and the same depth as the floor — 225 mm in this case. It is separated from the masonry walls and the columns by a full movement joint. It can be constructed from the same fibre reinforced concrete as the main floor or can be reinforced with longitudinal bars. Joints should be provided according to the reinforcement percentage as for the main floor. The perimeter strip joints need not align with the main floor joints, indeed, they should not be at column positions, otherwise points of weakness will occur. Figure 2.26 shows diamond surrounds to the columns. Alternatively, circular surrounds may be provided. A full movement joint should be provided around each isolation pad and around the inside of the perimeter strip. This joint should comprise a 12 mm wide piece of compressible material fixed the full depth of the perimeter strip. A sacrificial timber or synthetic strip is installed along the upper 15 mm of the perimeter beam, to be removed later and replaced with a permanent sealer.

### 2.9.11 Install concrete floor slab

If a slip membrane has been specified, it should be installed first, care being taken to avoid folds or ripples that might weaken the concrete slab. The concrete is then spread, compacted and finished by laser screed, with power trowelling taking place after the concrete has set but before it has gained strength. Often, the power trowelling takes place between 8 hours and 16 hours after the concrete is mixed. Fibres can be added to the concrete at the mixing plant or alternatively at the site by emptying fibre bags into the readymix truck and mixing for two minutes. Usually, the readymix trucks discharge at the intended position where the concrete will form the floor. If access is not possible, the concrete can be pumped up to 100 m — see Fig. 2.27.

### 2.9.12 Cut 40 mm deep grooves to induce joints

Grooves must be cut as soon as the concrete will allow the operation to take place without forming a ragged edge to the cuts. Usually, the cutting can commence

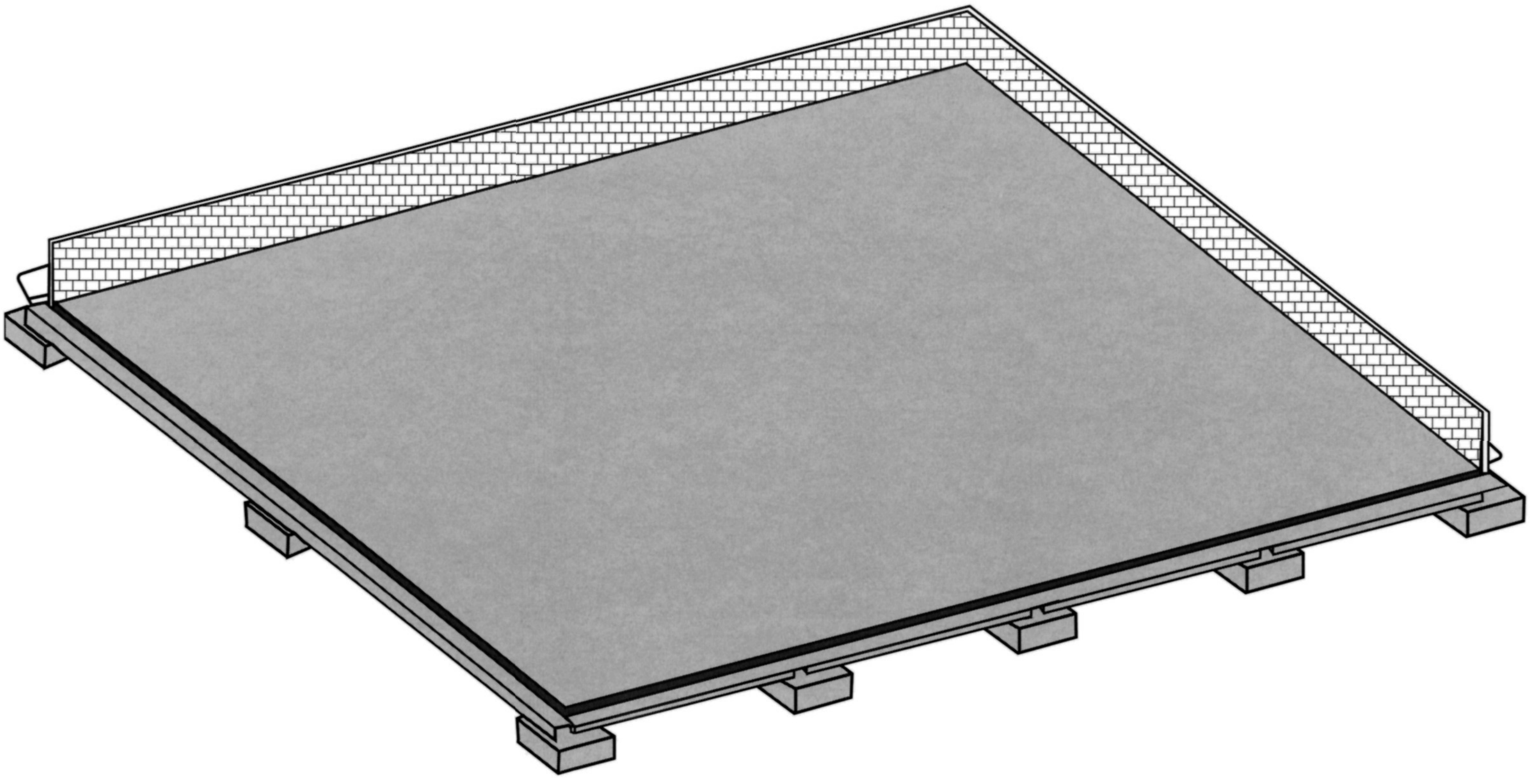


*Fig. 2.25. Construct sub-base to floor — structure, cladding and nearside walls removed from diagram for clarity*

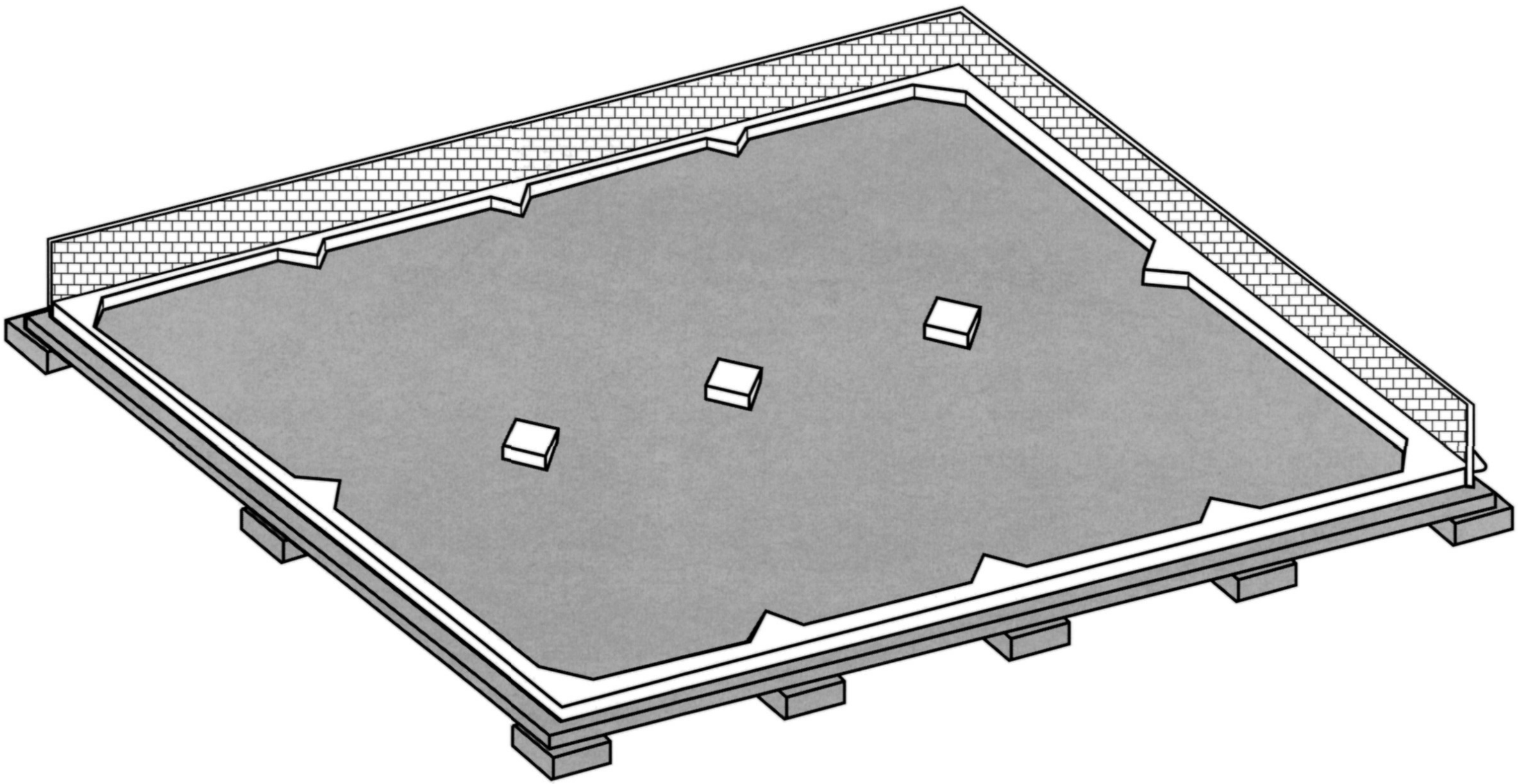


*Fig. 2.26. Construct perimeter strip and isolation pads around internal columns — nearside walls omitted from diagram for clarity*

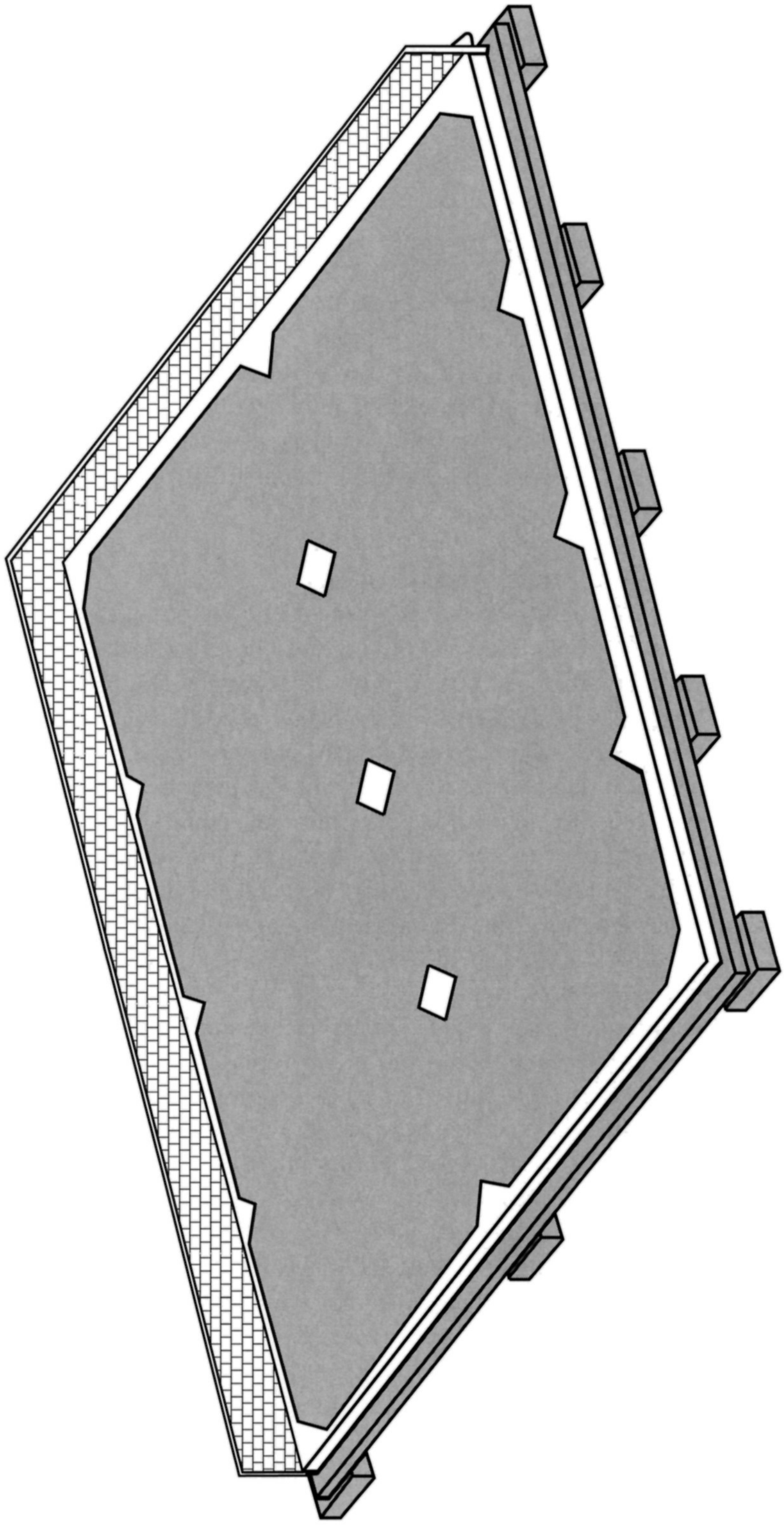


*Fig. 2.27. Install floor slab*

approximately 24 hours after mixing the concrete. Cutting should be complete within 48 hours of mixing in order to avoid the risk of uncontrolled cracking. A 3 mm wide vertically sided groove is formed using a grinding wheel. The joints are cleaned out using a high pressure air line and a sealer with an expansion capability of 30% or more is applied to the joints, including the full movement joints at the perimeter and around the internal columns — the sealer replaces the sacrificial strip placed above the compressible material — see Fig. 2.28.

### *2.9.13 Construct internal walls and mezzanine floor support columns*

Because the internal walls and mezzanine floor support columns are constructed onto the surface of the floor, they are left until last. The walls are constructed conventionally, with a layer of polyethelene being placed between the floor and the underside of the wall to act as a moisture barrier. The mezzanine floor columns are bolted to the floor using expanding bolts. Shims may be needed to ensure that the columns are vertical — see Fig. 2.29.

## 2.10 Drainage for external hardstandings

Hardstanding drainage needs to be considered alongside the design of the area since bay sizes, slopes (falls) and joint details may all influence the drainage system. The first issue to consider is where the precipitation falling on the hardstanding will go when it leaves the site. The designer will need to liaise with the local drainage authority to establish whether the downstream drainage system has sufficient existing capacity to accept the drainage from the proposed site. Also, the authority may require that a petrol interceptor be installed to remove any fuel or other light contaminents which become mixed with the surface drainage. In cases where there is insufficient downstream capacity, it may be necessary to introduce a detention system that will hold storm water until sufficient capacity becomes available. Large diameter pipes often constitute a cost-effective detention system. In this case, the diameter is designed to achieve the storage volume required rather than the flow rate.

Surface drainage can be by gulleys or by a linear drainage system, which may include 'built-in' falls. Calculations are require for the number of discharge points but a commonly adopted rule of thumb is that a gulley is required for each 300 $m^2$ of hardstanding with 150 mm diameter pipes leading to progressively higher capacity pipes as the water progresses towards the existing drainage system.

The surface falls are often specified as a compromise between the ideal 1 in 40 (2·5%), which ensures rapid removal of surface water, and the flat horizontal surface, which often suits operations best. Falls of 1 in 80 to 1 in 100 are common. Figure 2.30 shows a typical drainage arrangement for a medium sized hardstanding. The gulleys have been located in the slab corners. Care needs to be taken in the detailing of the adjacent concrete and some designers prefer to locate gulleys near the centre of slabs. Wherever a gulley is located there will be a reduction in slab strength. They should be located away from the zones of repeated heavy loading or container corner castings.

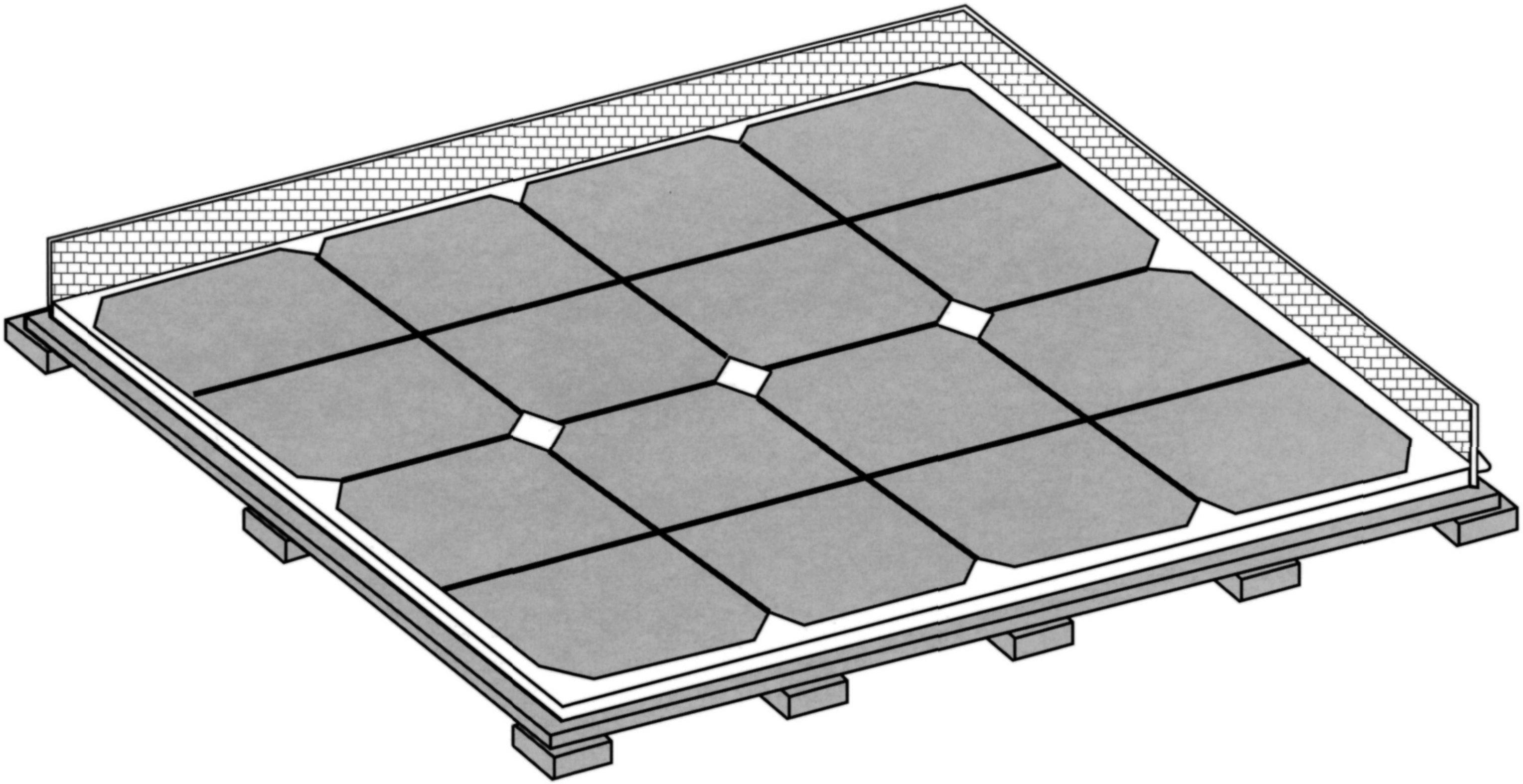


*Fig. 2.28. Cut 40 mm deep grooves to induce joints*

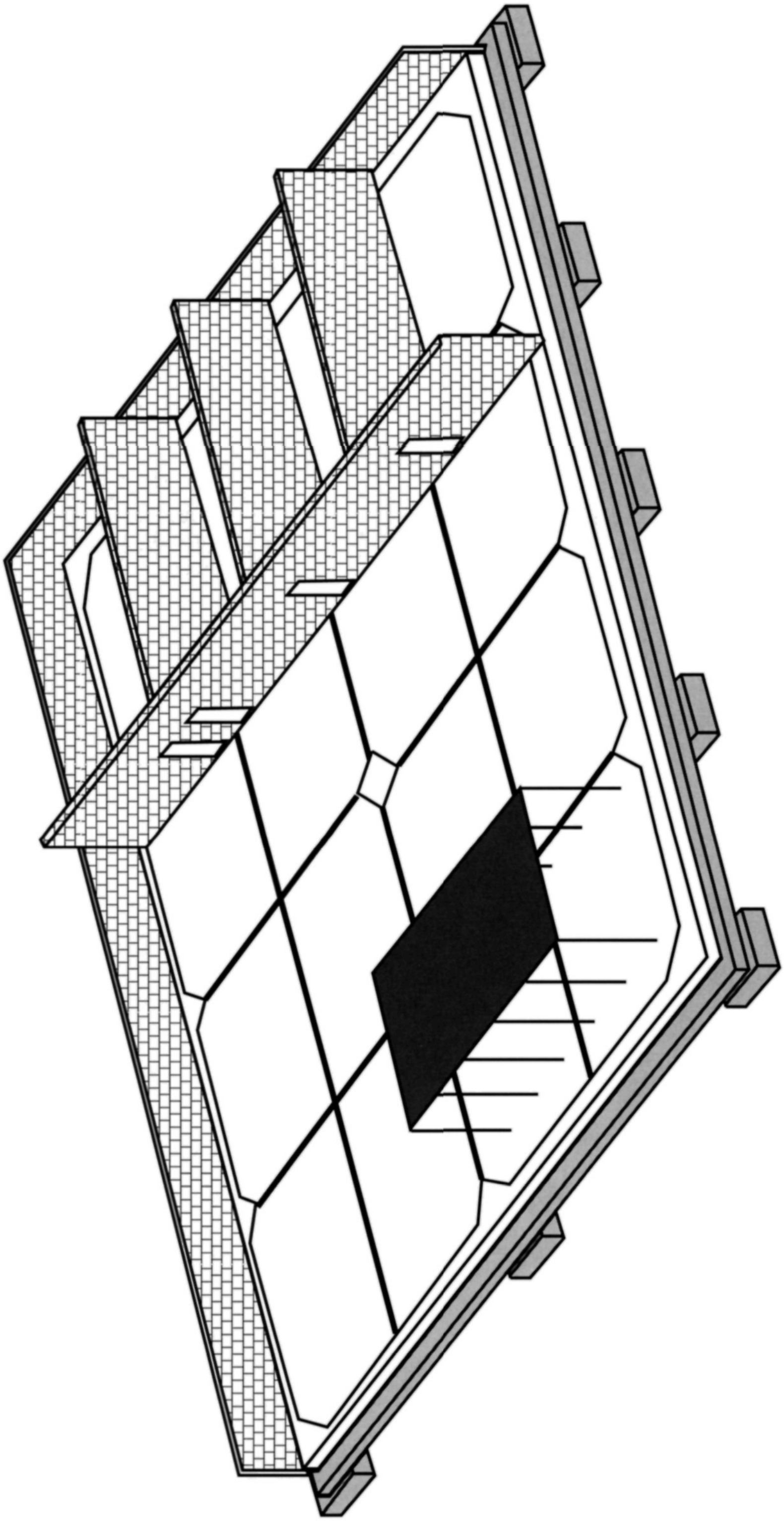


*Fig. 2.29. Construct internal walls and mezzanine floor*

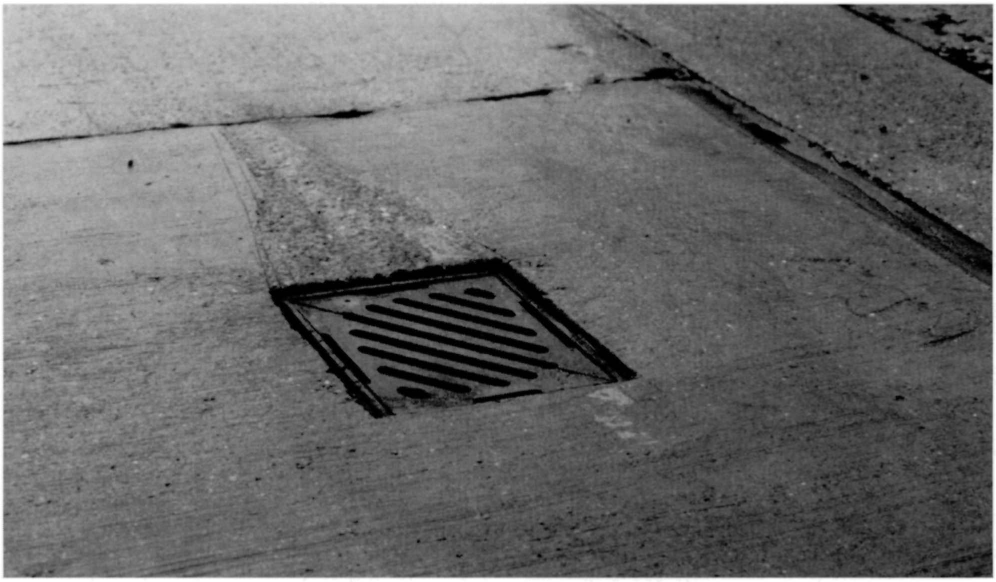


*Figure 2.30. Drainage gulley positioned within a slab. Note the channel formed in the concrete to encourage the water to reach the gulley grating*

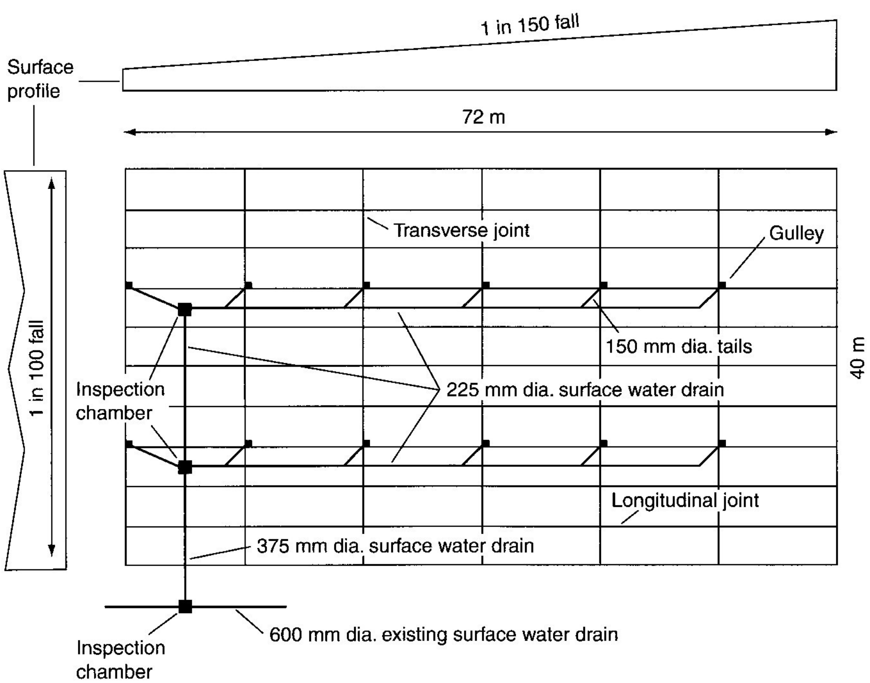



*Fig. 2.31. Typical hardstanding showing relationship between drainage and joints. This is a long strip hardstanding*

## 2.11 Hardstanding construction case study

Figure 2.31 illustrates a typical long strip hardstanding showing the joints, the falls and the drainage. If the operational characteristics of the area are determined at design time, the gulleys can be located away from the predominant loading. The contractor might opt to construct alternate strips, each with formwork at both sides, then construct the intermediate strips, using the existing concrete as formwork.

Figure 2.32 shows the way in which the individual sheets of 2·4 m × 4·8 m mesh are located within each bay. In this case, each sheet will have to be shortened from 4·8 m to just less than 4 m to fit the 4 m wide strips. Figure 2.33 shows that if A393 mesh is used, the distance between free joints can be up to 40 m. This means that all of the longitudinal joints can be tied and only one full movement transverse joint is required — the remaining transverse joints can be tied with a mid-depth sheet of mesh, or they can be eliminated.

Figure 2.34 shows the same hardstanding constructed using a laser guided screed machine. By including steel fibres in the concrete, the mesh can be omitted, so facilitating construction. It is usual to use the same joint spacing in both orthogonal directions. Increasing the weight of steel fibres increases the joint spacing and decreases the slab thickness required. The most cost effective solution is not obvious. The additional cost of fibres can be offset by a reduction in joint thickness and in a reduction in joint construction costs.

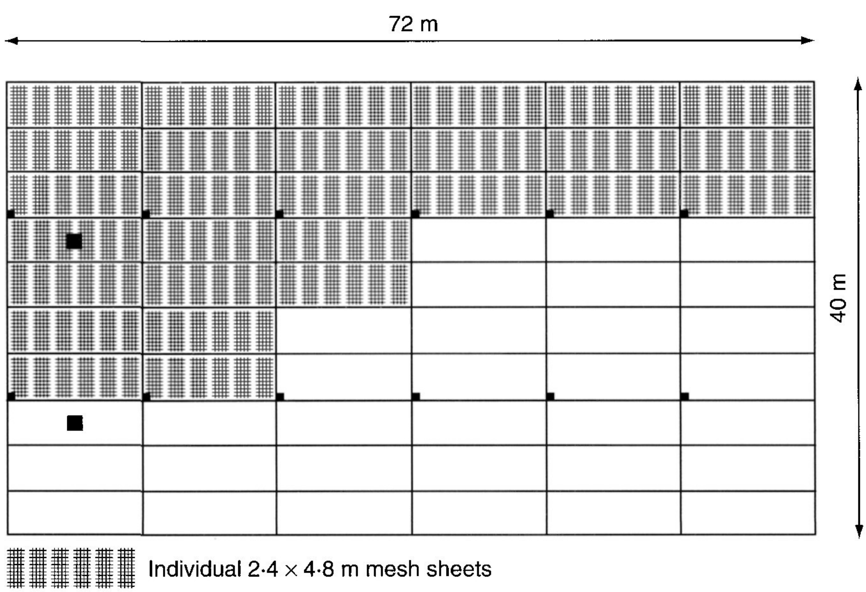



*Fig. 2.32. The positioning of steel mesh in a typical long strip industrial hardstanding. Each bay has six sheets of mesh with 200 mm overlap*

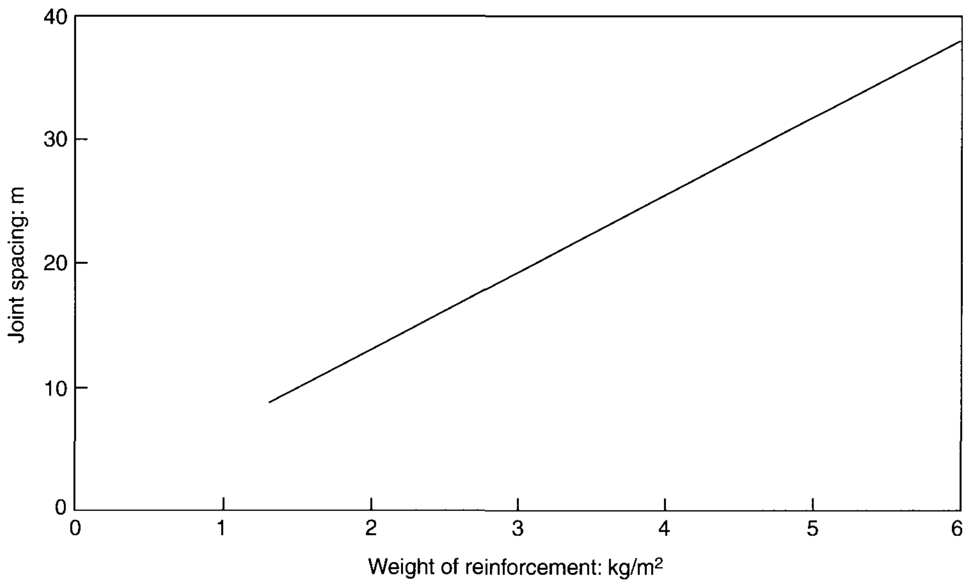



*Fig. 2.33. Joint spacing in relation to weight of steel mesh*

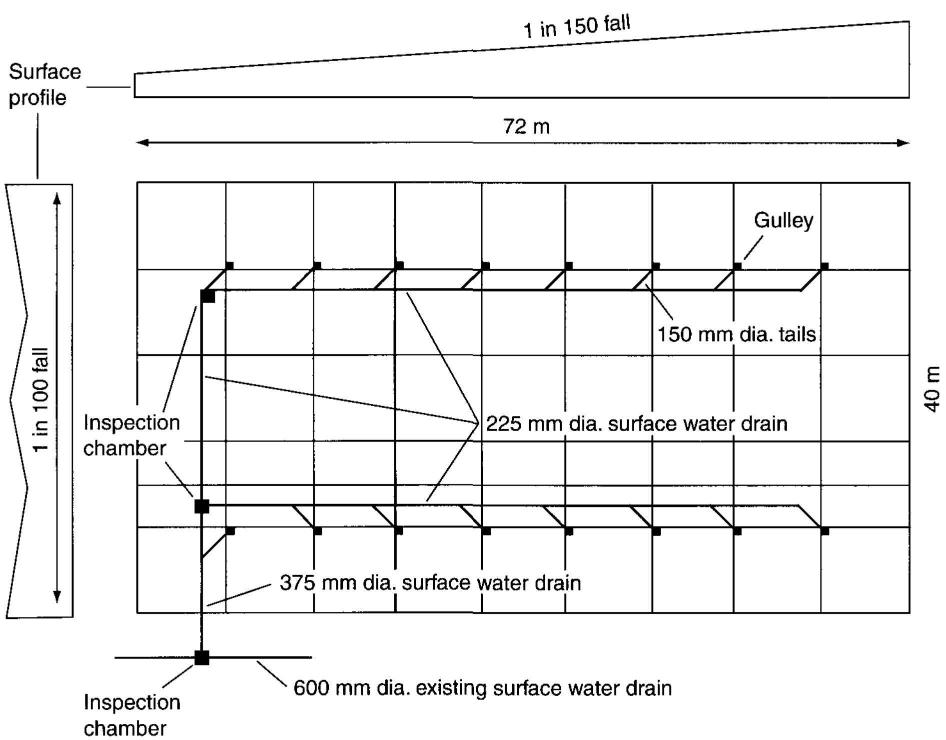



*Fig. 2.34. Typical hardstanding showing relationship between drainage and joints. This is an example of a single pour hardstanding in which the joints have been formed by sawing grooves*

# 3 Loading

## 3.1 External hardstanding wheel load values

The loading regime to be used with the equations in the case of patch loads or the design chart in the case of point loads in Chapter 4 is rationalised to a single equivalent load describing the actual regime. When the design process is started there is usually no unique load value which characterises the operational situation. Consequently it is necessary to gather information known about the loading environment in order to derive the equivalent single load to be used with the design procedure. Firstly, information regarding the types of loads that can be expected is given with factors that should be considered. This is followed by a rational method of deriving the single equivalent pavement load required for use with the design chart through proximity and dynamic factors.

Many industrial hardstandings are loaded by highway vehicles or by less onerous plant and equipment. The maximum legal axle load on a UK highway is 11 500 kg but surveys indicate that some vehicles are overloaded. It is recommended that in the absence of more accurate data, industrial hardstandings trafficked by highway vehicles or by lighter plant are assumed to be loaded by axles of weight 14 000 kg. This takes into account overloading and dynamics, but not wheel proximity. It may be that an industrial hardstanding can be designed for relatively few such vehicles, say 5% of the total vehicles expected to traverse the busiest point in the slab. Figures 3.1–3.3 show some examples of wheel loads common on industrial hardstandings.

Where loading exceeds highway levels, the usual reason is the handling of containers by off-road plant such as straddle carriers or front lift trucks (see Figs 3.4 and 3.5). In such cases, the value of the design wheel load depends upon the range of container weights being handled. Design should be based upon the critical load, which is defined as the load whose value and number of repetitions leads to the most pavement damage. Relatively few repetitions of a high load value may inflict less damage than a higher number of lesser load values. The entire load regime should be expressed as a number of passes of the critical load. The evaluation of the critical load and the effective number of repetitions of that load is as follows.

Table 3.1 shows the distribution of container weights normally encountered in UK ports for different proportions of 20 ft and 40 ft containers. Where local data are

*Fig. 3.1. Four wheel straddle carriers apply wheel loads in excess of 20 t*

*Fig. 3.2. Front lift trucks handling heavy 40 ft containers apply 100 t or more through the front axle*


*Fig. 3.3. Highway trailers may have three axles, each applying 11t. The small steel 'jockey' wheels may apply even greater load when the trailer is parked. A trailer is never moved using its jockey wheels and, in many cases, a plate is provided instead*


*Fig. 3.4. Straddle carriers are preferred to front lift trucks when significant travel distances are involved and where two or three high stacking occurs*


*Fig. 3.5. Because these containers are stacked three high, a front lift truck (FLT) is used. In this case, the containers are empty so a smaller truck is required*

*Table 3.1. Percentages of containers of different weights for five different combinations of 40 ft to 20 ft containers derived from statistics provided by UK ports*

| Container weight: kg | Proportion of 40 ft to 20 ft containers | | | | |
|---|---|---|---|---|---|
| | 100/0 | 60/40 | 50/50 | 40/60 | 0/100 |
| 0 | 0·00 | 0·00 | 0·00 | 0·00 | 0·00 |
| 1000 | 0·00 | 0·00 | 0·00 | 0·00 | 0·00 |
| 2000 | 0·00 | 0·18 | 0·23 | 0·28 | 0·46 |
| 3000 | 0·00 | 0·60 | 0·74 | 0·89 | 1·49 |
| 4000 | 0·18 | 1·29 | 1·57 | 1·84 | 2·95 |
| 5000 | 0·53 | 1·90 | 2·25 | 2·59 | 3·96 |
| 6000 | 0·98 | 2·17 | 2·46 | 2·76 | 3·94 |
| 7000 | 1·37 | 2·41 | 2·67 | 2·93 | 3·97 |
| 8000 | 2·60 | 3·05 | 3·16 | 3·27 | 3·72 |
| 9000 | 2·82 | 3·05 | 3·11 | 3·17 | 3·41 |
| 10 000 | 3·30 | 3·44 | 3·48 | 3·52 | 3·66 |
| 11 000 | 4·43 | 4·28 | 4·24 | 4·20 | 4·04 |
| 12 000 | 5·73 | 5·24 | 5·12 | 4·99 | 4·50 |
| 13 000 | 5·12 | 4·83 | 4·76 | 4·69 | 4·41 |
| 14 000 | 5·85 | 5·38 | 5·26 | 5·14 | 4·67 |
| 15 000 | 4·78 | 5·12 | 5·21 | 5·29 | 5·63 |
| 16 000 | 5·22 | 5·58 | 5·67 | 5·76 | 6·13 |
| 17 000 | 5·45 | 5·75 | 5·83 | 5·91 | 6·21 |
| 18 000 | 5·55 | 5·91 | 6·00 | 6·10 | 6·46 |
| 19 000 | 6·08 | 6·68 | 6·83 | 6·98 | 7·58 |
| 20 000 | 7·67 | 8·28 | 8·43 | 8·58 | 9·19 |
| 21 000 | 10·40 | 8·93 | 8·56 | 8·18 | 6·72 |
| 22 000 | 9·95 | 7·60 | 7·02 | 6·43 | 4·08 |
| 23 000 | 5·53 | 4·31 | 4·00 | 3·69 | 2·47 |
| 24 000 | 2·75 | 1·75 | 1·50 | 1·25 | 0·24 |
| 25 000 | 0·95 | 0·63 | 0·55 | 0·47 | 0·15 |
| 26 000 | 0·67 | 0·40 | 0·33 | 0·27 | 0·00 |
| 27 000 | 0·72 | 0·43 | 0·36 | 0·29 | 0·00 |
| 28 000 | 0·53 | 0·32 | 0·27 | 0·21 | 0·00 |
| 29 000 | 0·43 | 0·26 | 0·22 | 0·17 | 0·00 |
| 30 000 | 0·28 | 0·17 | 0·14 | 0·11 | 0·00 |
| 31 000 | 0·03 | 0·02 | 0·02 | 0·01 | 0·00 |
| 32 000 | 0·03 | 0·02 | 0·02 | 0·01 | 0·00 |
| 33 000 | 0·00 | 0·00 | 0·00 | 0·00 | 0·00 |
| 34 000 | 0·05 | 0·03 | 0·02 | 0·02 | 0·00 |

available, they can be used in place of Table 3.1. For each of the container weights shown in Table 3.1, calculate the damaging effect caused when plant is handling containers of that weight from the following equation:

$$D = (\boldsymbol{W}/12000)^{3\cdot75}(\mathbf{P}/0\cdot8)^{1\cdot25}N \tag{3.1}$$

where $D$ is the damaging effect, $\boldsymbol{W}$ is the wheel load corresponding with specific container weight (in kg), $\boldsymbol{P}$ is the tyre pressure (N/mm$^2$) and $N$ is the percentage figure from Table 3.1.

The container weight leading to the greatest value of $D$ is the critical weight container and all subsequent wheel load calculations should be based upon this load. Experience indicates that when the containers being handled comprise 100% 40 ft containers, the critical load is commonly 22 000 kg and when 20 ft containers are being handled, the critical load is 20 000 kg. In general, mixes of 40 ft/20 ft containers have a critical container weight of 21 000 kg. These values may be used in preliminary design studies. The number of repetitions to be used in design can be calculated accurately using a load value weighted system. However, if the total number of repetitions calculated solely from operational data is used, a negligible error will be generated. In the case of pavements trafficked by highway vehicles, an equivalent wheel load of 140 kN may be used.

## 3.2 Tyres

The contact area of a tyre of handling plant is assumed to be circular with a contact pressure equal to that of the tyre pressure. Some larger items of plant may be fitted with tyres for operating over soft ground. When such tyres travel over concrete the contact area is not circular and the contact stress under the tread bars is greater than the tyre pressure. This has little effect in the case of in-situ concrete. Some terminal trailers are fitted with solid rubber tyres. The contact stress depends upon the trailer load but a value of 1·7 N/mm$^2$ is typical and the higher pressure is dispersed satisfactorily through the pavement.

## 3.3 Dynamics

The effects of dynamic loading induced by cornering, accelerating, braking and surface unevenness are taken into account by the factor $f_d$. Where a section of a pavement is subjected to dynamic effects the wheel loads are adjusted by the factors given in Table 3.2 as explained in the notes to the table.

## 3.4 Lane channelisation

Plant movements over a wide pavement do not follow exactly the same course, but wander to one side or the other. If there are lane markings with the lane approximately the same width as the plant, then channeling becomes significant. As the lane width increases relative to the width of the plant the channelisation becomes less significant with the less channelised travel causing an ironing out effect more evenly over the area. For straddle carriers stacking containers in long rows, the wheels are restricted to very narrow lanes and consequently severe rutting may take place (Fig. 3.6). In such a case the operation techniques of the plant in that area should be reviewed periodically.

## 3.5 Static loading

Static loads from corner casting feet apply very high stresses to the pavement (Fig. 3.7). These stresses can be taken by the concrete but some superficial damage may occur to the surface. In extreme cases, this damage, in conjunction with frost attack, might be sufficient to introduce structural implications.

*Table 3.2. Table of dynamic load factors ($f_d$). Static loads are increased by the percentage figures in the table*

| Condition | Plant type | $f_d$: % |
|---|---|---|
| Braking | Front lift truck | ±30 |
| | Straddle carrier | ±50 |
| | Side lift truck | ±20 |
| | Tractor and trailer | ±10 |
| Cornering | Front lift truck | 40 |
| | Straddle carrier | 60 |
| | Side lift truck | 30 |
| | Tractor and trailer | 30 |
| Acceleration | Front lift truck | 10 |
| | Straddle carrier | 10 |
| | Side lift truck | 10 |
| | Tractor and trailer | 10 |
| Uneven surface | Front lift truck | 20 |
| | Straddle carrier | 20 |
| | Side lift truck | 20 |
| | Tractor and trailer | 20 |

Note: Where two or three of these conditions apply simultaneously, $f_d$ should take into account multiple dynamic effects. For example, in the case of a front lift truck cornering and accelerating over unneven ground, the dynamic factor is 40% + 10% + 20%, i.e. 70%, so that the static wheel load is increased by 70%. In the case of braking, the dynamic factor is additive for the front wheels and subtractive for rear wheels. In the case of plant with near centrally located wheels (e.g. straddle carriers), braking and accelerating dynamic factors to be applied to the near central wheels are reduced according to geometry.

*Fig. 3.6. When operating within container stacks, a straddle carrier tracks the same length of the slab each pass*

*Fig. 3.7. Failure of concrete slab in vicinity of container corner castings. When the deformation exceeds 12 mm, the containers rest on their underside and the slab load becomes small. This is unacceptable from the structural capacity of the containers*

### 3.5.1 Container corner casting load values

Containers are usually stacked in rows or blocks and, until recently, usually no more than three high, with a maximum of five high. However, in recent times containers have been stacked up to eight high in a few locations and this may become more common. Corner castings measure 178 × 162 mm and frequently they project 12·5 mm below the underside of the container. Table 3.3 gives the maximum loads and stresses for most stacking arrangements. Since it is unlikely that all containers in a stack will be fully laden, the maximum gross weights will be reduced by the amounts shown. The values shown in Table 3.3 can be used directly in the design chart. In the case of empty containers, pavement loads can be calculated on the basis that 40 ft containers weigh 3000 kg and 20 ft containers weigh 2000 kg.

### 3.5.2 Trailer dolly wheels

There are often two pairs of small or 'dolly' wheels on trailers which are 88 mm wide × 225 mm in diameter. When the trailer is parked, the contact area of each wheel is approximately 10 × 88 mm and stresses are 40 N/mm$^2$ (Fig. 3.8). Some trailers have pivot plates which measure 150 × 225 mm and produce contact stresses of 2·0 N/mm$^2$, which is sufficiently low to cause no difficulties within the concrete.

*Table 3.3. Pavement loads from stacking full containers*

| Stacking height | Reduction in gross weight: % | Contact stress: N/mm$^2$ | Load on pavement for each stacking arrangement: kN | | |
|---|---|---|---|---|---|
| | | | Singly | Rows | Blocks |
| 1 | 0 | 2·59 | 76·2 | 152·4 | 304·8 |
| 2 | 10 | 4·67 | 137·2 | 274·3 | 548·6 |
| 3 | 20 | 6·23 | 182·9 | 365·8 | 731·5 |
| 4 | 30 | 7·27 | 213·4 | 426·7 | 853·4 |
| 5 | 40 | 7·78 | 228·6 | 457·2 | 914·4 |
| 6 | 40 | 9·33 | 274·3 | 548·6 | 1097 |
| 7 | 40 | 10·9 | 320·0 | 640·0 | 1280 |
| 8 | 40 | 12·5 | 365·8 | 731·6 | 1463·2 |

*Fig. 3.8. These trailer dolly wheels have indented the bituminous material surfacing*

## 3.6 Wheel proximity factors

The active design constraint is horizontal tensile stress at the underside of the slab in the case of internal and edge loading, and horizontal tensile stress at the surface of the slab in the case of corner loading. If one wheel only is considered, the maximum horizontal tensile stress occurs under the centre of the wheel and reduces with distance from the wheel. If two wheels are sufficiently close together, the stress under each wheel is increased by a certain amount owing to the other wheel. The method described here should be used when the California Bearing Ratio (CBR) of the subgrade is known. In cases where the modulus of subgrade reaction ($K$) is known more accurately than CBR, the proximity factor calculation method presented in Chapter 4 should be used.

Wheel loads are modified by the appropriate proximity factor from Table 9. These factors are obtained as follows. If the wheel proximity were not considered, the relevant

*Table 3.4. Wheel proximity factors*

| Wheel spacing: mm | Proximity factor for effective depth to base of: | | |
|---|---|---|---|
| | 1000 mm | 2000 mm | 3000 mm |
| 300 | 1·82 | 1·95 | 1·98 |
| 600 | 1·47 | 1·82 | 1·91 |
| 900 | 1·19 | 1·65 | 1·82 |
| 1200 | 1·02 | 1·47 | 1·71 |
| 1800 | 1·00 | 1·19 | 1·47 |
| 2400 | 1·00 | 1·02 | 1·27 |
| 3600 | 1·00 | 1·00 | 1·02 |
| 4800 | 1·00 | 1·00 | 1·00 |

stresses would be the radial tensile stress directly beneath the loaded wheel. If there is a second wheel nearby, it generates tangential stress directly below the first wheel. This tangential stress is added to the radial stress contributed by the primary wheel. The proximity factor is the ratio of the sum of these stresses to the radial tensile stress resulting from the primary wheel. The following equations are used to calculate the stress:

$$\boldsymbol{\sigma}_{\mathrm{R}} = \frac{W}{2\pi}\left[\frac{3r^2 z}{\alpha^{5/2}} - \frac{1-2\nu}{\alpha + z\alpha^{1/2}}\right] \tag{3.2}$$

$$\boldsymbol{\sigma}_{\mathrm{T}} = \frac{W}{2\pi}[1-2\nu]\left[\frac{z}{\alpha^{3/2}} - \frac{1}{\alpha + z\alpha^{1/2}}\right] \tag{3.3}$$

where $\boldsymbol{\sigma}_{\mathrm{R}}$ is the radial stress, $\boldsymbol{\sigma}_{\mathrm{T}}$ is the tangential stress, $W$ is the load, $r$ is the horizontal distance between wheels, $z$ is the depth to position of stress calculations, $\nu$ is Poisson's ratio and $\alpha = r^2 + z^2$.

When more than two wheels are in close proximity, the radial stress beneath the critical wheel may have to be increased to account for two or more tangential stress contributions. Table 3.4 shows that the proximity factor depends on the wheel spacing and the effective depth of the slab. The effective depth can be approximated from the following formula and represents the depth of the slab should the slab have been constructed from subgrade material.

$$\text{Effective depth} = 300\sqrt[3]{\frac{35\,000}{CBR \times 10}} \tag{3.4}$$

where *CBR* is the California Bearing Ratio of the subgrade.

As an example, consider a front lift truck with three wheels at each end of the front axle. The critical location is beneath the centre wheel. Suppose a hardstanding were designed on ground with a CBR of 7% and the wheel lateral centres were 600 mm. From the formula, the approximate effective depth of the slab is:

$$\text{Effective depth} = 300\sqrt[3]{\frac{35\,000}{7 \times 10}} = 2381\ \text{mm} \tag{3.5}$$

By linear interpolation from Table 3.4 the proximity factor is 1·86. This should be applied twice for the central wheel. This means that the effective single load is scaled up by 0·86 twice, i.e. $1 + 0{\cdot}86 + 0{\cdot}86 = 2{\cdot}72$. Note that this is approximately 10% less than three so that this type of wheel arrangement effectively reduces slab load by 10%. For wheels bolted side by side where the wheel centres are separated by less than 300 mm, the entire load transmitted to the slab through one end of the axle can be considered to represent the wheel load. An investigation of the actual equivalent wheel load indicates that the actual equivalent wheel load is approximately 1·97 times one wheel load when there are two wheels bolted together at an axle end.

## 3.7 Wheel load calculations for handling plant

The following formulae are for guidance only and relate to plant having wheel configurations as illustrated in the diagrams. In cases where plant has an alternative wheel configuration, the loads can be derived from first principals, following a similar approach. In many cases wheel loads are provided by plant manufacturers and if this is the case, those values should be preferred. For each pass of the plant, a specific spot in the slab is loaded by all of the wheels at one side of the plant. Therefore, in the wheel load calculations, only one side of the plant is considered. In the case of assymetrical plant, the heavier side should be chosen.

### *3.7.1 Front lift trucks and reach stackers (Figs 3.9--3.11)*

In the examples illustrated in Figs 3.9–3.11:

$$\boldsymbol{W}_1 = \boldsymbol{f}_\mathrm{d}\left(\frac{A_1\boldsymbol{W}_\mathrm{c} + B_1}{M}\right) \tag{3.6}$$

$$\boldsymbol{W}_2 = \boldsymbol{f}_\mathrm{d}\left(\frac{A_2\boldsymbol{W}_\mathrm{c} + B_2}{2}\right) \tag{3.7}$$

where $\boldsymbol{W}_1$ is the load on front wheel (kg), $\boldsymbol{W}_2$ is the load on rear wheel (kg), $\boldsymbol{W}_\mathrm{c}$ is the weight of container (kg), $M$ is the number of wheels on front axle (usually 2, 4 or 6), and $f_\mathrm{d}$ is the dynamic factor.

$A_1$, $A_2$, $B_1$ and $B_2$ are given by:

$$A_1 = \frac{-X_2}{X_1 - X_2} \tag{3.8}$$

$$A_2 = \frac{X_1}{X_2 - X_1} \tag{3.9}$$

$$B_1 = \frac{\boldsymbol{W}_\mathrm{T}(X_T - X_2)}{X_1 - X_2} \tag{3.10}$$

$$B_2 = \frac{\boldsymbol{W}_\mathrm{T}(X_T - X_1)}{X_2 - X_1} \tag{3.11}$$

where $X_1$, $X_2$ and $\boldsymbol{W}_\mathrm{T}$ are shown in Fig. 3.11 and $\boldsymbol{W}_\mathrm{T}$ is the self weight of the truck.

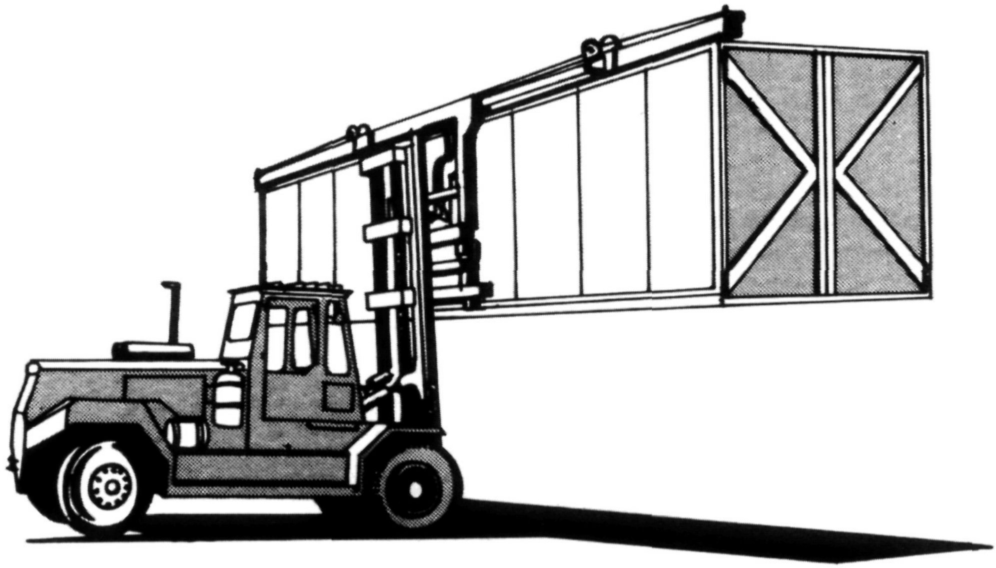


*Fig. 3.9. Front lift truck handling 40ft container*

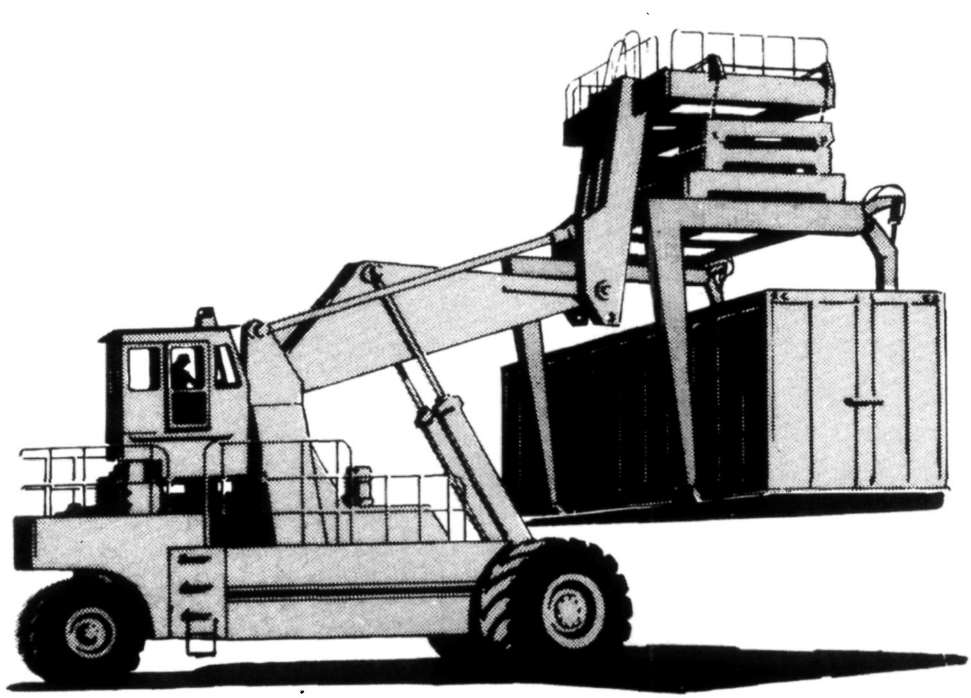


*Fig. 3.10. Reach stacker handling 40ft container*

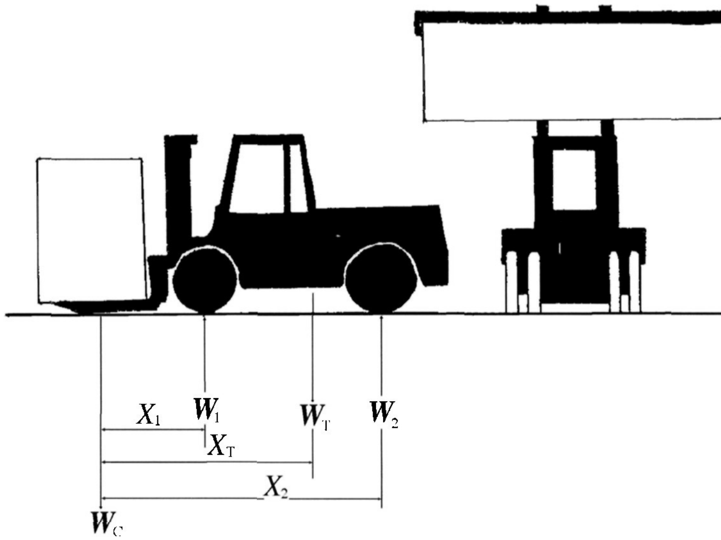



*Fig. 3.11. Dimensions and weights used in wheel load calculations*

### 3.7.2 Straddle carriers (Figs 3.12--3.14)

For straddle carriers:

$$W_i = f_d\left[U_i + \frac{W_c}{M}\right] \tag{3.12}$$

where $W_i$ is the wheel load of laden plant (kg), $U_i$ is the wheel load of unladen plant (kg), $W_c$ is the weight of container (kg), $M$ is the total number of wheels on plant, and $f_d$ is the dynamic factor.

### 3.7.3 Side lift trucks (Fig. 3.15)

Equation (3.12) is used:

$$W_i = f_d\left[U_i + \frac{W_c}{M}\right]$$

where the parameters are as defined previously.

### 3.7.4 Yard gantry cranes (Figs 3.16 and 3.17)

In the case of gantry cranes:

$$W_1 = f_d\left[U_1 + \frac{A_1 W_c}{M}\right] \tag{3.13}$$

$$W_2 = f_d\left[U_2 + \frac{A_2 W_c}{M}\right] \tag{3.14}$$

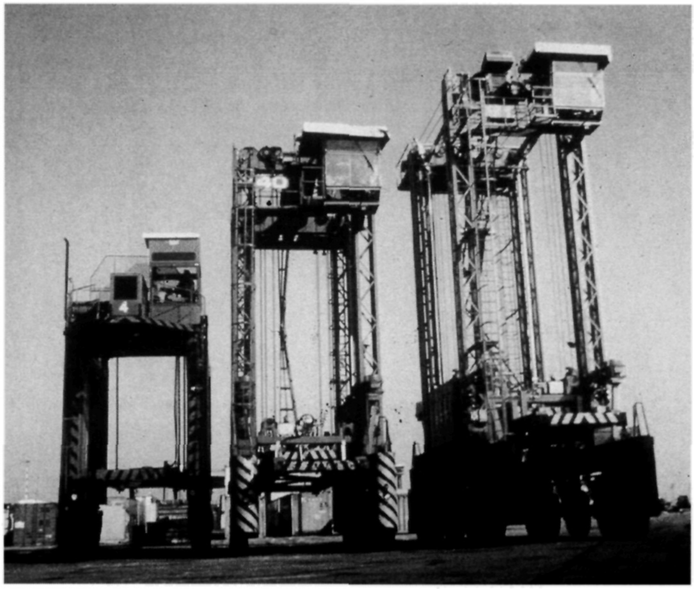


*Fig. 3.12. Three generations of straddle carriers at Europe Container Terminus, Rotterdam. The one on the left can place a container over another. The one in the centre can place a container over two others and the one on the right can place a container over three others. This evolution took place during the 1970s and the early 1980s*

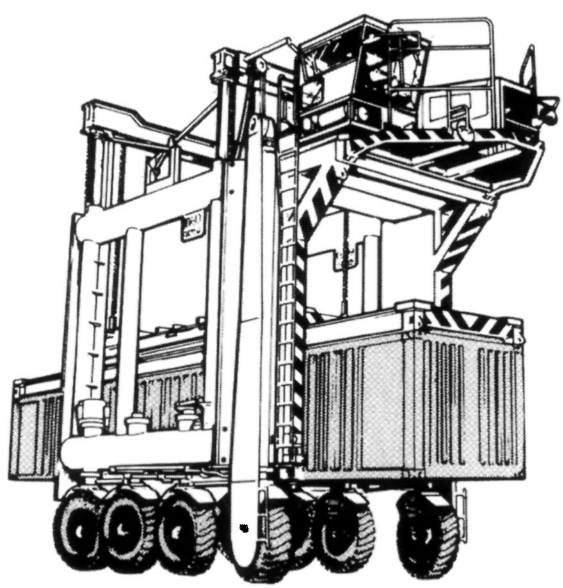


*Fig. 3.13. Eight wheel asymmetric straddle carrier handling 40ft container*

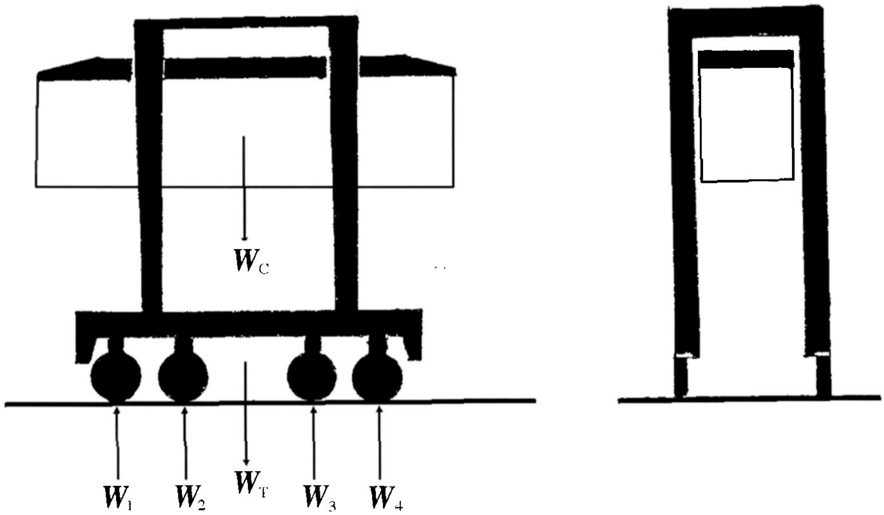



*Fig. 3.14. Dimensions and weights used in wheel load calculations*

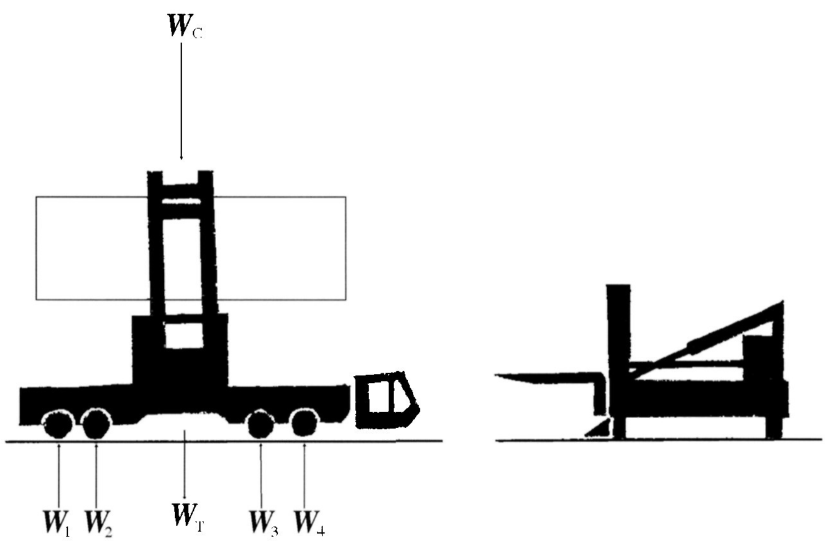



*Fig. 3.15. Dimensions and weights used in wheel load calculations*

where $\boldsymbol{W}_1$ is the wheel load on side 1 (kg), $\boldsymbol{W}_2$ is the wheel load on side 2 (kg), $\boldsymbol{W}_c$ is the weight of container (kg), $M$ is the number of wheels on each side (possibly 2), $f_d$ is the dynamic factor, $A_1 = 1 - (X_c/X_2)$, $A_2 = X_c/X_2$, $U_1$ is the unladen weight of gantry crane on each wheel of side 1 (kg), $U_2$ is the unladen weight of gantry crane on each wheel of side 2 (kg), and $X_2$ and $X_c$ are shown in Fig. 3.17.

Note. The front and rear wheels may have different unladen loads. This is taken into account by using the equation for both wheels on each side with their respective $f_d$ values.

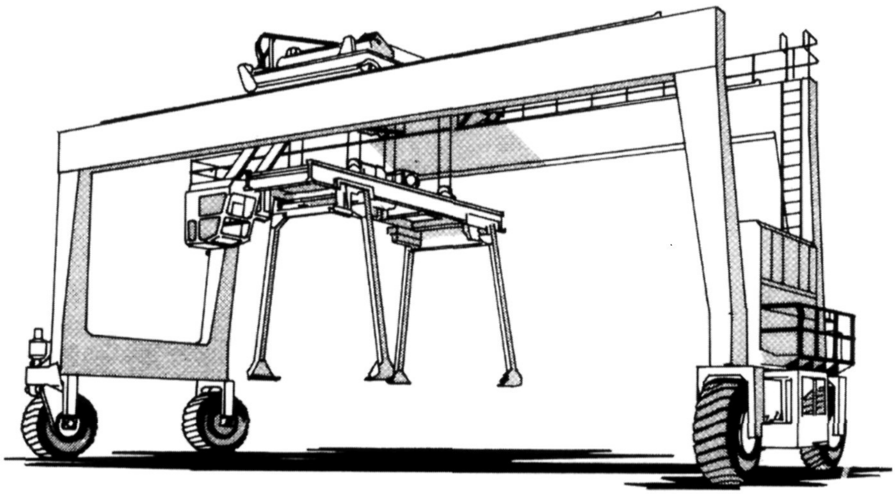


*Fig. 3.16. Rubber tyred gantry crane (RTG). Individual wheel loads can exceed 50 t*

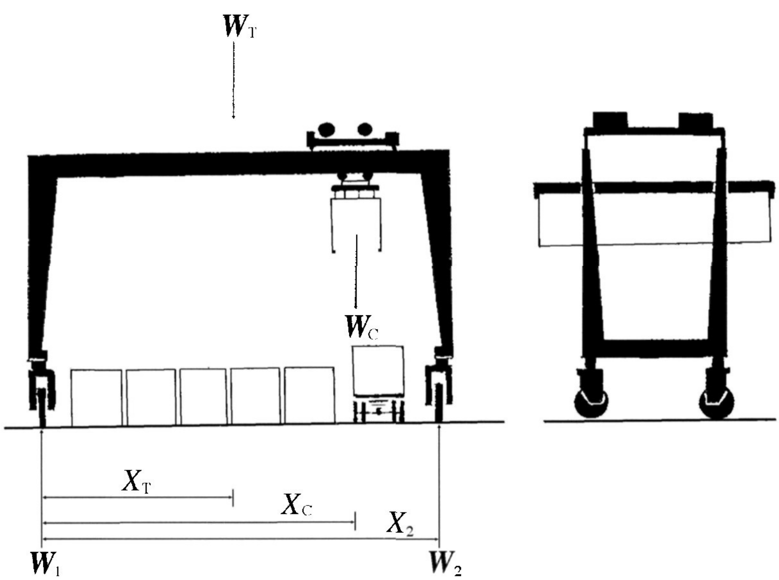



*Fig. 3.17. Dimensions and weights used in wheel load calculations*

### 3.7.5 Tractor and trailer systems (Figs 3.18 and 3.19)

In this example:

$$W_1 = f_d\left[U_1 + \frac{W_c[1-A][1-B]}{M_1}\right] \tag{3.15}$$

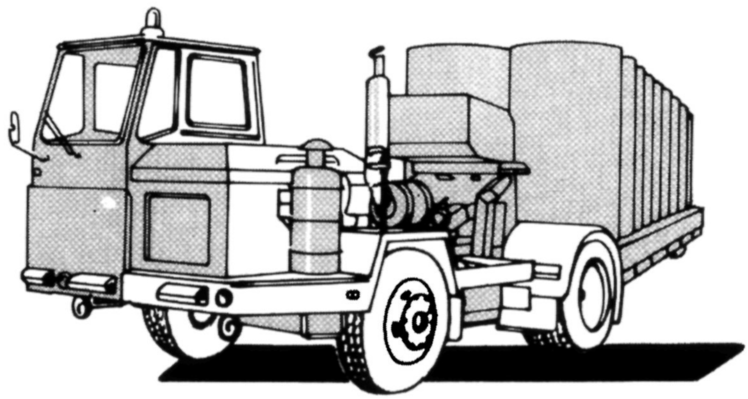


*Fig. 3.18. In some places, specialised off-highway tractor units are used to marshall specially developed trailers. In this case, a special small wheeled trailer is used to transport containers by sea. The small wheels allow the trailer to enter low headroom decks on board ships*

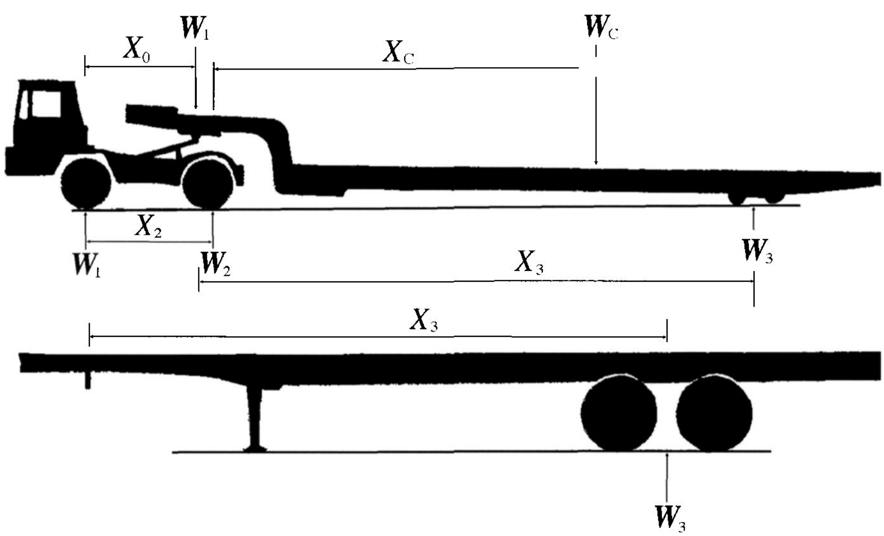



*Fig. 3.19. Dimensions and weights used in wheel load calculations*

$$W_2 = f_d\left[U_2 + \frac{W_c[1 - A]B}{M_2}\right] \tag{3.16}$$

$$W_3 = f_d\left[U_3 + \frac{W_c A}{M_3}\right] \tag{3.17}$$

where $W_1$ is the load on front wheels of tractor (kg), $W_2$ is the load on rear wheels of tractor, $W_3$ is the load on trailer wheels (kg), $W_c$ is the weight of container (or load) (kg), $M_1$ is the number of front wheels on tractor, $M_2$ is the number of rear wheels on tractor, $M_3$ is the number of wheels on trailer, $U_1$ is the load on front wheels of tractor – unladen (kg), $U_2$ is the load on rear wheels of tractor – unladen (kg), $U_3$ is the load on trailer wheels — unladen (kg), $f_d$ is the dynamic factor, $A = X_c/X_3$, $B = X_b/X_2$, $X_c$, $X_b$, $X_3$ and $X_2$ are shown in Fig. 3.19.

### *3.7.6 Mobile cranes (unladen) (Fig. 3.20)*

For the case of a mobile crane, the wheel load is:

$$W = W_T/M \tag{3.18}$$

where $W_T$ is the self weight of the crane and $M$ is the total number of wheels on the crane.

## 3.8 Loading considerations for industrial floors

Loading of industrial floors will be one of, or a combination of, the following:

- Uniform Distributed Load (UDL)
- Live Load (LL)
- Point Load (PL)
- Contact Pressure (CP)
- Horizontal Load (HL).

Floor design comprises assessing the loading which the floor is predicted to sustain and selecting materials, thicknesses and a joint configuration that will sustain those loads while at the same time satisfying flatness, durability, abrasion and riding quality requirements. An essential part of the design process is the assessment of the

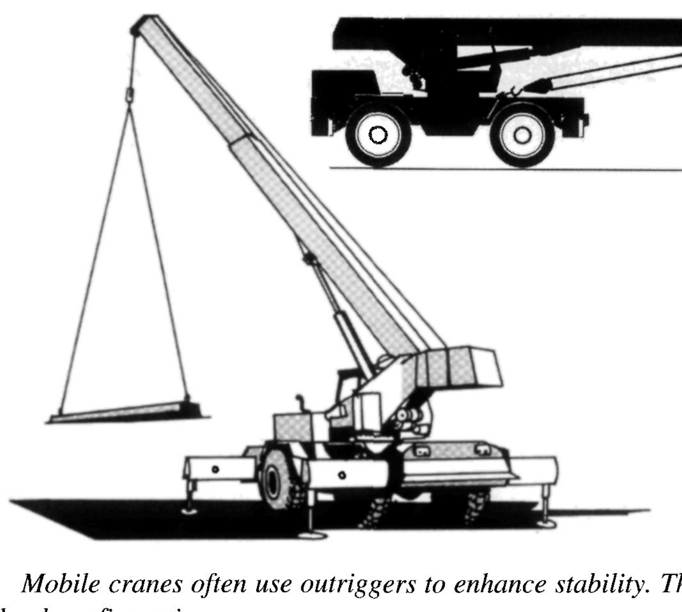


*Fig. 3.20. Mobile cranes often use outriggers to enhance stability. This can constitute a critical load configuration*

anticipated load regime. Frequently, the end use of the floor will be uncertain at the time of floor design and construction. In such cases, it is common for a distributed load and a point load to be specified and for all of the floor to be capable of withstanding the selected values. Horizontal load may be introduced into the floor if the building frame relies upon the floor as a ground level tie or if vehicles undertake turning manoeuvres. In such cases, special care is needed when detailing joints which must be able to transmit tension between neighbouring bays. In the case of tied portals, the value of the horizontal load can be calculated by undertaking an analysis of the building frame, taking the horizontal reaction at the foot of each structural column and distributing the load into the slab. It may be necessary to detail reinforcing bars radiating from column feet to ensure that the true behaviour of the horizontal forces approximates closely to the design assumptions. A common category of loading comprises leg loads applied by storage systems or mezzanine floor columns. Different types of storage systems are considered explicitly. Often, floors used for storage require particularly tight tolerances.

Very Narrow Aisle (VNA) high bay racking systems are a special case and are dealt with in Section 3.20.

## 3.9 Unit pallet racking

For warehouse storage, the standard Euro Pallet (0·8 m × 1·0 m) has a loading range of 5–20 kN, with a storage height of between 1·2 m and 1·4 m (see Fig. 3.21).

Some multi-national storage companies have designed alternative pallets to the standard Euro Pallet. The modified pallet tends to be longer owing to the ease of storing larger products (Fig. 3.22). The only dimensional change to the modified pallet is the

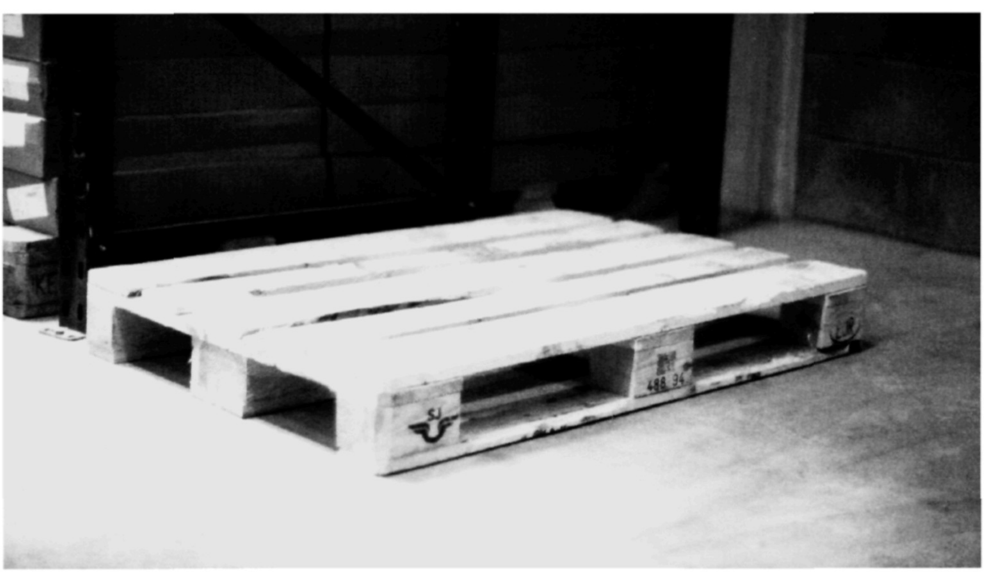


*Fig. 3.21. The standard Euro Pallet*

*Fig. 3.22 Comparison between a modified pallet and a Euro Pallet*

length since all of the pallets have to be transported on the same vehicles for economic reasons.

## 3.10 Block stacking

Block stacking (Fig. 3.23) is usually limited to the width of aisles, size of the fork lift and the actual weight of the pallet plus load. The stacking process is based upon placing pallet loads directly on top of each other (materials ranging from paper to steel and where handling is not of great importance). The maximum height is around 9·5 m. Settlement can take place to a greater extent than with other loading regimes. Uplift can be a major problem and can be seen especially if there are mid-aisle joints — hence the monitoring of unit pallet load stacking is recommended.

## 3.11 Pallet storage racking

Racking (Fig. 3.24) allows storage to a greater height and facilitates access to intermediate pallets. Adjustable pallet racking is the common design used and comprises a steel framework where the pallets rest on steel beams. The racks are often placed back to back for maximum access for the fork lifts. The height of the warehouse always dictates the number of levels of racking with adequate operating clearance for each individual pallet. The aisle widths vary from 2·7 m (Fig. 3.25) for free ranging fork lift trucks to 1·2 m (Fig. 3.26) for narrow aisle trucks. Figure 3.27 illustrates a racking fixing foot.

## 3.12 Pantograph racking

Pantograph racking (Fig. 3.28) is similar to standard storage racking but with extra horizontal depth. Pantograph racking is two pallets deep giving a greater density of storage and a greater concentration of point loads.

## 3.13 Raised storage platforms (mezzanine floors)

Mezzanine floors are classed as 'free-standing steel floors' which give extra storage and optimise the use of headroom above ground level. Construction is often in a grid formation

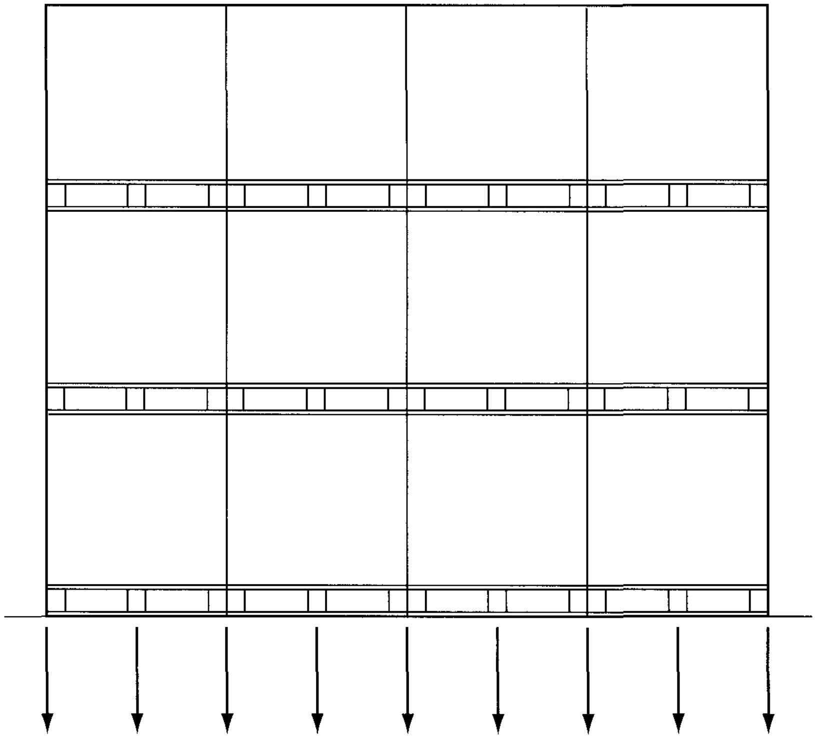


*Fig. 3.23. Typical block stacking*

(approximately 4 × 3 m) supported by steel posts each having typical leg loads of between 30 and 75 kN. Loads greater than 75 kN are usually taken down to a foundation beneath the floor.

## 3.14 Mobile pallet racking

Mobile pallet racking comprises rows of racking mounted on rails which are driven by an electric motor. Usually limited to 6 m high, the system is very efficient in the use of floor space (approximately 80%) and every pallet can be accessed by moving the racks along the rails. Being mounted on rails, this racking system often applies large line loads of the magnitude of 23 kN/m.

## 3.15 Live storage systems

Live storage racking (Fig. 3.29) comprises a last-in first-out rotation storage system which operates by placing a pallet at one end of the racking which is on a downward conveyor. Gravity moves the pallets to the opposite end of the system to the output location where distribution takes place.

## 3.16 Drive-in racking

Each pallet is supported on cantilever brackets that are bolted to the racking steelwork. This system has no permanent access aisles. Depending on the closeness of the legs,

*Fig. 3.24. Typical pallet storage racking*

*Fig. 3.25. 2·7 m wide storage racking*


*Fig. 3.26. 1·2 m wide storage racking system. There are two fixing feet per bay (except at end) where the whole load is transmitted to the floor*

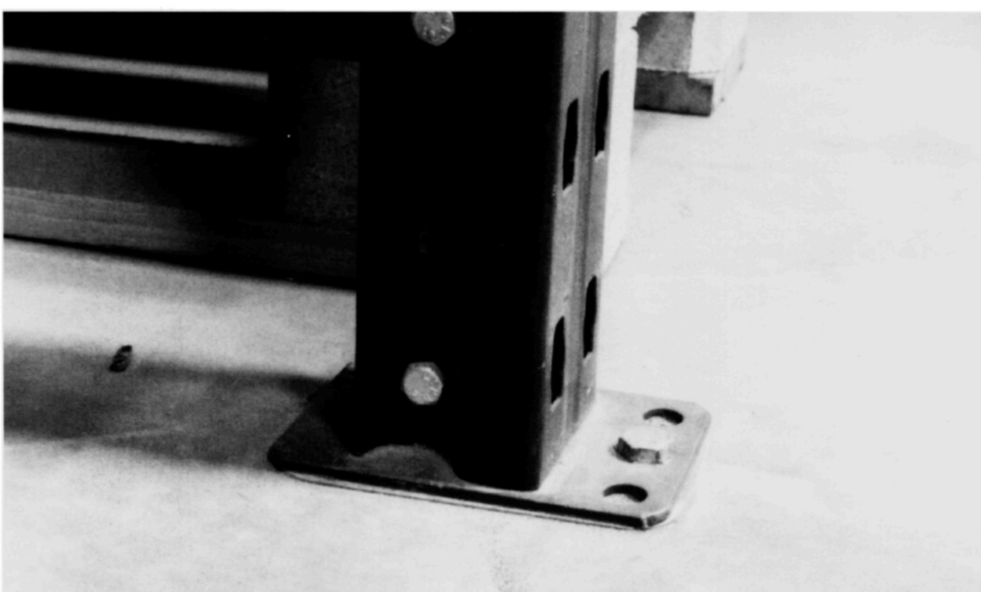


*Fig. 3.27. Fixing racking foot. If the racking is high the loading on each leg can be substantial. The load may exceed 300 kN but the critical stress beneath the legs has no bearing on the width of the aisle. To prevent damage to concrete floors and to help distribute the loading, shims and footplates are often specified with racking systems*

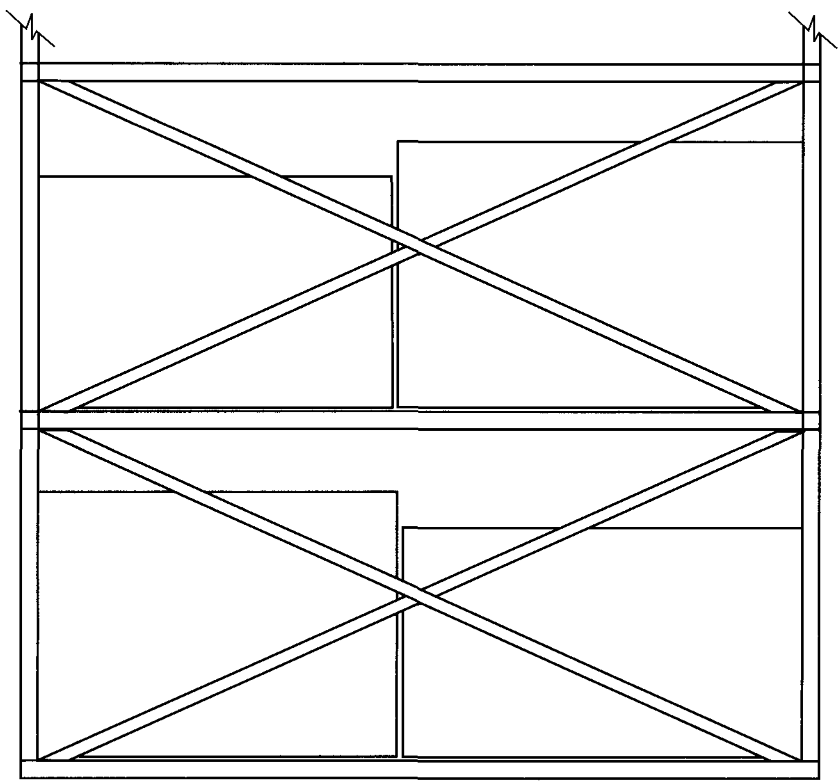


*Fig. 3.28. Pantograph racking*

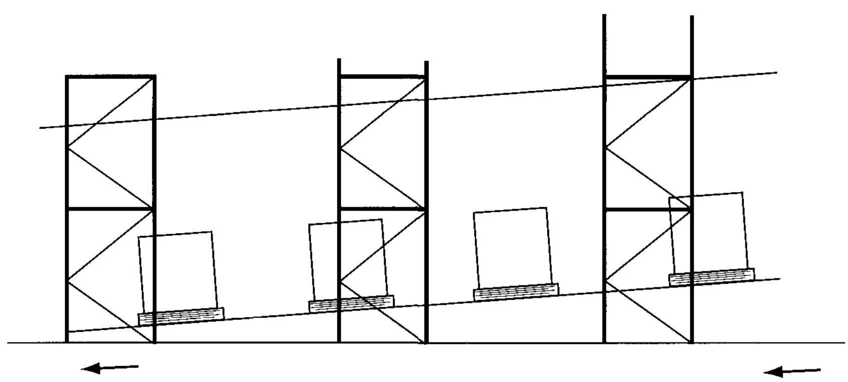


*Fig. 3.29. Live storage racking*

with a 20 kN pallet the maximum load on the racking is 40 kN leg loads for five storeys high.

## 3.17 Shelving

Shelving systems store smaller products (Fig. 3.30). The shelves are often of uniform cross-section and are placed back to back which results in concentrated leg loads.

*Fig. 3.30. Typical shelving system*

## 3.18 Mobile handling equipment

Mobile transporters are required with many storage systems and they impose loads through the contact areas of their tyres. Pallet transporters (Fig. 3.31) comprise hand-pushed trailers carrying a maximum load of 20 kN on very small wheels with an average contact pressure of 9 N/mm$^2$.

Counterbalanced fork lift trucks (Fig. 3.32) are front loading telescopic mast vehicles with an average load of 30 kN. Height is usually limited to 8·5 m for stability reasons and the maximum wheel load occurs from the rear wheels when the truck is unladen.

Reach trucks (Fig. 3.33) have a moving telescopic mast or pantographic load extender and often operate in very narrow aisles with a maximum load of 30 kN and height limited to 8·5 m.

Within very narrow aisles, rubber stoppers line the aisle to stop the Very Narrow Aisle (VNA) truck hitting and damaging the products already in storage. The sideloader reach truck has a telescopic mast at right angles to the driver's cabin. The VNA trucks can transport pallet loads in any direction because of their rotating traversing mast. These trucks can be wire or rail guided but usually they move freely. Within a 1·3 m wide aisle a load of 20 kN can be lifted up to a maximum 12 m. Order pickers carry only empty pallets to a height of 9·5 m where the operator can stack by hand. Stacker cranes run on rails bolted to the floor slab. This system gives a possible lifting height of 30 m.

## 3.19 Methods of specifying floor loading

### *3.19.1 Contact pressure*

Note that rack footplates, as shown in Figure 3.34, are designed to accommodate the fixing bolts rather than to distribute the load so the nominal floor contact area is relatively small.

*Fig. 3.31. Pallet transporter*


*Fig. 3.32. Counterbalance fork lift truck*

*Fig. 3.33. Reach truck*

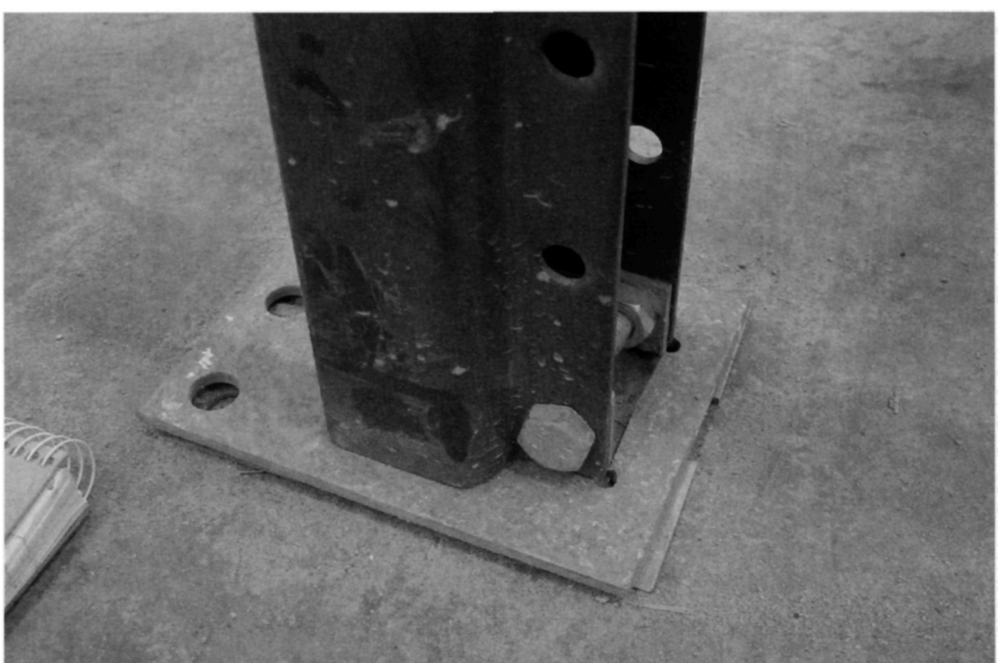


*Fig. 3.34. Typical footplate with fixing bolt. Note the packing material beneath the plate which accommodates errors in floor flatness*

### 3.19.2 Average contact pressure

From TR 34,[3.1] contact pressure values (N/mm$^2$) for typical storage systems and equipment are:

- 30 kN Counterbalance lift truck 2.4
- 20 kN Reach truck 5.6
- 20 kN Pallet truck 11.0

### 3.19.3 Horizontal loads

Horizontal loads applied by the braking of mobile materials handling equipment are usually ignored. When the building frame uses the floor as a tie, it may be necessary to consider the tie force, both in the structural design of the floor slab and in the detailing of joints. In large floors, it may be that tie forces will be dissipated into the sub-base near to the perimeter of the floor. The transfer of horizontal tie forces into the sub-base can be assessed by considering the vertical force applied by the self-weight of the floor slab and applying a coefficient of friction according to whether a slip membrane is present.

### 3.19.4 Loading table

In the design of concrete formless floors, loading is a major consideration and Table 3.5 can be used to assess loads in situations where no further information is available.

## 3.20 High bay racking case study

A common storage system comprises Very Narrow Aisle (VNA) high bay racking systems. Such systems introduce special requirements in terms of floor flatness and leg loads. Figures 3.35–3.45 illustrate the installation and use of typical installations.

*Table 3.5. Load classification*

| Load classification | Typical load: kN | Usual type of loading |
|---|---|---|
| Light | 42 | Pallet racking<br>Mezzanine floor<br>Shelving<br>Fork lift |
| Medium | 60 | Pallet racking<br>Mezzanine floor<br>Shelving<br>Fork lift |
| Heavy | 87 | Pallet racking<br>Mezzanine floor |
| Very heavy | 114 | Pallet racking<br>Mezzanine floor |

*Fig. 3.35. This back-to-back high bay racking system occupies the full height of the building. It was installed in Scotland in 2002 in a distribution warehouse. Note how the system is built around the building's portal prop column. Although a loading bay is lost, the reduction in the cost of the structural steelwork makes this solution cost effective. Stability is provided by moment connections between the horizontal beams and the uprights*

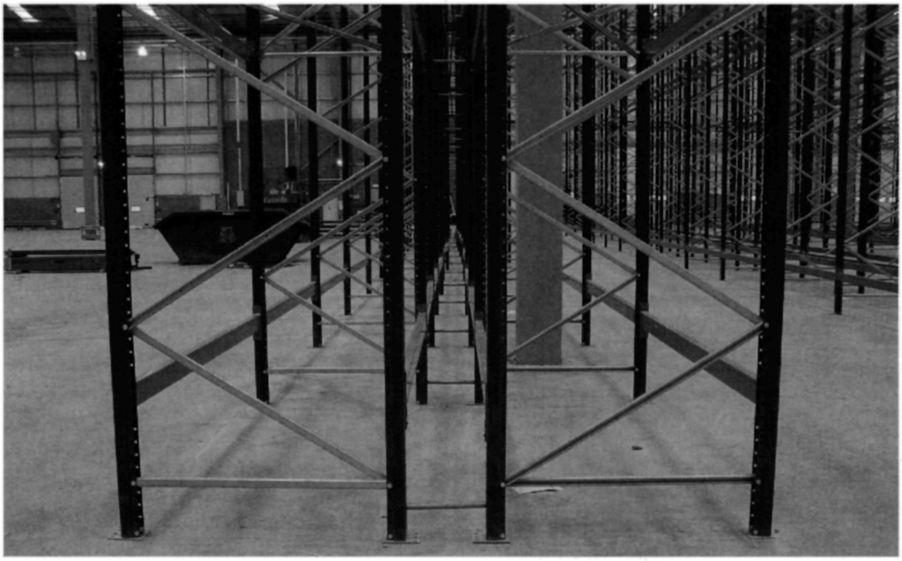


*Fig. 3.36. Notice how the diagonal members provide strength and stability to this racking system in the plane shown. From the floor's perspective, the maximum load condition occurs at the intermediate supports where two legs are located close together. Notice also the structural column in the right-hand second bay. Also, note that this system is designed such that all the goods are stored on the racks and none are stored on the floor, unlike the system shown in Figs 3.39–3.45 where the lower pallets rest directly on the floor*

*Fig. 3.37. The floor between the racks needs to be particularly flat. Any deviation from flatness will be amplified by the height of the materials handling trucks and may lead to damage to the racks and to lost production time. Tables 5.9 and 5.10 deal with floor construction tolerances*

*Fig. 3.38. The flatness of the floor beneath the racks is less critical than it is between the racks. Errors in floor level within the racks can be dealt with by providing shims between the steel base plates and the concrete, as shown in the case of the right-hand base plate*

*Fig. 3.39. The goods are stored on timber pallets which are positioned on the racks and on the floor beneath the racks by the VNA truck shown. The VNA truck moves goods between the racks and the edge of the racking system where they are then handled by fork lift trucks*


*Fig. 3.40. This VNA truck is shown in its lowered position. Note the solid rubber tyred wheels. The truck is steered by the operator when travelling over an open area of floor but uses a wire guidance system inserted in the floor when operating between racks*

*Fig. 3.41. The darker tyre tracks show how little clearance is required by a VNA truck. The line running along the centre of the aisle is where the guide wire is located. In this system, the lower pallets are stored directly on the floor, so introducing a distributed load in addition to the rack leg loads*

*Fig. 3.42. The importance of the accuracy of the floor surface can be appreciated here. Deviation from flatness will be magnified and the truck may collide with the racking. The operator's cab is raised to the level of the material being accessed — see Tables 5.9 and 5.10*

*Fig. 3.43. VNA systems may include ancillary equipment such as these containers that have solid plastic wheels. While the wheel loads are not excessive, such equipment requires well constructed joints. These containers are wheeled into highway delivery vehicles where they are then locked in place*

*Fig. 3.44. VNA trucks place pallets of goods beside racks where they are taken to highway delivery vehicles by manual pallet trucks, such as the one shown, or by mororised electric or diesel fork lift trucks. Manually operated pallet trucks have solid plastic wheels which can damage joints*


*Fig. 3.45. Specially constructed docking bays allow weatherproof transfer of goods between the racking system and the delivery vehicles. Goods are handled by fork lift trucks with solid rubber tyres*

# 4 Design

## 4.1 Introduction

A series of ground bearing slab design methods is presented in this chapter. The methods are all similar in that they model the loading regime numerically, they assign elastic properties to the ground and they use the resulting information to produce data that allows a slab to be understood and installed. They differ in the way in which they transform the loading regime and ground properties into design output data. The appropriate design procedure depends upon the nature of the loading, upon the engineering properties of the materials from which the slab is to be constructed and upon the end use of the slab. This chapter considers slabs that are truly ground bearing and slabs that are fully or partially supported by piles.

The design methods described in this chapter are as follows.

1. Using Hetenyi's equations for a slab subjected to a uniformly distributed load over part of its area.
2. Using Westergaard's equations for patch loads.
3. Using Meyerhof's equations for patch loads.
4. Using the Eurocode 2 method for punching shear.
5. Using design charts developed from Westergaard's equations for a slab subjected to patch loads.
6. Using a design chart developed from finite element analysis for patch loads.
7. Preliminary design method.
8. Using equilibrium to assess the effect of restraint to a slab contracting.
9. Using equilibrium to assess the effect of restraint to a slab curling or hogging.
10. Using a yield line analysis for a slab supported by edge beams and piles.

Table 4.1 shows the situations where each of the ten design methods is appropriate.

In all of the design methods, concrete characteristic flexural strength is used as the basis of stress assessment and load induced stresses are computed using classical methods. See Table 1.1 for concrete strength values.

*Table 4.1. Design methods reviewed in this chapter*

| Design method | Type of slab | Type of load | Output |
|---|---|---|---|
| Hetenyi (Section 4.6) | Floor slabs | Distributed | Slab thickness |
| Modified Westergaard (Section 4.7) | All ground bearing slabs | Patch load | Slab thickness and elastic deflexion |
| Meyerhof (Section 4.8) | Floor slab where cracking is acceptable | Patch load/ point load | Floor slab thickness |
| Eurocode 2 (Section 4.9) | All ground bearing slabs | Patch load | Confirmation that slab will not suffer shear failure |
| Westergaard design charts (Section 4.10) | All ground bearing slabs | Patch load/ point load | Slab thickness |
| Finite element design chart (Section 4.11) | External slabs | Patch load/ point load | Slab thickness |
| Preliminary design method (Section 4.12) | All ground bearing slabs | Patch load | Slab thickness |
| Restraint to contracting (Section 4.13) | All ground bearing slabs | Stress generated by restraint to shrinking | Additional stresses so generated |
| Restraint to curling and hogging (Section 4.14) | All ground bearing slabs | Stress generated by restraint to curling | Additional stresses so generated |
| Yield line analysis of pile supported slabs (Section 4.15) | Moderately loaded slabs supported on piles | Uniformly distributed load | Slab thickness and reinforcement details |

The following factors have to be taken into account in ground floor design:

- loading regime
- strength of concrete
- strength of existing ground and effect of the sub-base.

## 4.2 Loading regime

Loading has been described in Chapter 3. It may be necessary to increase loads above their static values to account for vehicle dynamics, channelisation (all plant or vehicles moving over the same point in the pavement/floor) and repetition. In the case of point or patch loads, the position of the load relative to the slab edge is critical and three alternative cases may need to be considered, namely internal loading (greater than 0·5 m from edge slab), edge loading and corner loading. With internal or edge loading the maximum stress occurs beneath the heaviest load at the underside of the slab. Corner loading creates tensile stress at the upper surface of the slab a distance away from the corner. This distance can be calculated from:

$$d = 2[(2^{0.5})rl]^{0.5} \tag{4.1}$$

where $r$ is the radius of loaded area (mm), $l$ is the radius of relative stiffness (mm), and $d$ is the distance from slab corner to position of maximum tensile stress (mm).

## 4.3 Strength of concrete

Concrete slabs are frequently constructed from C30, C35 or C40 concrete with a minimum cement content of 300 kg/m$^3$ with a slump of 50 mm or less. Design is based upon comparing concrete characteristic flexural strength with calculated flexural stresses, whereas specification is by characteristic compressive strength. Table 4.2 shows flexural strength values for a range of commonly used concretes.

## 4.4 Strength of existing ground and effect on sub-base

The design methods require a value for the modulus of subgrade reaction ($K$) which defines the stiffness of the material beneath the slab. The following four values of $K$ are used in the design procedures:

- $K = 0{\cdot}013$ N/mm$^3$, very poor ground
- $K = 0{\cdot}027$ N/mm$^3$, poor ground
- $K = 0{\cdot}054$ N/mm$^3$, good ground
- $K = 0{\cdot}082$ N/mm$^3$, very good ground (no sub-base needed).

A subgrade with a $K$ value of 0·027 N/mm$^3$ will deflect vertically by 1 mm when a vertical pressure of 0·027 N/mm$^2$ or 27 kN/m$^2$ is applied through a standard disk. The beneficial effect of a granular sub-base is taken into account by increasing $K$ according to the thickness and strength of the sub-base, as shown in Table 1.5, Section 1.2.3.

## 4.5 Stress in concrete

The stress in a floor slab depends upon:

- the properties of the subgrade
- the loading regime:
  - uniformly distributed load (UDL)
  - patch/point loads
- the thickness of the floor slab
- the strength of the sub-base.

*Table 4.2. Mean and characteristic 28 day flexural strength values for various concrete mixes — see Table 1.1 for other properties of concrete*

| | Flexural Strength: N/mm² | |
|---|---|---|
| | Mean | Characteristic |
| Plain C30 concrete | 2·0 | 1·8 |
| Micro-silica C30 concrete | 2·4 | 1·9 |
| C30 concrete 20 kg/m³ ZC 60/1·00 steel fibre[a] | 2·8 | 2·0 |
| C30 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 3·2 | 2·2 |
| C30 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 3·8 | 2·7 |
| Plain C35 concrete | 2·2 | 1·95 |
| Micro-silica C35 concrete | 2.6 | 2.0 |
| C35 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·0 | 2·1 |
| C35 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 3·4 | 2·3 |
| C35 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 4·0 | 2·9 |
| Plain C40 Concrete | 2·4 | 2·1 |
| Micro-silica C40 Concrete | 2·8 | 2·15 |
| C40 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·2 | 2·2 |
| C40 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 3·6 | 2·5 |
| C40 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 4·2 | 3·2 |
| Plain C45 Concrete | 2·7 | 2·3 |
| Micro-silica C45 Concrete | 3·1 | 2·4 |
| C45 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·5 | 2·5 |
| C45 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 4·0 | 2·8 |
| C45 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 4·8 | 3·6 |
| Plain C50 Concrete | 3·0 | 2·5 |
| Micro-silica C50 Concrete | 3·4 | 2·6 |
| C50 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·8 | 2·7 |
| C50 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 4·2 | 3·0 |
| C50 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 5·0 | 3·8 |
| Plain C55 Concrete | 3·4 | 2·7 |
| Micro-silica C55 Concrete | 3·8 | 2·8 |
| C55 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 4·2 | 2·9 |
| C55 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 4·6 | 3·2 |
| C55 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 5·4 | 4·1 |

[a] ZC 60/1·00 refers to a commonly used anchored bright wire fibre of total length 60 mm and wire diameter 1·00 mm.

## 4.6 Hetenyi design method for uniformly distributed loading

The common loading system comprises alternate unloaded aisles and loaded storage areas. The maximum negative bending moment (hogging) occurs within the centre of the aisles and is given by:

$$M_{\text{hog}} = \frac{q}{s\lambda^2}(B_{\lambda a'} - B_{\lambda b'}) \tag{4.2}$$

where $a$ is the width of the aisle, $b$ is the width of the loaded area, $a' = a/2$, $b' = a/2 + b$, $q$ is the UDL (characteristic load × load factor, in N/mm²), $\lambda = (3K/Eh)^{1/4}$, $K$ is the modulus of subgrade reaction, $E$ is the concrete modulus (10 000 N/mm$^2$ for sustained load), and $h$ is the slab thickness.

The maximum positive bending moment (sagging) occurs beneath the centre of the loaded area (loading from adjacent blocks is ignored):

$$M_{sag} = \boldsymbol{q}/2\lambda^2 B_{(1/2)\lambda b} \tag{4.3}$$

where $B_x = e^{-x} \sin x$.

The two moment equations can be simplified into a single conservative equation for any combination of aisle width and stacking zone width:

$$M_{max} = -0{\cdot}168\boldsymbol{q}/\lambda^2 \tag{4.4}$$

The corresponding maximum flexural stress is given by:

$$\sigma_{max} = 6M_{max}/h^2 \tag{4.5}$$

$$\sigma_{max} = 1008\boldsymbol{q}/(\lambda^2 h^2)(\mathrm{N/mm^2}) \tag{4.6}$$

where the maximum flexural strength cannot exceed the relevant characteristic value from Table 4.2. To ease calculation, Table 4.3 shows values of $\lambda^2 h^2$ for common combinations of modulus of subgrade reaction ($K$) and slab thickness.

### 4.6.1 Uniformly distributed load example

Consider a 150 mm thick floor slab carrying a uniformly distributed load of 50 kN/m$^2$ between the aisles on poor ground.

$$\sigma_{max} = \frac{1{\cdot}008\boldsymbol{q}}{\lambda^2 h^2}\ \mathrm{N/mm^2}$$

$$\boldsymbol{q} = 0{\cdot}05\ \mathrm{N/mm^2} \times 2 = 0{\cdot}10\ \mathrm{N/mm^2}$$

(In the above equation, 2 is the fatigue factor. A value of 2 means a load of 0·05 N/mm$^2$ can be repeated indefinitely.)

For poor ground $K = 0{\cdot}027\ \mathrm{N/mm^3}$.

Therefore:

$$\sigma_{max} = 1{\cdot}008\left(\frac{0{\cdot}10}{0{\cdot}035}\right) = 2{\cdot}88\ \mathrm{N/mm^2} \tag{4.7}$$

*Table 4.3. Values of $\lambda^2 h^2$ for combinations of slab thickness and modulus of subgrade reaction*

| Modulus of subgrade reaction $K$: N/mm$^3$ | $\lambda^2 h^2$ for slab thickness (mm): | | | | |
|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 |
| 0·082 | 0·061 | 0·066 | 0·070 | 0·074 | 0·078 |
| 0·054 | 0·049 | 0·053 | 0·057 | 0·060 | 0·063 |
| 0·027 | 0·035 | 0·038 | 0·040 | 0·043 | 0·045 |
| 0·013 | 0·024 | 0·026 | 0·028 | 0·029 | 0·031 |

From the characteristic strength values in Table 4.2, this stress can be withstood by a C40 concrete incorporating 40 kg/m$^2$ of 60 mm long 1 mm diameter anchored steel fibre. This design solution can be used for any combination of aisle and stacking zone width.

## 4.7 Modified Westergaard design method for patch loads

Highway vehicles and handling equipment apply loads to the surface of a floor as *patch loads*. In most cases, sufficient accuracy is gained by assuming the patch to be circular and to apply uniform stress throughout the patch. These assumptions lead to minor errors — the contact patch shape in the case of a commercial vehicle is nearer to a rectangle than to a circle but to assume accurate shaped patch loads would preclude the use of Westergaard equations. The error in assuming constant stress circular loading is very small and is conservative, i.e. true analysis would lead to a slightly thinner floor. The maximum flexural tensile stress occurs at the bottom (or top, in the case of corner loading) of the slab under the heaviest wheel load. The maximum stress under a patch load can be calculated by the elastic Westergaard[4.1] and Timoshenko equations, sometimes referred to as the modified Westergaard equations, as follows.

(*a*) Patch load in mid-slab (i.e. more than 0·5m from slab edge):

$$\boldsymbol{\sigma}_{\max} = \frac{0{\cdot}275(1+\mu)}{h^2}\boldsymbol{P}\log\left(\frac{0{\cdot}36Eh^3}{Kb^4}\right) \tag{4.8}$$

(*b*) Patch load at edge of slab:

$$\boldsymbol{\sigma}_{\max} = 0{\cdot}529(1+0{\cdot}54\mu)\frac{\boldsymbol{P}}{h^2}\log\left(\frac{0{\cdot}20Eh^3}{Kb^4}\right) \tag{4.9}$$

(*c*) Patch load at slab corner:

$$\boldsymbol{\sigma}_{\max} = \frac{3\boldsymbol{P}}{h^2}\left(1-\left(\frac{1{\cdot}41b}{\left(\frac{Eh^3}{12(1-\mu^2)K}\right)^{0{\cdot}25}}\right)^{0{\cdot}6}\right) \tag{4.10}$$

where

| | | |
|---|---|---|
| $\boldsymbol{\sigma}_{\max}$ | | is the flexural stress (N/mm$^2$), |
| $\boldsymbol{P}$ | = | point load (N) (i.e. characteristic wheel load × fatigue factor) |
| $\mu$ | = | Poisson's ratio, usually 0·15 |
| $h$ | = | slab thickness (mm) |
| $E$ | = | elastic modulus, usually 20 000 N/mm$^2$ |
| $K$ | = | modulus of subgrade reaction (N/mm$^3$) |
| $b$ | = | radius of tyre contact zone (mm) |
| | = | $(\boldsymbol{P}/\pi\boldsymbol{p})^{1/2}$ |
| $\boldsymbol{p}$ | = | contact stress between wheel and floor (N/mm$^2$). |

Twin wheels bolted side by side are assumed to be one wheel transmitting half of the axle load to the floor. In certain cases, wheel loads at one end of an axle magnify the

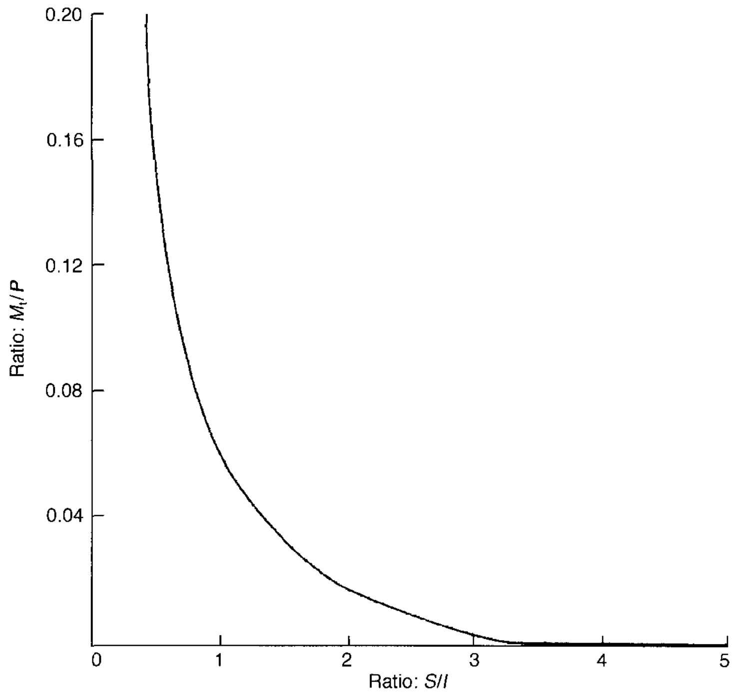



*Fig. 4.1. Relationship between ratio S/l and ratio $M_t/P$ used in assessing the influence of load proximity*

stress beneath wheels at the opposite end of the axle, distance $S$ away ($S$ is measured between load patch centres). To calculate the stress magnification, the characteristic length (radius of relative stiffness, $l$) has to be found from Equation (4.11).

$$l = \left(\frac{Eh^3}{12(1-\mu^2)K}\right)^{0\cdot25} \tag{4.11}$$

Table 4.4 provides values for $l$. Once Equation (4.11) has been evaluated, the ratio $S/l$ can be determined so that Fig. 4.1 can be used to find $M_t/\boldsymbol{P}$, where $M_t$ is the tangential moment. The stress under the heaviest wheel is to be increased to account for the other wheel. The stress to be added is calculated from the following equation:

$$\sigma_{\text{add}} = \frac{M_t}{\boldsymbol{P}}\frac{6}{h^2}\boldsymbol{P}_2 \tag{4.12}$$

where $\boldsymbol{P}$ is the greater patch load, and $\boldsymbol{P}_2$ is the other patch load.

Sum the stresses and verify that the characteristic flexural strength in Table 4.1 has not been exceeded for the appropriate concrete mix.

### 4.7.1 Highway vehicle example using Westergaard equations

Consider a highway vehicle with a rear axle load of 8000 kg and assume a slab thickness of 200 mm on good ground ($K = 0{\cdot}054\,\text{N/mm}^3$). The fatigue factor is 2·0 so the slab can withstand an infinite number of such repetitions. The design axle load is 160 000 N (i.e. 80 000 N × fatigue factor (2·0)) so the design wheel load is 80 000 N. The following parameters are known or calculated:

$$
\begin{aligned}
p &= \text{contact stress between wheel and floor} = 0{\cdot}7\ \mathrm{N/mm^2} \\
b &= \text{radius of contact area} = (W/\pi p)^{1/2} \\
&= (80\,000/0{\cdot}7)^{1/2} \\
&= 191\ \mathrm{mm.}
\end{aligned}
$$

Substituting known values into Equation (4.9):

$$\sigma_{\max} = 0{\cdot}529(1 + 0{\cdot}54 \times 0{\cdot}15)\frac{80\,000}{200^2}\log\left(0{\cdot}2\frac{20\,000 \times 200^3}{0{\cdot}054 \times 191^4}\right)$$

$$\sigma_{\max} = 3{\cdot}03\ \mathrm{N/mm^2}\ \text{beneath one wheel.}$$

Assume that the wheel at the other end of the axle is 2·7m away ($S = 2700$ mm). The radius of relative stiffness ($l$), using Equation (4.11) is:

$$l = \left(\frac{20\,000 \times 200^3}{12(1 - 0{\cdot}15^2)0{\cdot}054}\right)^{0{\cdot}25}$$

$$l = 709\ \mathrm{mm}$$

Thus:

$$\frac{s}{l} = \frac{2700}{709} = 3{\cdot}8$$

From Fig. 4.1:

$$\frac{M_t}{P} = 0{\cdot}005$$

Therefore, using Equation (4.12):

$$
\begin{aligned}
\sigma_{\mathrm{add}} &= \frac{M_t}{P}\frac{6}{h^2}P_2 \\
&= 0{\cdot}005\left(\frac{6}{200^2}\right)80\,000 \\
&= 0{\cdot}06\ \mathrm{N/mm^2}
\end{aligned}
$$

The total stress is 3·03 + 0·06 N/mm$^2$ = 3·09 N/mm$^2$.

By comparing this stress with the characteristic strength of C40 concrete reinforced with 40 kg/m$^3$ steel fibres (3·2 N/mm$^2$), it can be seen that the proposed mix is satisfactory. A C40 concrete incorporating 40 kg/m$^2$ 60 mm long 1 mm diameter anchored steel fibres is inadequate for this design. However, the inclusion of a 250 mm thick granular sub-base would enhance $K$ from 0·054 to 0·073 which would reduce the stress to below the characteristic value.

## 4.8 Meyerhof design method for point and patch loads

The Meyerhof design method is based upon yield line analysis and provides a relatively simple way of selecting the thickness of a concrete slab for a single patch load and for some combinations of similar patch loads. Meyerhof developed his equations by considering how a concrete slab would fail. His initial work related to suspended slabs but he later applied the method to ground bearing slabs. He considered the way in which a slab would ultimately fail and equated the potential energy lost when the load moved downwards to the strain energy absorbed along the cracks that form at the time of failure.

Consider a patch load applied to a ground bearing slab gradually increasing in magnitude. The slab directly beneath the load sags and develops tension towards the underside of the slab. Further away from the patch load, tension develops in the upper part of the slab as the bending transforms from sagging to hogging. For failure to occur, the concrete has to crack in these tension zones. The yield line method assumes that at failure, the slab breaks into a series of plates and that sufficient cracks (or 'plastic hinges') need to form to generate a collection of hinged plates. Figure 4.2 shows typical patterns of hinges generated by one, two and four patch loads.

The following are the Meyerhof equations for point loads.

(*a*) Single point load applied within the body of the slab:

$$\boldsymbol{P}_{\mathrm{u}} = 2\pi[M_{\mathrm{p}} + M_{\mathrm{n}}] \tag{4.13}$$

(*b*) Single point load applied along a free edge of a slab:

$$\boldsymbol{P}_{\mathrm{u}} = (\pi[M_{\mathrm{p}} + M_{\mathrm{n}}]/2) + 2M_{\mathrm{n}} \tag{4.14}$$

(*c*) Single point load applied at a corner of a slab:

$$\boldsymbol{P}_{\mathrm{u}} = 2M_{\mathrm{n}} \tag{4.15}$$

(*d*) Two similar point loads spaced $x$ apart and away from edges or corners of the slab:

$$\boldsymbol{P}_{\mathrm{u}} = \left[2\pi + \frac{1{\cdot}8x}{l}\right][M_{\mathrm{p}} + M_{\mathrm{n}}] \tag{4.16}$$

(*e*) Four similar point loads applied at corners of a rectangle of side lengths $x$ and $y$, all four loads away from edges or corners of the slab:

$$\boldsymbol{P}_{\mathrm{u}} = \left[2\pi + \frac{1{\cdot}8(x+y)}{l}\right][M_{\mathrm{p}} + M_{\mathrm{n}}] \tag{4.17}$$

The corresponding equations for patch loading are as follows.

(*a*) Single patch load applied within the body of the slab:

$$\boldsymbol{P}_{\mathrm{u}} = 4\pi[M_{\mathrm{p}} + M_{\mathrm{n}}]\Big/\left[1 - \frac{a}{3l}\right] \tag{4.18}$$

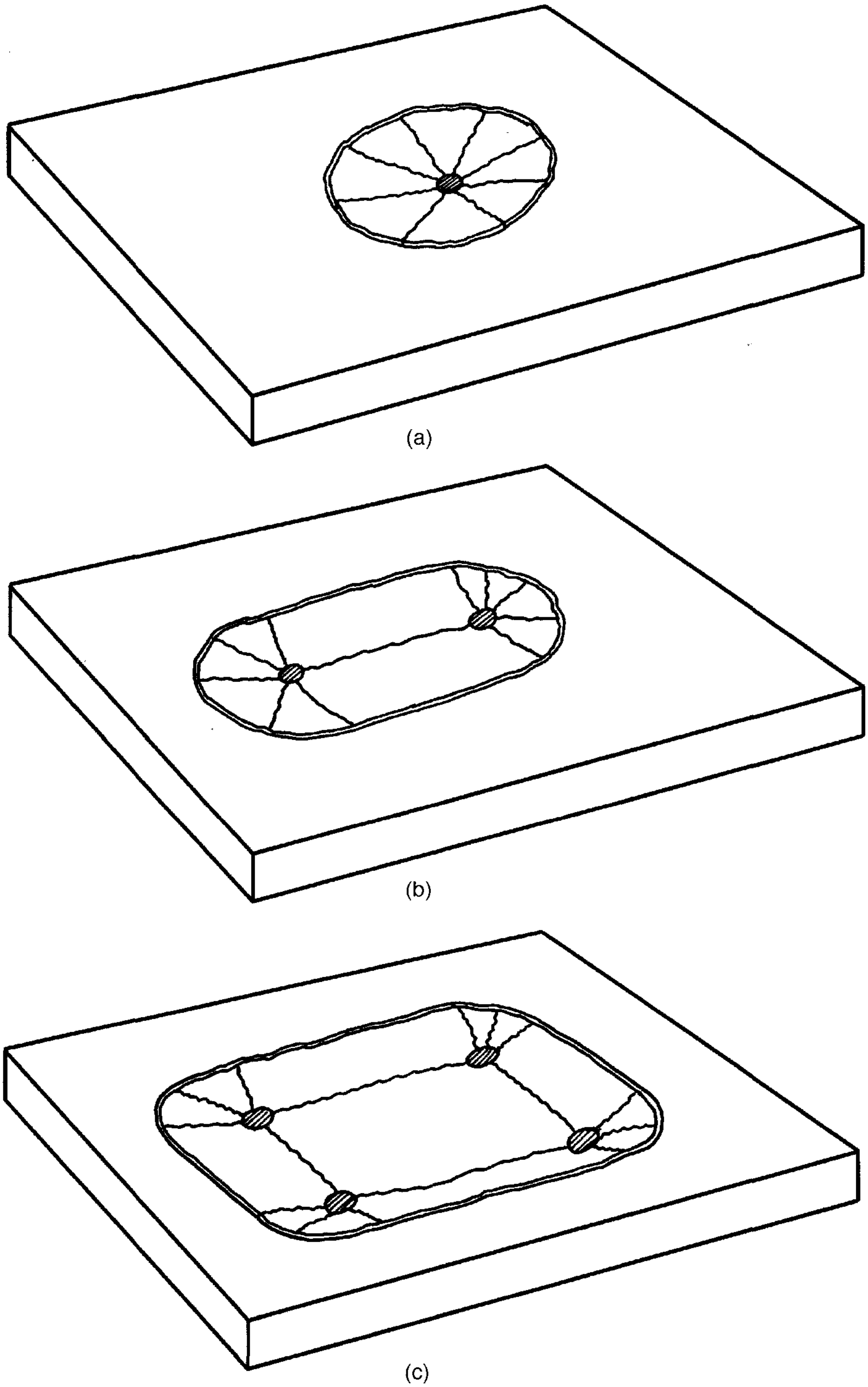



*Fig. 4.2. Yield lines or plastic hinges resulting from (a) single, (b) twin and (c) four patch loads*

(*b*) Single patch load applied along a free edge of a slab:

$$\boldsymbol{P}_\mathrm{u} = (\pi[M_\mathrm{p} + M_\mathrm{n}] + 4M_\mathrm{n})/\left[1 - \frac{2a}{3l}\right] \tag{4.19}$$

(*c*) Single patch load applied at a corner of a slab:

$$\boldsymbol{P}_\mathrm{u} = 2\left[1 + \frac{4a}{l}\right]M_\mathrm{n}] \tag{4.20}$$

(*d*) Two similar patch loads of radius *a*, spaced *x* apart and away from edges or corners of the slab:

$$\boldsymbol{P}_\mathrm{u} = \left[\frac{4\pi}{\left(1 - \frac{a}{3l}\right)} + \frac{1{\cdot}8x}{\left(l - \frac{a}{2}\right)}\right][M_\mathrm{p} + M_\mathrm{n}] \tag{4.21}$$

(*e*) Four similar patch loads applied at corners of a rectangle of side lengths *x* and *y*, all four loads away from edges or corners of the slab:

$$\boldsymbol{P}_\mathrm{u} = \left[\frac{4\pi}{\left(1 - \frac{a}{3l}\right)} + \frac{1{\cdot}8(x + y)}{\left(l - \frac{a}{2}\right)}\right][M_\mathrm{p} + M_\mathrm{n}] \tag{4.22}$$

Where $\boldsymbol{P}_\mathrm{u}$ is the ultimate load, $M_\mathrm{p}$ is the ultimate positive (sagging) moment of resistance of the slab, and $M_\mathrm{n}$ is the ultimate negative (hogging) moment of resistance of the slab. The above two values can be obtained from the equation:

$$M_\mathrm{p,n} = f\left(\frac{h^2}{6}\right)$$

where

| | | |
|---|---|---|
| $f$ | = | characteristic flexural strength of concrete — see Table 4.2 |
| $h$ | = | slab thickness |
| $a$ | = | radius of patch load |
| $x$ | = | spacing of two point of patch loads (centre to centre of load) |
| $y$ | = | spacing of four point loads forming corners of a rectangle — $y$ measured at right angles to $x$ |
| $l$ | = | radius of relative stiffness: |
| $l$ | = | $\left(\frac{Eh^3}{12(1 - \mu^2)K}\right)^{0{\cdot}25}$ |

(See Table 4.4 for values of $l$. Table 4.4 reproduced below for convenience.)
Note that the Meyerhof patch load equations apply when:

$$\left(\frac{a}{l}\right) \geq 0{\cdot}2$$

For values of $a/l$ between 0·2 and zero, interpolate between the patch load and the point load $M_\mathrm{u}$ values.

*Table 4.4. Radius of relative stiffness values (l) for different slab thicknesses and support conditions*

| Slab thickness: mm | Modulus of subgrade reaction $K$: N/mm$^3$ | | | |
|---|---|---|---|---|
| | 0·013 | 0·027 | 0·054 | 0·082 |
| 150 | 816 | 679 | 571 | 515 |
| 175 | 916 | 763 | 641 | 578 |
| 200 | 1012 | 843 | 709 | 639 |
| 225 | 1106 | 921 | 774 | 698 |
| 250 | 1196 | 997 | 838 | 755 |
| 275 | 1285 | 1071 | 900 | 811 |
| 300 | 1372 | 1143 | 961 | 865 |

### 4.8.1 Highway vehicle example using Meyerhof equations

Consider the same highway vehicle that was used as an example in Section 4.7. It has a rear axle load of 8000 kg. Assume a slab thickness of 200 mm on good ground ($K = 0{\cdot}054\,\text{N/mm}^3$). The load is dynamic so a factor of 1·6 can be applied. In the case of a highway vehicle, this will take care of both dynamics and fatigue. The design axle load is 128 000 N (i.e. 80 000 N × dynamic/fatigue factor (1·6)) so the design wheel load is 64 000 N (64 kN). Note that in Section 4.7, the design load was 80 000 N since a factor of 2·0 was applied. However, in Westergaard-based design, the characteristic strength of the concrete is used in design unfactored. In Meyerhof, the characteristic strength of the concrete is divided by its Material Partial Safety Factor of 1·3.

Consider a slab of thickness 200 mm and try C40 concrete containing 40 kg/m$^3$ of anchored steel fibres. (This was the output from the example in Section 4.7.)

$$\text{Radius of patch load, } a = (64\,000/\pi 0{\cdot}7)^{1/2}$$
$$= 170 \text{ mm}$$

From Table 4.2, characteristic strength of concrete = 3·2 N/mm$^2$. Apply the Material Partial Safety Factor of 1·3 to obtain design strength: 3·2/1·3 = 2·5 N/mm$^2$.

Now use:

$$M_{\text{p,n}} = f\left(\frac{h^2}{6}\right)$$

to obtain the ultimate positive and negative moments of resistance of the proposed slab:

$$M_\text{p} = M_\text{n} = 2{\cdot}5 \times 200^2/6 = 16\,667 \text{ Nmm per mm width of slab.}$$

From Table 4.4, the Radius of Relative Stiffness is 709 mm. Note that this is the distance from the centre of the load to where the bending moment in the slab changes from sagging to hogging.

Consider the situation where this single patch load is applied away from the slab edges or corners.

$$\boldsymbol{P}_\text{u} = 4\pi[M_\text{p} + M_\text{n}]/\left[1 - \frac{a}{3l}\right]$$

$\boldsymbol{P}_{\mathrm{u}} = 455\,312\,\mathrm{N} = 455\,\mathrm{kN}(64\,\mathrm{kN}$ required)

Consider the slab edge condition:

$$\boldsymbol{P}_{\mathrm{u}} = (\pi[M_{\mathrm{p}} + M_{\mathrm{n}}] + 4M_{\mathrm{n}}/\left[1 - \frac{2a}{3l}\right]$$

$\boldsymbol{P}_{\mathrm{u}} = 204\,\mathrm{kN}(64\,\mathrm{kN}$ required)

Consider the load applied at the corner of the slab:

$$\boldsymbol{P}_{\mathrm{u}} = 2\left[1 - \frac{4a}{l}\right]M_{\mathrm{n}}$$

$\boldsymbol{P}_{\mathrm{u}} = 65\,\mathrm{kN}$ (64 kN required)

Consider also both wheels separated by 2·7 m.

Introduce all of the now known parameters into the equation:

$$\boldsymbol{P}_{\mathrm{u}} = \left[\frac{4\pi}{\left(1 - \frac{a}{3l}\right)} + \frac{1\cdot 8x}{\left(l - \frac{a}{2}\right)}\right][M_{\mathrm{p}} + M_{\mathrm{n}}]$$

($x$ is the spacing of the rear wheels, which in this case is 2700 mm.)

$$\boldsymbol{P}_{\mathrm{u}} = \left[\frac{4\pi}{\left(1 - \frac{170}{2127}\right)} + \frac{4860}{\left(2127 - \frac{170}{2}\right)}\right][16\,667 + 16\,667]$$

$$\boldsymbol{P}_{\mathrm{u}} = \left[\frac{12\cdot 56}{0\cdot 92} + \frac{4860}{2042}\right]33\,334$$

$$= 534\,344\,\mathrm{N} = 534\,\mathrm{kN}$$

i.e. each wheel load = 534/2 = 267 kN (64 kN required).

This result indicates that a 200 mm thick slab comprising C40 steel fibre reinforced concrete can sustain between 455 kN and 65 kN depending upon the position of the load in relation to the slab. The Westergaard solution suggested that such a slab could sustain 80 kN. Both methods used similar levels of safety factor, although they were applied in different ways.

The reason for the difference between Westergaard (or other elastic methods set out in this chapter) and Meyerhof solutions is the redundancy of a concrete slab. The Westergaard solution considers the critical stress point and ensures that the actual stress remains below the strength of the concrete by the safety factor at that point. Meyerhof recognises that the slab can crack only when sufficient plastic hinges have formed to develop one of the mechanisms shown in Fig. 4.2. This means that there is a significant reserve of strength beyond the condition when stress reaches strength at the critical point, i.e. there is a significant reserve of strength beyond the Westergaard elastic failure criterion. Loading beyond this condition will not lead to failure, rather the stress pattern is redistributed until stress has reached concrete strength along all of the plastic hinges which form the collapse mechanism. As can be appreciated from the Meyerhof example, the load can grow to several times the Westergaard limiting value. However, there remains the possibility that cracks will develop in the critical stress location once the Westergaard elastic condition has been surpassed. In the case of an unreinforced slab, this could lead to the slab's breaking into two which would be unacceptable in most circumstances. However, in the case of steel mesh or steel fibre reinforced concrete, the crack would be bridged by the reinforcement. The author's tests have shown that steel fibre reinforced concrete has a residual flexural strength after the development of the first crack. Therefore, it is likely that steel fibre reinforced concrete slabs will crack before the Meyerhof condition is reached but that the cracks will be controlled by the steel fibres. Also, it should be recognised that the benefit of redundancy is less marked in the situation where a single patch load is applied at the corner of a slab. For this reason, Meyerhof is most beneficial in jointless slabs where this condition can be omitted.

Which should be used, Westergaard (or the other elastic methods set out in this chapter) or Meyerhof? The author has been involved in full-scale tests in which a steel fibre reinforced slab (C40 concrete, 30 kg/m$^3$ steel fibre content) of thickness 140 mm withstood a patch load of 290 kN, suggesting that the Meyerhof method is not unconservative. For several years, floor construction contractors have reported that slabs much thinner than those which Westergaard's equations would require appear to function satisfactorily. Therefore, it would seem sensible to use the Meyerhof equations for ground bearing floors subjected to point or patch loads in situations where cracks are acceptable. However, in situations where any degree of cracking would be unacceptable, then Westergaard equations or the other elastic methods presented in this chapter should be used. This will be the case generally for external slabs subjected to the effects of weather. The Meyerhof equations should be used for steel mesh or steel fibre reinforced concrete floor slabs where some controlled cracking is acceptable.

However, it should be recognised that stresses induced by patch or point loads may represent a small part of the stress regime. Restraint to thermal and moisture induced movement may be the principal structural action. In the past, it is likely that the conservative Westergaard equations provided the requisite factor of ignorance. If Meyerhof equations are to be used in design, then a full understanding of the behaviour of the slab will be required. In particular, the possibility of punching shear failure must be investigated and the stresses arising from restraint to shrinkage and curling must be evaluated. The slab must then be proportioned on the basis of the most adverse combination of all of the stresses that are likely to occur, not just those resulting from patch or point loads.

## 4.9 Eurocode 2 design method for patch loads

Even when elastic and plastic design methods confirm that a slab is adequate, there is the possibility that the load can punch directly through the slab as a result of shear stresses exceeding the shear strength of the slab around the perimeter of the patch load. A check should be made as follows. Firstly, calculate the area over which the shear stress acts. This is the depth of the slab multiplied by the perimeter of the load patch.

Divide the factored load by this area to obtain the shear stress. This stress should not exceed the shear strength of the concrete $v_m$ which is calculated from:

$$v_m = 0{\cdot}5 v f_{cd}$$

where $f_{cd}$ is the design cylinder strength (see Table 1.1 — divide the characteristic cylinder strength by 1·3)

$$\text{and } v = 0{\cdot}6\left[1 - \frac{f_{ck}}{250}\right]$$

($f_{ck}$ = characteristic cylinder strength — see Table 1.1)

Consider a 175 mm thick C35 slab loaded by a circular 90 kN patch load of diameter 250 mm. The area over which the vertical shear force acts is: $\pi \times$ load patch diameter $\times$ slab thickness $= \pi 250 \times 175 = 137\,444\,\text{mm}^2$. The shear stress is $90\,000/137\,444 = 0{\cdot}65\,\text{N/mm}^2$. Now calculate the allowable shear stress as follows:

$$v = 0{\cdot}6\left[1 - \frac{f_{ck}}{250}\right]$$

$$v = 0{\cdot}6\left[1 - \frac{28{\cdot}5}{250}\right]$$

(see Table 1.1 for the $f_{ck}$ value of 28·5)
Therefore, $v = 0{\cdot}53$.

Substitute in $v_m = 0{\cdot}5 v f_{cd}$ where the cylinder design strength $f_{cd}$ is $28{\cdot}5/1{\cdot}3 = 22\,\text{N/mm}^2$. (1·3 is the Material Partial Safety Factor for concrete.)

Therefore, the allowable shear stress $v_m = 0{\cdot}5 \times 0{\cdot}53 \times 22 = 5.83\,\text{N/mm}^2$. By comparing this figure with the applied shear stress of $0{\cdot}65\,\text{N/mm}^2$, it can be concluded that there is no likelihood of punching shear failure in this instance.

Note that for suspended slabs, a further shear check is made at a distance of twice the slab depth from the perimeter of the patch load. Because of the support offered to a ground bearing slab, this is rarely necessary.

## 4.10 Design method from patch loads using design charts derived from Westergaard equations

The above example illustrates the complexity of undertaking calculations in the case of patch loads. A simplified version of the elastic Westergaard method has been developed by the author specifically for designing industrial floors. In order to eliminate much of the effort, a series of design charts has been developed using Westergaard calculations. These charts are reproduced at the end of this chapter.

The procedure is as follows:

(1) Assess the existing conditions.
Determine the Actual Point Load (APL) and modulus of subgrade reaction ($K$) values from Section 4.4.1 to confirm the category of subgrade.

*Table 4.4. Radius of relative stiffness values (l) for different slab thicknesses and support conditions*

| Slab thickness: mm | Modulus of subgrade reaction $K$: N/mm$^3$ | | | |
|---|---|---|---|---|
| | 0·013 | 0·027 | 0·054 | 0·082 |
| 150 | 816 | 679 | 571 | 515 |
| 175 | 916 | 763 | 641 | 578 |
| 200 | 1012 | 843 | 709 | 639 |
| 225 | 1106 | 921 | 774 | 698 |
| 250 | 1196 | 997 | 838 | 755 |
| 275 | 1285 | 1071 | 900 | 811 |
| 300 | 1372 | 1143 | 961 | 865 |

(2) Calculate the additional stress generated by point or patch loads or wheels in close proximity.
   (*a*) If the distance between loads ($S$) is greater than 3 m, the APL can be used directly (depending on the radius of contact zone) to calculate the thickness of the slab using the relevant Design Chart. In this case, go directly to Stage 6.
   (*b*) If the distance between loads is less than 3 m, the radius of relative stiffness ($l$) has to be determined. Table 4.4 shows values of radius of relative stiffness for different $K$ values and slab thicknesses; $E$ = elastic modulus = 20 000 N/mm$^2$; $\nu$ = Poisson's ratio = 0·5.
   From Figure 4.1, determine $M_t/P$, the ratio of the tangential moment to the greater point load by calculating $S/l$. Then use Equation (4.12) to determine the stress to add.

(3) From Table 4.2, select a proposed concrete mix, hence characteristic strength, $\sigma_{char}$.

(4) When two patch loads are acting in close proximity (i.e. less than 3 m apart), the greater patch load ($\boldsymbol{P}$) produces a flexural strength $\sigma_{max}$ directly beneath its point of application. The nearby smaller patch load ($\boldsymbol{P}_2$) produces additional stress $\sigma_{add}$ beneath the larger load. Calculate $\sigma_{max}$ from:

$$\sigma_{max} = \sigma_{char} - \sigma_{add} \tag{4.23}$$

(5) Calculate the Single Point Load (SPL) which, acting alone, would generate the same flexural stress as the actual loading configuration:

$$SPL = APL\left(\frac{\sigma_{char}}{\sigma_{max}}\right) \tag{4.24}$$

where APL is the Actual Point Load.

(6) Prior to using the Design Charts, it is necessary to modify the SPL (Single Point Load) to account for contact area as well as wheel proximity to obtain the ESPL (Equivalent Single Point Load). Design Charts 1 to 10 apply directly when patch loads have a radius of contact between 150–250 mm. Some racking systems and

*Table 4.5. Patch load multiplication factors for loads with a radius of contact outside the range 150–250 mm*

| Radius of contact: mm | Modulus of subgrade reaction $K$: N/mm$^3$ | | | |
|---|---|---|---|---|
| | 0·013 | 0·027 | 0·054 | 0·082 |
| 50 | 1·5 | 1·6 | 1·7 | 1·7 |
| 100 | 1·2 | 1·2 | 1·3 | 1·3 |
| 150 | 1·0 | 1·0 | 1·0 | 1·0 |
| 200 | 1·0 | 1·0 | 1·0 | 1·0 |
| 250 | 1·0 | 1·0 | 1·0 | 1·0 |
| 300 | 0·9 | 0·9 | 0·9 | 0·9 |

pallet transporters have a contact radius of less than 150 mm and some vehicles have a contact radius greater than 250 mm. In these cases multiply the patch load by a factor in Table 4.5 prior to use in the Design Chart.

(7) Use the corresponding Design Chart for the mix selected in (3) to determine slab thickness and return to (3) if an alternative concrete mix is required.

### 4.10.1 Design example for multiple patch loading using Design Chart

A concrete floor is subjected to two patch loads. A 60 kN patch load is applied 1m away from a 50 kN patch load. The 60 kN patch load has a contact zone radius of 100 mm and the 50 kN patch load has a 300 mm radius. The existing ground conditions are poor ($K =$ 0·027 N/mm$^3$). Assume thickness of slab to be 225 mm. The radius of relative stiffness, $l$, is given by Table 4.4 as $l-$ 921 mm.

The distance apart is 1 m (i.e. $S = 1000$ mm), so:

$$\frac{S}{l} = \frac{1000}{921} = 1{\cdot}086$$

From Fig. 4.1:

$$\frac{M_t}{P} = 0{\cdot}053$$

Thus from Equation (4.12):

$$\sigma_{\text{add}} = 0{\cdot}053\left(\frac{6}{200^2}\right)50\,000$$

$$\sigma_{\text{add}} = 0{\cdot}4\ \text{N/mm}^2$$

Try steel fibre reinforced C30 concrete with a characteristic strength of 2·2 N/mm$^2$ (30 kg/m$^3$ steel fibre dosage — see Table 4.2), i.e.:

$$\sigma_{char} = 2 \cdot 2\,\text{N/mm}^2$$
$$\begin{aligned}\sigma_{max} &= \sigma_{char} - \sigma_{add} \\ &= 2 \cdot 2 - 0 \cdot 4 \\ &= 1 \cdot 8\,\text{N/mm}^2\end{aligned}$$

This is the maximum flexural stress that the 60 kN load can be allowed to develop. Calculate the SPL using Equation (4.24):

$$\begin{aligned}SPL &= APL\left(\frac{\sigma_{char}}{\sigma_{max}}\right) \\ &= 60\left(\frac{2 \cdot 2}{1 \cdot 8}\right) = 73\,\text{kN}\end{aligned}$$

From Table 4.5, the modified factor to be applied to the SPL to obtain the ESPL is 1·2:

$$73 \times 1 \cdot 2 = 88\,\text{kN}$$

From Design Chart 4, the thickness of the slab is 225 mm.

Thus, a 225 mm thick C30 concrete slab incorporating 30 kg/m$^3$ steel fibre is adequate for this design.

## 4.11 Design method using finite element derived Design Chart

This design method was developed by the author specifically for heavy lifting equipment using external hardstandings. It is frequently applied to pavements in ports or in similarly loaded areas. It uses elastic finite element analysis to develop a pattern of stresses. The critical stress is compared with the strength of the pavement base material so as to allow a design section to be developed. Wheel proximity, handling equipment dynamics and fatigue are taken into account. The method uses a Design Chart that is reproduced on page 162 (Fig. 4.14).

The method is explained by way of an example. In this example a straddle carrier operation is assessed for loading and subsequent use with the Design Chart to produce a pavement section. In the loading calculations, the damaging effect of one side of the item of plant is considered, as explained in this example.

### *4.11.1 Data*

| | |
|---|---|
| Unladen weight of straddle carrier including spreader beam | = 56 310 kg ($W_T$) |
| Critical container weight | = 22 000 kg |
| Track width | = 4·5 m |
| Wheel spacings | = 2·4 m — 3·6 m — 2·4 m (see Fig. 4.3) |
| Number of likely passes of staddle carriers over the most highly trafficked part of the pavement during design life of pavement | = 960 000 passes |
| CBR of soil | = 5% |
| Sub-base thickness | = 225 mm |

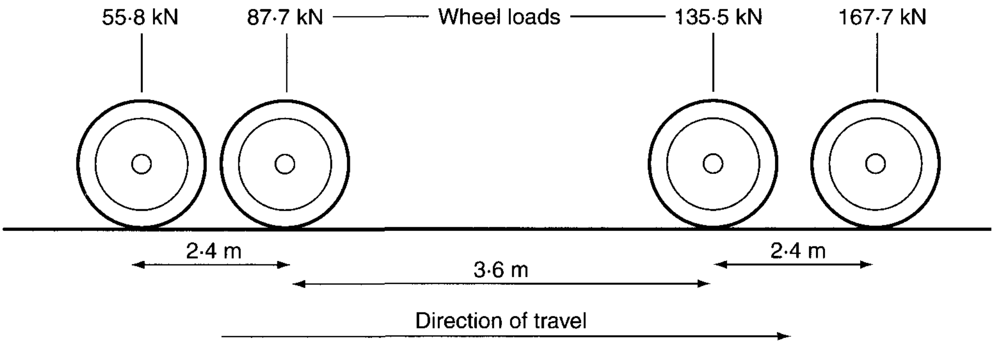



*Fig. 4.3. Straddle carrier wheel loads during braking*

Having defined the foundation material properties, the base material is now calculated and is dependent on the load applied.

## 4.11.2 Calculations

| | |
|---|---|
| Total number of wheels on plant | = 8 |
| Wheel load of unladen plant (kg) | = 56 310/8 = 7 039kg |
| Weight of critical container (kg) | = 22 000 kg, see Chapter 3, Section 3.1. |
| $f_d$ = dynamic factor for braking | = ±50% for extreme wheels, see next paragraph for inner wheels |

Static wheel load = 7 039 + 22 000/8 = 9789 kg = 97·9 kN

The proximity of the wheel loads is now considered to assess their stress interaction using the method shown in Chapter 3, Section 3.6, to calculate the effective depth.

$$\text{Effective depth} = 300^3\sqrt{\frac{35\,000}{5 \times 10}} = 2664\,\text{mm}$$

From Table 3.4, the proximity factor can be interpolated to be 1·14. Therefore, the Effective Static Wheel Load is 97·9 × 1·14 = 111·6 kN. Consider the most adverse loading case of braking and apply the appropriate dynamic factor of ±50% to the wheels at the extreme front and rear, applying the increase in load to the front wheels and the decrease to the rear wheels. The inner wheel loads need to be similarly adjusted but using a factor lower than ±50% determined by considering their relative distance from the vehicle's centre line. In this case, each extreme wheel is 4·2 m from the centre of the vehicle and each inner wheel is 1·8 m from the centre. Therefore, the lower braking factor to be applied to the inner wheels is ±21·4% (i.e. ±50% × 1·8/4·2). We now need to express the four load values that will pass over one spot as an equivalent number of passes of the highest wheel load (167·7 kN) as follows. The damaging effect equation in Section 3.1 is applied to each wheel load in turn.

- **Front** wheel is equivalent to one pass of a load of 167·7 kN.

- **Second** wheel is equivalent to $(135{\cdot}5/167{\cdot}7)^{3{\cdot}75}$, i.e. 0·45 equivalent passes of the front wheel load.
- **Third** wheel is equivalent to $(87{\cdot}7/167{\cdot}7)^{3{\cdot}75}$, i.e. 0·09 equivalent passes of the front wheel load.
- **Fourth** wheel is equivalent to $(55{\cdot}8/167{\cdot}7)^{3{\cdot}75}$, i.e. 0·02 equivalent passes of the front wheel load.

All of the repetitions are converted to an equivalent number of repetitions of the heaviest wheel so that the Equivalent Single Wheel Load used in the Design Chart is derived from the heaviest wheel load. It would be unsafe to convert wheel loads to one of the plant's lower wheel load values.

Therefore, each time the straddle carrier passes over one spot, it applies the equivalent of $(1 + 0{\cdot}45 + 0{\cdot}09 + 0.02) = 1{\cdot}56$ repetitions of the front wheel load of 167·7 kN. This means that the pavement needs to be designed to accommodate 1·5 million passes (i.e. $1{\cdot}56 \times 960\,000$) of a load of 167·7 kN. The base thickness Design Chart can now be used as follows.

- On the vertical axis, the Equivalent Single Wheel Load is 167·7 kN.
- The appropriate curve is the one corresponding to 1·5 million passes.
- The following alternative thicknesses can be used:
  - C10 concrete 400 mm
  - Plain C30/C40 concrete 300 mm
  - 20 kg/m$^3$ steel fibre C30/C40 concrete 250 mm
  - 30 kg/m$^3$ steel fibre C30/C40 concrete 225 mm
  - 40 kg/m$^3$ steel fibre C30/C40 concrete 215 mm.

Consider how the pavement section required would change if alternative dynamic factors were used. For example, if the straddle carriers were to brake while cornering, the wheel loads would increase by 60% of their static value (i.e. $0{\cdot}6 \times 111{\cdot}6 = 67{\cdot}0$ kN) so that the wheel loads would be as in Fig. 4.4.

We now need to express the four load values that will pass over one spot as an equivalent number of passes of the highest wheel load (224·7 kN) as follows. The damaging effect equation in Section 3.1 is applied to each wheel load in turn.

- **Front** wheel is equivalent to one pass of a load of 224·7 kN.
- **Second** wheel is equivalent to $(202{\cdot}5/224{\cdot}7)^{3{\cdot}75}$, i.e. 0·68 equivalent passes of the front wheel load.
- **Third** wheel is equivalent to $(154{\cdot}7/224{\cdot}7)^{3{\cdot}75}$, i.e. 0·25 equivalent passes of the front wheel load.
- **Fourth** wheel is equivalent to $(122{\cdot}8/224{\cdot}7)^{3{\cdot}75}$, i.e. 0·10 equivalent passes of the front wheel load.

Therefore, each time the straddle carrier passes over one spot, its outside wheels apply the equivalent of $(1 + 0{\cdot}68 + 0{\cdot}25 + 0{\cdot}10) = 2{\cdot}03$ repetitions of the front wheel load

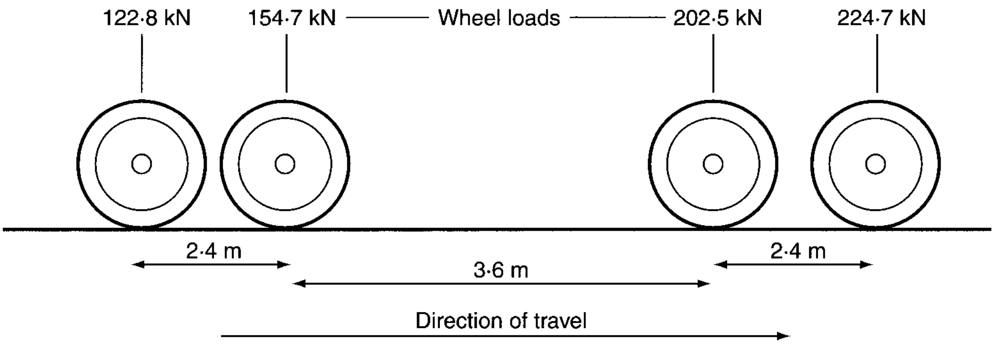



*Fig. 4.4. Straddle carrier wheel loads during braking and cornering*

of 224·7 kN. This means that the pavement needs to be designed to accommodate 2 million passes (i.e. $2{\cdot}03 \times 960\,000$) of a load of 224·7 kN. The base thickness design chart can now be used as follows.

- On the vertical axis, the Equivalent Single Load is 224·7 kN.
- A 2 000 000 passes curve has to be interpolated between the 1 500 000 and the 6 000 000 passes curves.
- The following alternative thicknesses can be used:
  - C10 concrete 500 mm
  - Plain C30/C40 concrete 350 mm
  - 20 kg/m$^3$ steel fibre C30/C40 concrete 325 mm
  - 30 kg/m$^3$ steel fibre C30/C40 concrete 275 mm
  - 40 kg/m$^3$ steel fibre C30/C40 concrete 250 mm.

Finally, consider the case where straddle carriers are running freely on a smooth surface so that no dynamic factors need be applied. In this configuration, the wheel loads are as in Fig. 4.5.

The pavement withstands four repetitions of a wheel load of 111·6 kN as each straddle carrier passes so the pavement must be designed to withstand 3 840 000 passes (say 4 000 000) of an Equivalent Single Load of 111·6 kN. The following alternative thicknesses can be used:

- C10 concrete 325 mm
- Plain C30/C40 concrete 250 mm
- 20 kg/m$^3$ steel fibre C30/C40 concrete 225 mm
- 30 kg/m$^3$ steel fibre C30/C40 concrete 200 mm
- 40 kg/m3 steel fibre C30/C40 concrete 175 mm.

In the case of plain concrete, different operational conditions led to pavement thicknesses required varying between 250 mm and 350 mm. In some cases, it may be possible to take advantage of known modes of operation and proportion the pavement courses to meet the thicknesses required exactly. While this may reduce initial construction costs, it has the disadvantage of constraining future operations and may

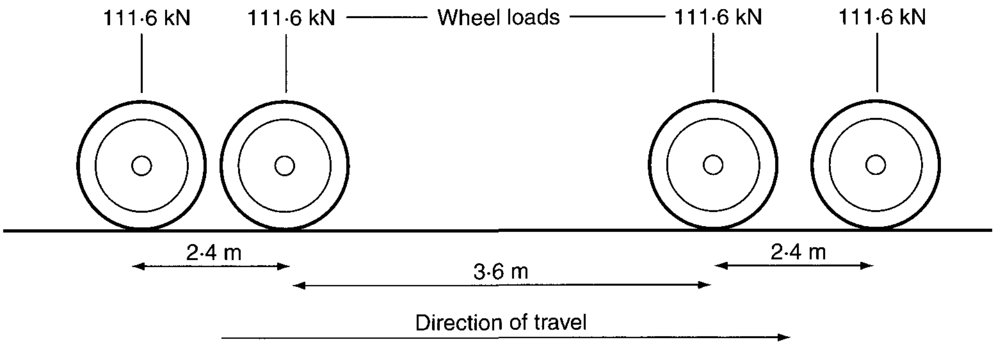



*Fig. 4.5. Straddle carrier wheel loads during free running*

lead to additional complexity in the construction process. It may prove cost effective to provide an initial pavement that will not sustain all potential operational situations and to allow the plant to become the proof testing system so that small areas may have to be strengthened later. While this staged approach has the advantage of lowering initial costs, this must be balanced against the disadvantage associated with the disruption that may occur should the pavement need to be upgraded later.

## 4.12 Preliminary design method to determine initial slab thickness

The following simplified design procedure can be used to establish an initial estimate of slab thickness required. The detailed procedures set out in this chapter should be followed to obtain an accurate design solution. It is assumed that the design load will not be applied to one spot in the slab more than 1 500 000 times.

1. Assess static load.
2. In the case of industrial plant, modify load for dynamic effects according to Table 4.6.
3. Modify design load when two or more loads are applied 2·4 m apart or closer according to Table 4.7.
4. Select plain C30/C40 concrete slab thickness required from Table 4.8.
5. If 30 kg/m$^3$ or more steel fire is to be added, reduce slab thickness by 30%.
6. Select joint spacings according to Table 2.1.

## 4.13 Restraint to contracting

The basic premise underlying most concrete pavement design methods is that stresses developed as a result of the concrete slab changing temperature or moisture content are contained by the provision of stress relieving joints, whereas stresses developed by traffic and other applied loads are controlled by proportioning the thickness of the slab and its underlying supporting courses. The exception is in continuously reinforced concrete pavements (CRCP) in which case temperature and moisture loss stresses are contained by the composite action of the reinforcement and the concrete.

*Table 4.6. Multiplication factors for dynamic wheel loads*

| Condition | Plant type | Multiplication factor |
|---|---|---|
| Braking | Front lift truck | 1·3 |
| | Straddle carrier | 1·5 |
| | Side lift truck | 1·2 |
| | Tractor and trailer | 1·1 |
| Cornering | Front lift truck | 1·4 |
| | Straddle carrier | 1·6 |
| | Side lift truck | 1·3 |
| | Tractor and trailer | 1·3 |
| Acceleration | Front lift truck | 1·1 |
| | Straddle carrier | 1·1 |
| | Side lift truck | 1·1 |
| | Tractor and trailer | 1·1 |
| Uneven surface | Front lift truck | 1·2 |
| | Straddle carrier | 1·2 |
| | Side lift truck | 1·2 |
| | Tractor and trailer | 1·2 |

*Table 4.7. Wheel load multiplication table*

| Load spacing: mm | Multiplication factor |
|---|---|
| 300 | 2 |
| 600 | 1·9 |
| 900 | 1·8 |
| 1200 | 1·7 |
| 1800 | 1·5 |
| 2400 | 1·3 |

*Table 4.8. Slab thickness required for various design loads*

| Design load: kN | Slab thickness: mm |
|---|---|
| 50 | 150 |
| 100 | 200 |
| 150 | 250 |
| 200 | 300 |
| 250 | 350 |
| 300 | 400 |
| 350 | 425 |
| 400 | 450 |
| 450 | 475 |
| 500 | 500 |

Whether temperature or moisture loss stresses are predominant depends upon many factors that are difficult to calculate. Moisture related stresses are potentially greater than temperature related ones by an order of magnitude. However, it is often the case that a highway pavement or external hardstanding retains much of its moisture throughout its life. Also, moisture related stresses develop slowly so creep often reduces them significantly. Temperature stresses, on the other hand, are often at their most severe immediately following construction as the setting concrete cools. Furthermore, temperature related stresses are usually diurnal so creep has little mitigating effect. For this reason, temperature related effects are the ones of most concern in most concrete highway pavement and external hardstanding projects.

Although moisture loss effects are usually less important than temperature related effects, they need careful consideration in the case of highways constructed in a dry climate. Both temperature and moisture can cause a slab to shrink uniformly, to curl upwards at its perimeter or to curl downwards at its perimeter. The way in which these three conditions impart stress into the slab is now considered.

As a result of uniform temperature fall or moisture loss, a concrete slab will shrink uniformly about its centre on plan. Theoretically, the centre will remain stationary. At a distance from the centre the slab will attempt to displace horizontally and this displacement will increase uniformly towards the edge of the slab. Frictional restraint between the underside of the concrete slab and the surface of the sub-base will inhibit or prevent this movement and so generate tensile stress within the slab. This stress in the concrete resulting from frictional restraint to shrinkage can be calculated. The force required to overcome the friction force is given by the expression, $\boldsymbol{F}_{\mathrm{f}} = w\mu$, where $\boldsymbol{F}_{\mathrm{f}}$ is the friction force, $w$ is the weight of the concrete (calculated by assuming a density of $24\,\mathrm{kN/m^3}$) and $\mu$ is the coefficient of friction. As the weight of concrete generating frictional restraint increases with distance from the slab edge, the stress gradually increases to a maximum at the slab centre (there is zero stress at the edge of the slab and at the joints).

Assuming the values for the coefficient of friction between concrete and sub-base and polythene are 0·65 and 0·15 respectively, the theoretical stresses that result from uniform shrinkage friction are shown in Fig. 4.6. Even without a slip membrane, the stresses are low, attaining a value of less than $0{\cdot}05\,\mathrm{N/mm^2}$ in a 6 m long slab. Figure 4.6 illustrates why the provision of a slip membrane is not a crucial issue. Indeed, in an unreinforced concrete slab with closely spaced joints at, say, 6 m spacings, the provision of a slip membrane may be detrimental in that it may concentrate movement at one joint, which can then become a maintenance problem, while other joints never operate (it may be preferable to provide a layer of polyethelene to prevent concrete water loss into the sub-base).

## 4.14 Restraint to curling and hogging

### *4.14.1 Slab perimeter curling downwards (hogging)*

As a result of the underside of the concrete slab cooling or drying faster than the top, non-uniform shrinkage develops throughout the slab with the lower concrete shrinking more than the upper. The result of this non-uniform shrinkage will be hogging of the

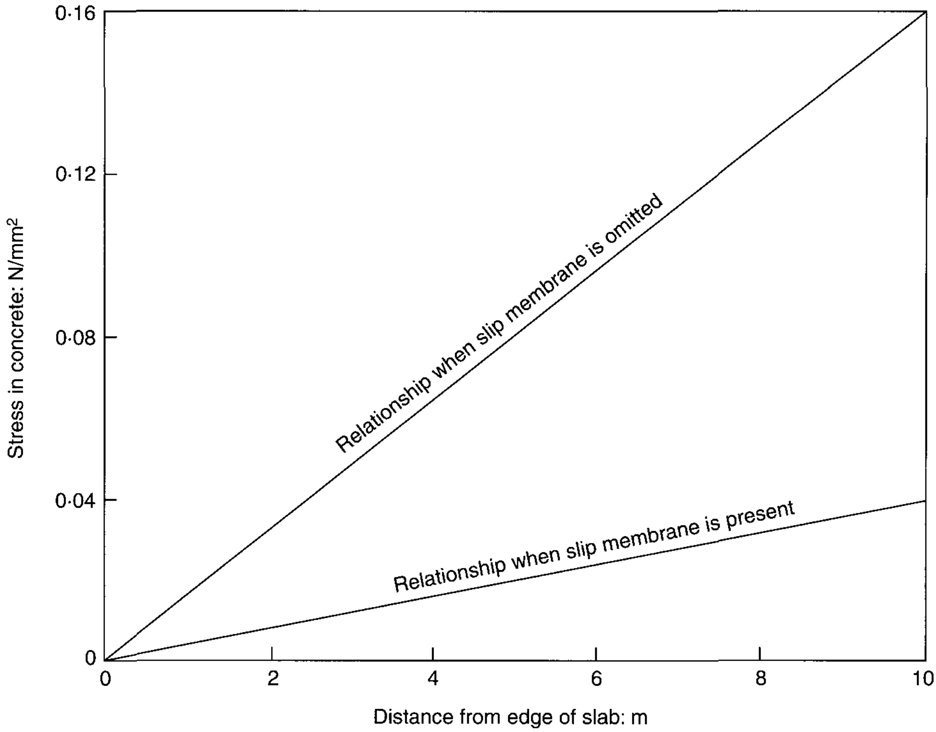



*Fig. 4.6. Relationship between friction induced stress developed in slab against distance from edge of slab for slabs with and without slip membranes*

slab. Assuming the hogging slab can be represented by a simply supported beam of length $L$ with a uniformly distributed load equal to the concrete weight then the maximum moment, $M = wL^2/8$ where $L$ is the length of the slab (the distance between joints) and $w$ is the dead weight of the concrete assumed to be 24 kN/m$^3$. The stress is calculated from the equation $\sigma/y = M/I$ where $\sigma$ is the stress, $y$ is the depth to the neutral axis, $M$ is the bending moment and $I$ is the second moment of area. In the extreme case, $L$ would be the distance between joints allowing rotational freedom. Figure 4.7 shows stresses which would develop in the hogging situation for a 200 mm thick slab.

Figure 4.7 demonstrates that such behaviour would crack each bay. In fact, the temperature fall or moisture loss is usually insufficient to cause the slab to separate from its sub-base so this extreme condition rarely occurs.

### *4.14.2 Slab perimeter curling upwards (curling)*

The result of the upper side of the slab cooling or drying faster than the underside will be that the slab will attempt to curl upwards at its edges. This curling can be represented by a cantilever of length $L$ equal to the curled length. Assuming this cantilever carries a uniformly distributed load generated by the weight of the concrete (based upon an

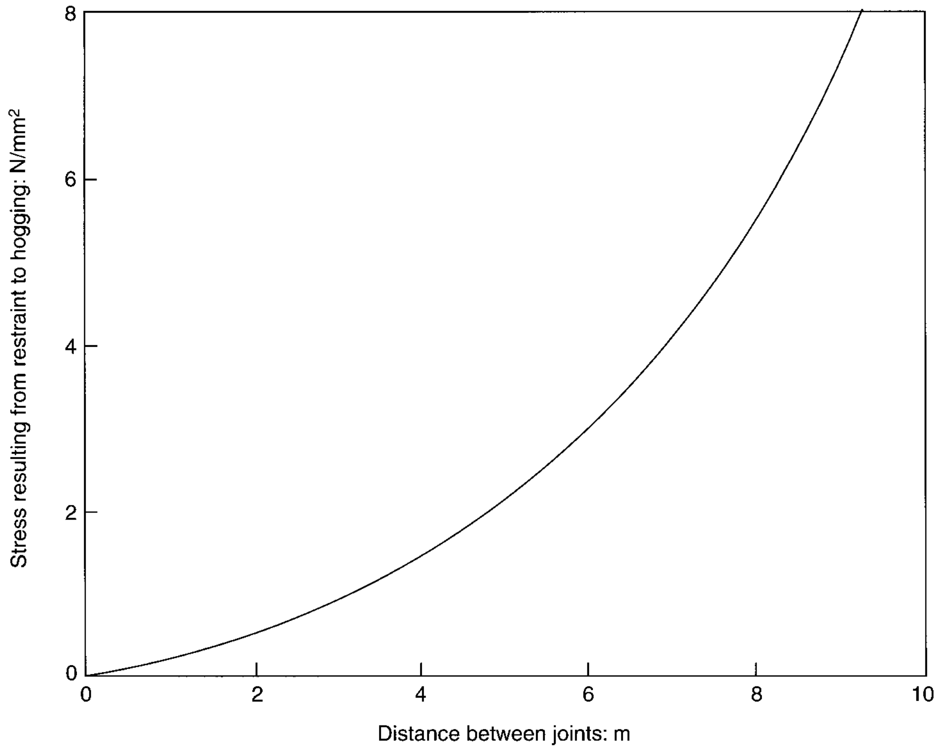



*Fig. 4.7. Relationship between joint spacing and stress developed as a result of restraint to hogging*

assumed density of 24 kN/m$^3$), the bending moment at the slab's point of contact with the ground is given by the expression, $M = wL^2/2$. As in the case of hogging, the stress can then be calculated from the expression $\sigma/y = M/I$. Figure 4.8 shows curling stresses calculated for a 200 mm thick slab.

### 4.14.3 Calculation of slab temperature changes

In order to determine temperature related stresses through the depth of a slab, the temperature profile at the time of set needs to be known. From that time forward, whenever the 'at set' temperature profile is replicated, temperature stresses will disappear. The 'at set' temperature profile has been investigated by several researchers and they have concluded that there can be no standard profile of value to the designer. The profile depends upon the type of concrete, the curing regime, the weather during concreting and the time of day at which the concrete set.

Figure 4.9 illustrates differing temperature profiles at set and at subsequent times in different climatic conditions. Figure 4.9 shows that concrete that sets during a warm mid-afternoon may have a locked-in profile as shown in Fig. 4.9(b). If this slab were in a warm climate, then, it might subsequently be subjected to a profile as in Fig. 4.9(e) during the night. In such a case, the temperature of the slab surface has fallen by say 20° and the

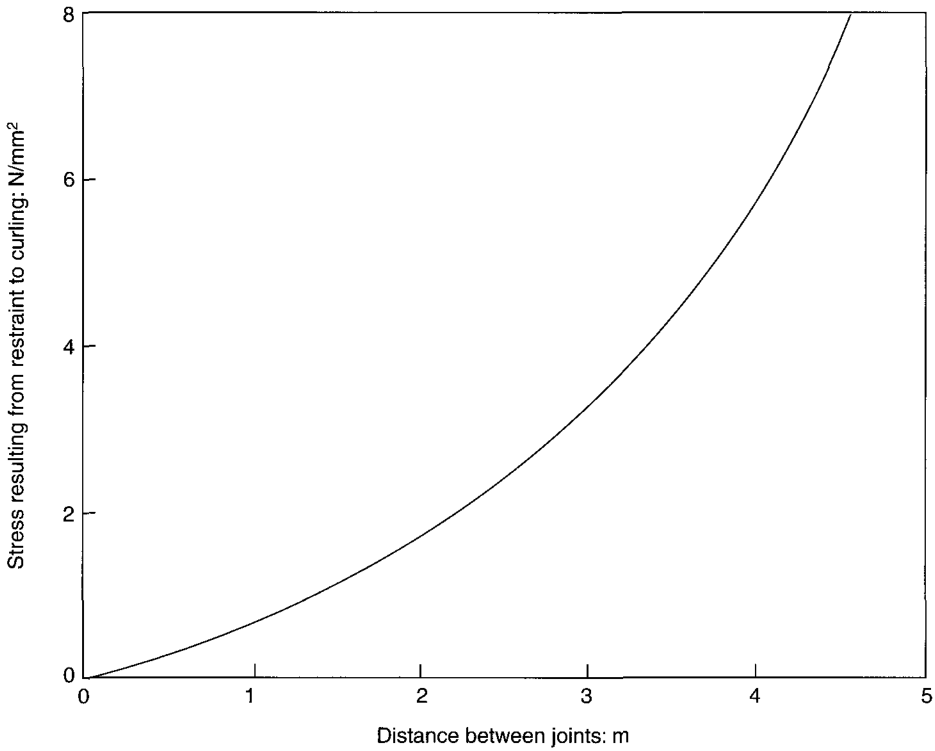



*Fig. 4.8. Relationship between joint spacing and stress developed as a result of restraint to curling*

temperature of the underside of the slab has increased by 17°. This will cause the slab to attempt to curl upwards at its perimeter (curling). The self-weight of the slab together with applied loading will attempt to keep the slab in contact with its sub-base so tensile stresses will develop at and near the upper surface of the slab. Their value will depend upon the relative elastic properties of the slab and the underlying sub-base material.

The opposite effect would occur for slabs that developed their initial set during a cold morning following a relatively warm night, in which case the 'as set' profile shown in Fig. 4.9(a) would apply. If, subsequently, this slab were subjected to the temperature profile as shown in Fig. 4.9(d), the surface temperature rises by say 25° and the underside temperature falls by 15°. This causes the slab to attempt to curl downwards at its perimeter (hogging).

The above two cases represent extremes that slabs might be expected to sustain in normal situations. In the first case, the upwards perimeter curling is caused by a temperature differential of 37° and, in the second case, the downwards perimeter curling temperature differential is 40°. In order to gauge the magnitude of the stresses that might be generated, consider the extreme case in which the slab is fully restrained against hogging. The concrete might have a coefficient of thermal expansion of 0·000009 per degree Celcius. Therefore, the maximum tensile strain would be 40 times 0·000009 or

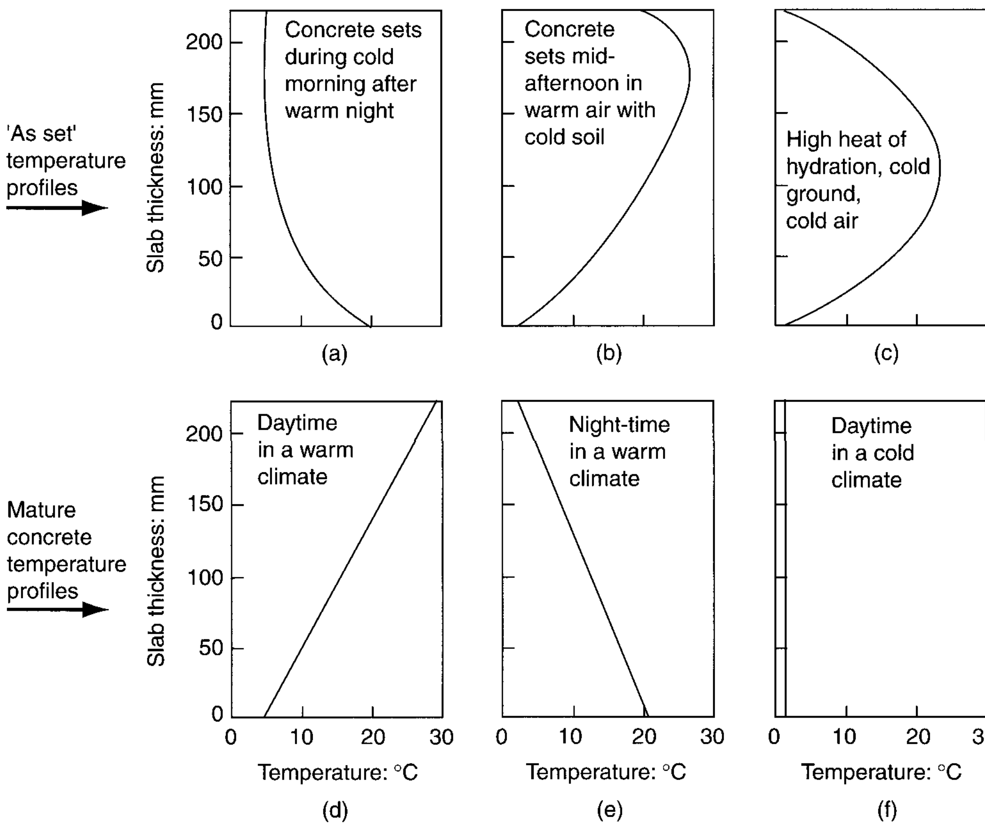



*Fig. 4.9 Concrete slab temperature profiles at set and at subsequent times*

0·00036. Concrete with a Young's modulus of 20 000 N/mm$^2$ would develop a tensile stress of approximately 7 N/mm$^2$, which is sufficient to crack the concrete. This value is never attained in the concrete because full restraint is never achieved and because curing regimes ensure that the 'as set' profiles shown in Fig. 4.9 are not attained in well controlled projects. In practice, it is found that providing transverse warping joints, i.e. joints that permit rotation, at spacings of between 5 and 30 m, depending upon the level of reinforcement, is sufficient to control temperature stresses. Table 2.1 shows joint spacings that have been found to be sufficient to control temperature and moisture related cracking.

Difficulties can occur when different concretes are used in two layer construction. The use of low heat concrete below pavement quality concrete can lead to excessive tensile stresses and difficulties were experienced, for example, on the London Orbital Motorway (M25) in which transverse cracking occurred.

Moisture related shrinkage occurs as the concrete slab loses its free water through evaporation. Typically, pavement quality concrete will have a water/cement ratio of 0·35. Approximately two-thirds of this water is combined chemically with the cement during hydration and the remainder acts as a lubricant, creating workability. All of this free water has the opportunity to evaporate and the volume which it occupied is no longer present so shrinkage occurs. There may be 50 l/m$^3$ of water evaporating over a period of several months following construction. This represents 5% of the volume of the concrete, so significant shrinkage can occur. In fact, creep and precipitation reduce the effect of shrinkage to a controllable level and the joint spacings discussed earlier are usually sufficient to prevent distress.

The theoretical stresses induced by friction are negligible and the slip membrane is not required from the point of view of the stresses due to friction. However, in the cases of hogging and curling the theoretical stresses can exceed the flexural strength of the concrete. Hogging is uncommon when a slip membrane is used because the slip membrane reduces loss of moisture from the bottom of the slab so preventing it from hogging. Good curing of the concrete will reduce drying shrinkage and, provided the stresses are developed slowly, concrete creep will reduce curling stresses to levels that require no additional reinforcement.

## 4.15 Pile supported floors

The remainder of this chapter deals with steel-fibre reinforced concrete floors supported by an orthogonal grid of piles, such that the floor spans between the piles in two directions. Conventional reinforcement is provided to create support strips linking the piles in both orthogonal directions. Floors of this type are particularly well suited to moderate to lightly loaded floors constructed over ground of low bearing capacity. The capability of this type of floor has been demonstrated by large scale physical experiments and a non-linear finite element based study carried out at Braunschweig Technical University.

The procedures that are now explained were developed so as to simplify design in the case of an industrial floor supported on piles and including strengthening support beams spanning between the piles. The procedure allows for the extreme case in which the ground beneath the slab settles so that the slab is entirely supported by its edge support strips and thence by the piles.

The design methods consider, firstly, the analysis of bending and shear, and, secondly, detailed design and construction methods. Note that the method applies only to those floor slabs used as storage areas and which would not create a health and safety hazard should they deflect excessively. This will include most industrial floors. Floors serving high storage racking systems do not fall into this category and the method should not be applied to traditional suspended floors, including floors at ground level but with basements or cellars beneath. For those cases, more detailed analysis is required.

## 4.16 Design criteria

Because a conventional ground bearing concrete floor slab is not designed on the basis of limiting crack width, neither is the type considered here. Floor slabs are designed to transmit applied loads directly to the ground and to accept stresses that develop as a result of restraint to temperature or moisture induced movement. A slab designed as described here will perform at least as well as a conventional steel fibre reinforced concrete floor slab.

Notwithstanding the above, even when the construction is of high quality, there can be no guarantee of a maximum crack width. Contractors and clients should be aware of this and should recognise that cracks may be approximately 0·4 mm wide and, in some circumstances, cracks of that width may need to be sealed. In this respect, pile supported floors are similar to other steel fibre reinforced floors but it should be recognised that in zones of high stress, additional percentages of reinforcement will help to control cracking in piled floors. It is recommended that a warranty document should be developed which should state that cracks over a certain width, say, 0·4 mm, may occur and that such cracks will require sealing.

Crack widths resulting from unfactored loads will be substantially smaller than those that would develop in conventional floors. This is because the design method is based upon predicting the loads that would lead to yield lines developing and then applying a load safety factor to guard against the development of those yield lines. This load safety factor is accounted for by limiting stresses to values that are significantly lower than those which would allow yield lines to develop. Also, a minimum steel fibre content of $20\,kg/m^3$ should be specified so that in the post-cracking condition, the applied moment can still be sustained by a combination of the concrete taking compressive stress and the steel fibres taking the tensile stress.

## 4.17 Limit state of deflexion and cracking

Eurocode 2, Table 4.14, provides slenderness values for unrestrained slabs, which, if not exceeded, will lead to acceptable deflexion. Depending upon concrete stress, slenderness values of 21–30 will result in acceptable deflexions. Providing these slenderness values are not exceeded, there is no need to undertake deflexion calculations. In the case of pile supported floors, the relevant slenderness value is 21 and this value is henceforth used as the deflexion limiting value.

Limiting crack width in ground bearing floors is considered necessary in only some special applications. Where crack widths do need to be calculated, many input parameters need to be taken into account. In particular, moisture and temperature related shrinkage restraint, restraint to settlement and the horizontal forces that develop between the slab, the underlying soil and the piles are particularly important. Calculating the effect of the parameter can be difficult and may lead to inconclusive results.

Where crack width calculations are required, a full elastic analysis of the slab will have to be carried out. Separate calculation will have to be carried out for applied load and for the effects of restraint to thermal and moisture induced movements and the most adverse situation will become the design condition. Redistribution of moments can be undertaken for such analysis, i.e. the peak moments altered to reflect the change in the shape of the bending moment values at failure. Effectively the reaction line can be moved up or down on the bending moment diagram.

## 4.18 Additional criteria

To enhance the performance of pile supported slabs, the following should be taken into account.

- A minimum reinforcement percentage is required in the zones of negative (sagging) moments along the support strips.
- The stresses in the steel bars reinforcing the support strips should be calculated elastically.
- By limiting concrete tensile stresses to strength value given in Eurocode 2, the panels between support strips will remain crack free.

## 4.19 Assessment of load bearing capability

### 4.19.1 Loading

Stresses resulting from the most adverse combination of dead and imposed load should be calculated. Calculations for bending and punching shear should be undertaken separately.

### 4.19.2 Partial safety factors

Ultimate load calculations are undertaken for the following partial safety factors.

*Material partial safety factors*

| | |
|---|---|
| Concrete, including steel fibre reinforced concrete: | $\gamma_c = \gamma_f = 1{\cdot}50$ |
| Conventional steel reinforcement: | $\gamma_s = 1{\cdot}15$ |

*Load partial safety factors*

| | |
|---|---|
| Dead load: | $\gamma_G = 1{\cdot}35$ |
| Imposed load: | $\gamma_Q = 1{\cdot}50$ |

For situations where loading would be of benefit rather than harm:

$$\gamma_G = 1{\cdot}00 \text{ and } \gamma_Q = 0{\cdot}00$$

### 4.19.3 Moment of resistance calculations

A slab's bending strength is checked by yield line analysis. The whole floor is divided into panels for which bending is checked individually. Tables are provided for each type of panel, which avoids the need to postulate yield line patterns for each panel type. The tables apply only when the spans of neighbouring panels do not differ by more than 25% of the lesser span. In the case of interior panels subjected to uniform loading, the usual yield line pattern is one in which orthogonal yield lines develop (i.e. yield lines at right angles to each other). More complicated yield line patterns may occur for piles of large cross-sectional areas since secondary yield lines may develop in close proximity to those piles. This does not occur in the case of edge and corner panels, in which case simple orthogonal yield line patterns form with the yield lines running parallel with the panel sides.

Because the bending strength of steel fibre reinforced concrete panels is less than that of their reinforced concrete support strips, global yield lines will form running parallel with and normal to the support beams. Both the behaviour of individual panels and the behaviour of local critical regions of each panel are considered in this chapter. Table 4.9 shows the cases that need to be considered.

*Table 4.9. Coefficients for load assessment of panel loads*

| Type of panel | Global yield line failure | Partial yield line failure |
|---|---|---|
| Interior | $\lambda_M$ | $\lambda_{M,i}$ |
| Edge | $\lambda_E$ | $\lambda_{E,i}$ |
| Corner | $\lambda_C$ | $\lambda_{C,i}$ |

In the case of point loads, it will be necessary to determine the yield line pattern by minimising the total strain and potential energy in the system.

### 4.19.4 Punching shear calculation

Eurocode 2 allows punching shear to be taken into account in a simplified manner ignoring the strength enhancing benefit of the steel fibres (see Section 4.3.9).

## 4.20 Detailing and construction recommendations

1. In view of the anticipated loadings and the limited strength of panels between piles and support strips, the maximum grid width $a_{max} = 4{\cdot}0$ m for an imposed uniformly distributed load $q_k > 5{\cdot}00$ kN/m$^2$ and $a_{max} = 5{\cdot}0$ m for $q_k \leq 5{\cdot}00$ kN/m$^2$.
2. Slabs should be between 200 and 400 mm thick.
3. The slenderness ratio of the slab should not exceed 20 and if possible should be limited to 15 or less.
4. Special care is needed in the case of high patch or point loads. In cases where patch or point loads are fixed, consideration should be given to the provision of additional conventional steel reinforcement in the slab or to installing additional piles directly beneath the point loads.
5. Pile supported floor slabs cannot be used as elements in water retaining structures. In such cases, a prestressed steel fibre reinforced concrete floor might be considered.
6. In the case of uniformly distributed loading a lower cost floor can be provided by reducing the size of the edge and corner panels by 25%.

## 4.21 Selection and detailing of reinforcement

1. In order to form ties in the two orthogonal directions, support strip reinforcement should be continuous through the entire floor in both orthogonal directions and there should be a near constant level of reinforcement throughout the length of each support strip.
2. Calculation will produce areas of steel in the support strips of $A_{s,support}$ over a pile and $A_{s,field}$ between piles. The ratio $A_{s,support}/A_{s,field}$ does not affect the load that neighbouring panels can support. Nonetheless, reinforcement should be located in the position of maximum bending.

## 4.22 Construction measures for dealing with restraint

1. Details should be developed to deal with restraint that might be introduced by shrinkage of the concrete during its early life, abrupt changes in floor shape,

possibly with stress inducing sharp corners, changes in concrete thickness and voids, or foundation elements within the floor slab.

2. The underside of the slab should remain level in the vicinity of the piles and there should be no downstands.
3. Ensure that the floor is isolated from other structural members. An isolation joint should be provided around the slab.
4. A slip membrane should be provided beneath the floor slab.
5. Large changes in cross-section may influence the stresses within the slab. Each individual case should be considered and additional conventional reinforcement may be required.
6. Providing all piles are located within 100 mm of their design location, no adverse effects will be generated. Where piles have not been located to this tolerance level, calculations should be undertaken to assess the effect.
7. In order to eliminate the notch effect, additional conventional steel reinforcement should be provided at re-entrant corners.
8. Reinforcement provided in the lower zone of support strips should be continuous throughout the length of each support strip, including the lengths at pile location.

## 4.23 Notation

### 4.23.1 *Latin letters*

The following notation is used in Sections 4.24–4.39.

| | |
|---|---|
| $a_x$ $a_y$ | panel dimensions (normally shown in the local coordinate system so that $a_x < a_y$) |
| $d$ | static height |
| $g$ | dead load |
| $q$ | imposed load |
| $f$ | concrete strength |
| $h$ | structural element thickness |
| $s$ | slenderness |
| $m$ | bending moment per unit length (kN-m/m) |
| $M$ | bending moment (kN-m) |
| $\mathbf{R}_{d,m}$ | design resistance moment stress (kN/m$^2$) or (kN) |
| $S_{d,m}$ | design load (kN/m$^2$) or (kN) |

### 4.23.2 *Greek letters*

| | |
|---|---|
| $\alpha$ | creep factor |
| $\Lambda$ | orthotropic factor |
| $\sigma$ | stress |
| $\varepsilon_c$, $\varepsilon_s$ | concrete, steel strain |
| $\rho_u$ | unfactored ratio of the moment of resistance of support strip/moment of resistance of steel fibre reinforced panel |
| $\rho_d$ | factored ratio of the moment of resistance of support strip/moment of resistance of steel fibre reinforced panel |

| | |
|---|---|
| $\epsilon$ | ratio of spans in orthogonal direction |
| $\eta$ | geometric proportion factor for defining location of yield line |
| $\lambda$ | load factor |
| $\xi$ | ratio of pile diameter to span |
| $\kappa$ | support load factor when considering pile cross-section |

### 4.23.3 Subscripts

| | |
|---|---|
| c | concrete |
| d | design value |
| eq | equivalent |
| f | effect of steel fibres |
| i | internal panel |
| k | characteristic values |
| m | structural element/effect of moment stress |
| min | minimum reinforcement |
| o,u | upper/lower reinforcement position |
| s | reinforcing steel in support strips |
| t | tension |
| $x$,$y$ | axis direction (coordinates) |
| $R_d$ | design values of structural elements' resistance (kN/m$^2$) |
| M,E,C | inner panel, edge panel, corner panel |
| F,S | field/support range |

## 4.24 Resistance to bending

Bending calculations are carried out based upon the principle that the internal work absorbed in developing the pattern of yield lines is equal to the external work done by the load as it descends in creating the yield lines. A plastic analysis of the yield lines is undertaken as rotation takes place. Even with a slab thickness of 400 mm and a dosage of 35–40 kg/m$^3$ of steel fibres, it can be shown that the section has sufficient rotation capacity to attain a full plastic moment.

Owing to the possible effect of slab restraint on the development of cracking, the initial tensile strength created by the fibres spanning over a crack is taken to be:

$$f_{\text{fct}} = 0{\cdot}37 f_{\text{fct,eq,150}}$$

As a result of the partial safety factor concept and the different material partial safety factors for fibre reinforced concrete and conventional reinforcing steel, there are differences between the ratio $\rho_d$ and $\rho_u$ ($\rho_u$ represents the ratio of unfactored moments of resistance of the support strip and the panel, whereas $\rho_d$ represents the ratio of their factored values in which partial safety factors are included). Effectively, the designer uses more conservatism in the case of steel fibres because they are assigned the lower partial material safety factor of plain concrete.

Although $\rho_d$ would be appropriate in design, $\rho_u$ is used in the development of the theory which follows since it represents true behaviour.

## 4.25 Rectangular interior panel

### 4.25.1 Strength of an interior panel

In the case of a rectangular interior panel (Fig. 4.10), the plastic moment of resistance can be calculated for each of the two orthogonal axes separately. Assuming that edge effects are insignificant, the panel can be analysed by postulating orthogonal symmetrical yield lines at the crack locations. These yield lines are independent of the span ratio $\epsilon$ and of the ratio of reinforcement and $\rho$ values.

Firstly, consider the overall yielding of the panel — Section 4.25.3 considers the development of local yield lines which is an alternative failure mode: both need to be considered. The design moment of resistance of the panel, $\boldsymbol{R}_{\mathrm{d,m}}$, expressed as a load per unit area is obtained as follows:

$$\boldsymbol{R}_{\mathrm{d,m}} = \min\,(\boldsymbol{R}_{\mathrm{d,mx}}, \boldsymbol{R}_{\mathrm{d,my}}) \tag{4.25}$$

With:

$$\boldsymbol{R}_{\mathrm{d,my}} = \frac{\lambda_{\mathrm{M},x}}{a^2x}(1 + \rho_{\mathrm{d},x}) \times m_{\mathrm{Rd,f}} \tag{4.26}$$

$$\boldsymbol{R}_{\mathrm{d,mx}} = \frac{\lambda_{\mathrm{M},y}}{a^2y}(1 + \rho_{\mathrm{d},y}) \times m_{\mathrm{Rd,f}} \tag{4.27}$$

where

$$\lambda_{\mathrm{M},x} = \lambda_{\mathrm{M},y} = 16 \text{ in the case of overall panel yield line failure} \tag{4.28}$$

$$\rho_{\mathrm{d},x} = \frac{M_{\mathrm{Rd,s}x}}{M_{\mathrm{Rd,f}x}} \tag{4.29}$$

$$\rho_{\mathrm{d},y} = \frac{M_{\mathrm{Rd,s}y}}{M_{\mathrm{Rd,f}y}} \tag{4.30}$$

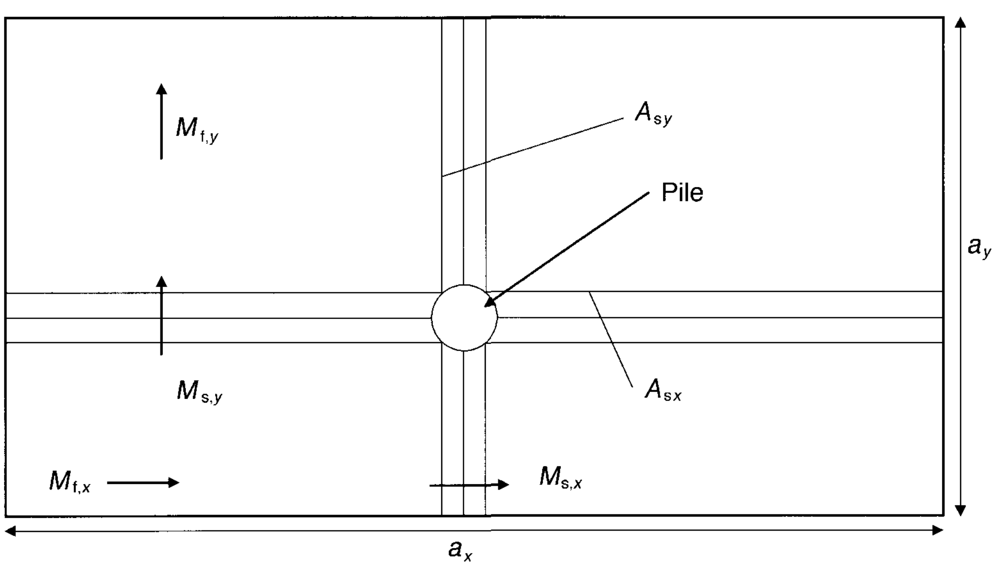



*Fig. 4.10. Plan of rectangular interior panel*

$m_{\mathrm{Rd,f}}$ = moment of resistance per unit length of the fibre reinforced panel (kN-m/m)
$M_{\mathrm{Rd,s},x}$ = moment of resistance of the support strip about the $x$-axis (kN-m)
$M_{\mathrm{Rd,f},x}$ = moment of resistance of the fibre reinforced panel about the $x$-axis (kN-m)
$M_{\mathrm{Rd,s},y}$ = moment of resistance of the support strip about the $y$-axis (kN-m)
$M_{\mathrm{Rd,f},y}$ = moment of resistance of the fibre reinforced panel about the $y$-axis (kN-m)

$M_{\mathrm{Rd,s},x}$ and $M_{\mathrm{Rd,s},y}$ are obtained from Table 4.7. $M_{\mathrm{Rd,f},x}$ and $M_{\mathrm{Rd,f},y}$ values are obtained by multiplying $m_{\mathrm{Rd,f}}$ values by the respective panel side lengths in the $x$ and $y$ directions. See Section 4.33 for a worked example of this procedure.

The design load $S_{\mathrm{d,m}}$ (kN/m$^2$) has to be less than or equal to the resistance of the panel:

$$\boldsymbol{R}_{\mathrm{d,m}} \geq S_{\mathrm{d,m}} \tag{4.31}$$

$S_{\mathrm{d,m}}$ is obtained from:

$$(\boldsymbol{g}_{\mathrm{k}} \times \gamma_{\mathrm{G}}) + (\boldsymbol{q}_{k} \times \gamma_{\mathrm{Q}}) \tag{4.32}$$

### 4.25.2 *Influence of pile dimensions*

In the case of interior panels, the cross-sectional dimensions of the pile may have an important influence on the strength. The pile size can be taken into account as follows. Assume that the interior panel is square or nearly so ($\epsilon < 1{\cdot}3$) and that the panel is isotropic, i.e. it has similar properties in two orthogonal directions so that $\boldsymbol{R}_{\mathrm{d,m}x} = \boldsymbol{R}_{\mathrm{d,m}y}$. If $d_{\mathrm{p}}$ is the characteristic dimension of the pile ($d_{\mathrm{p}}$ is the side length for the square pile or the diameter for the circular pile) and $a_y$ is the longer panel dimension, then we can define the support influence factor $\xi$ by:

$$\xi = \frac{d_{\mathrm{p}}}{a_{\mathrm{y}}} \tag{4.33}$$

We can now obtain a modified load coefficient $\lambda_{\mathrm{M}}$ for an interior panel, which takes into account the pile size as follows. First calculate $\kappa$ from:

$$\kappa = \frac{3}{3 - 6\sqrt{2}\xi + 8\sqrt{2}\xi^3} \tag{4.34}$$

Then use Equation (4.35) to calculate the modified load coefficients which now replace the value of 16 used in Equation (4.28):

$$\lambda_{\mathrm{M},x} = \lambda_{\mathrm{M},y} = \kappa\lambda_{\mathrm{M}} \tag{4.35}$$

### 4.25.3 *Calculations for moment capacity assuming local yield line failure of an interior panel*

Second, consider a local yield line failure as follows. Under certain combinations of load geometry and reinforcement, local yield lines may form at the centre of an interior panel which do not propagate to the peripheral regions of the panel. Since this situation may be the critical one, it is necessary to undertake an analysis of its effects.

From a knowledge of grid size, pile diameter, slab thickness and support strip reinforcement cage width, the lengths of the local yield lines can be determined with sufficient accuracy. First calculate $a_{x,\mathrm{i}}$ and $a_{y,\mathrm{i}}$ from Equations (4.36) and (4.37).

$$a_{x\mathrm{i}} = a_{\mathrm{x}} - \max\,(d_{\mathrm{p}}, b_{\mathrm{b}}) \tag{4.36}$$

$$a_{y\mathrm{i}} = a_{y} - \max\,(d_{\mathrm{p}}, b_{\mathrm{b}}) \tag{4.37}$$

where
$d_{\mathrm{p}}$ = diameter (or side length for a square pile) of pile
$b_{\mathrm{b}}$ = width of support strip reinforcement cage (measured over outer stirrups).

For a rectangular interior panel, the moment of resistance in the case of local centre panel yielding is calculated from:

$$\boldsymbol{R}_{\mathrm{d,mi}} = \frac{\lambda_{\mathrm{Mi}}}{a_{x\mathrm{i}}^{2}} \times m_{\mathrm{Rd,f}} \tag{4.38}$$

with:

$$\lambda_{\mathrm{M,i}} = 16\frac{1+\dfrac{1}{2\eta\epsilon_{\mathrm{i}}^{2}}}{1-\dfrac{2}{3}\eta} \tag{4.39}$$

Values of $M_{\mathrm{Rd,s},x}$ and $M_{\mathrm{Rd,s,y}}$ are shown in Table 4.10 and these values are used to determine $\boldsymbol{R}_{\mathrm{d,mi}}$ in Equation (4.38).

## 4.26 Rectangular edge panel

### *4.26.1 Calculating moment of resistance for an edge panel*

As with an internal panel, the two cases of overall yield line failure and local yield line failure need to be considered. In the case of a panel supported along one edge and with

*Table 4.10. Support strip moment of resistance values $M_{\mathrm{Rd,s},x}$ and $M_{\mathrm{Rd,s},y}$ for rectangular internal panel yield line failure*

| $\varepsilon_{\mathrm{i}} = a_{y,\mathrm{i}}/a_{x,\mathrm{i}}$ | $M_{\mathrm{Rd,s},x}$ and $M_{\mathrm{Rd,s,y}}(\lambda_{\mathrm{M,I}})$ | Tension, $\eta$ |
|---|---|---|
| 1·00 | 48·00 | 0·50 |
| 1·10 | 43·78 | 0·48 |
| 1·20 | 40·49 | 0·45 |
| 1·30 | 37·86 | 0·43 |
| 1·40 | 35·72 | 0·41 |
| 1·50 | 33·94 | 0·40 |
| 1·60 | 32·44 | 0·38 |
| 1·75 | 30·60 | 0·36 |
| 2·00 | 28·28 | 0·33 |
| $\infty$ | 16·00 | 0·00 |

its neighbouring panels having similar spans, several yield line patterns need to be considered. In the following analysis, it is assumed that the edge is free to rotate and that there is a row of piles along the edge.

The Braunschweig work has identified which of the yield line patterns is critical for various conditions. Tables 4.10, 4.11 and 4.12 summarise this work and the values in these tables refer to the critical yield line pattern for each combination of variables. To use Tables 4.10, 4.11 and 4.12, first calculate the span ratio $\epsilon_{\mathrm{E}}$ from:

$$\epsilon_{\mathrm{E}} = \frac{a_y}{a_{x\mathrm{E}}} \qquad \text{span ratio} \tag{4.40}$$

The local coordinate system is such that the $x$ direction is normal to the edge. Now calculate $\Lambda$ from:

$$\Lambda = \frac{\rho_{\mathrm{k},x}}{\rho_{\mathrm{k},y}} \tag{4.41}$$

where

$$\rho_{\mathrm{k},x} = \frac{M_{\mathrm{k,s},x}}{M_{\mathrm{k,f},x}} \tag{4.42}$$

$$\rho_{\mathrm{k},xy} = \frac{M_{\mathrm{k,s},y}}{M_{\mathrm{k,f},y}} \tag{4.43}$$

*Table 4.11. Support strip moment of resistance values $M_{\mathrm{k,s},x}$ and $M_{\mathrm{k,s},y}$ for rectangular edge panel yield line failure*

| $\varepsilon$ | $M_{\mathrm{k,s},x}$ and $M_{\mathrm{k,s},y}(\lambda_{\mathrm{E,l}})$ | $\eta$ |
|---|---|---|
| 0·75 | 38·82 | 0·500 |
| 1·00 | 29·35 | 0·452 |
| 1·25 | 24·62 | 0·395 |
| 1·50 | 21·83 | 0·350 |
| 2·00 | 18·72 | 0·283 |
| $\infty$ | 11·66 | 0 |

*Table 4.12. Load factors $\lambda_c$ for corner panels*

| $\rho_{\mathrm{k},y} = \rho_{\mathrm{k},x}$ | $\lambda_c$ |
|---|---|
| 0·00 | 13·67 |
| 0·50 | 15·08 |
| 0·75 | 15·40 |
| 1·00 | 15·62 |
| 1·25 | 15·79 |
| 1·50 | 15·76 |

In the case of the edge panel, use Table 4.11 and read the value of $\lambda_E$. Now introduce this value of $\lambda_E$ in Equation (4.44) to obtain the design moment of resistance of the edge panel:

$$\boldsymbol{R}_{d,m,E} = \frac{\lambda_E}{a_{x,E}^2}(1 + \rho_{d,y}) \times m_{Rd,f} \tag{4.44}$$

### 4.26.2 *Calculations for moment capacity in the case of local yield line failure for an edge panel*

The lengths of local yield lines can be determined in the case of an edge panel from a knowledge of the panel dimensions, the pile diameter, the slab thickness and the width of the support strip reinforcement case. First, calculate the reduced effective panel dimensions from:

$$a_{x,i} = a_{x,E} - \max\ (d_p, b_b)/2 \tag{4.45}$$

$$a_{y,i} = a_{yE} - \max\ (d_p, b_b) \tag{4.46}$$

where

$a_{x,E}$ = distance between pile centres normal to the edge
$a_y$ = distance between pile centres along the edge
$d_p$ = pile diameter (or side length for a square pile)
$b_b$ = width of support strip reinforcement cage, including stirrups.

Table 4.11 can now be used to determine $M_{k,s,x}$ and $M_{k,s,y}$ which can be substituted into Equation (4.47) to determine the moment of resistance of an edge panel for the partial yield line case:

$$\boldsymbol{R}_{d,m,Ei} = \frac{\lambda_{E,i}}{a_{x,i}^2} \times m_{Rd,f} \tag{4.47}$$

## 4.27 Corner panel

### 4.27.1 *Calculating moment of resistance for a corner panel*

To simplify the calculation, a rectangular corner panel is considered to be square with all sides equal to the longer side of the rectangle. The pile dimension can be assumed to have no influence on the panel's bending strength. Firstly, consider overall yield line failure. Equation (4.48) can be used conservatively to obtain the bending strength of the corner panel:

$$\boldsymbol{R}_{d,m,C} = \frac{\lambda_C}{a_c^2}(1 + \rho_{d,y}) \times m_{Rd,f} \tag{4.48}$$

The value of $\lambda_c$ required in Equation (4.48) is taken from Table 4.12.

### 4.27.2 *Calculations for moment capacity in the case of local yield line failure for a corner panel*

Two types of yield line need to be considered in the local failure for a corner panel. The first type is orthogonal negative yield lines and the second type is diagonal positive yield lines. The lengths of the yield lines can be determined from a knowledge of the pile spacings, pile size, slab thickness and width of support strip reinforcement cage. First, determine the reduced effective corner slab width from Equation (4.49):

$$a_i = a_e - \max\ (d_p, b_b)/2 \qquad (4.49)$$

where

$a_e$ = length of longer edge of corner panel
$d_p$ = pile diameter or side length
$b_b$ = width of support strip reinforcement cage including stirrup.

Second, calculate the moment of resistance of the locally yielding panel from:

$$\boldsymbol{R}_{d,m,Ci} = \frac{\lambda_{c,i}}{a^2} \times m_{Rd,f} \quad (\text{with } \lambda_{c,i} = 34{\cdot}97) \qquad (4.50)$$

## 4.28 Calculations for point loads

Depending on their magnitude and location, point loads may generate greater stresses than uniformly distributed loads. Their influence can be assessed by converting them into equivalent uniformly distributed loads as follows. To be able to make this conversion, it has to be assumed that any local yield lines that might have developed in the vicinity of the point load would be less critical than the yield lines developed by the equivalent uniformly distributed load. This is true when point loads are applied near the centre of a panel or near positive yield lines (positive yield lines are ones which form a valley).

The equivalency of point loads with uniformly distributed loads is as follows. Let:

$\boldsymbol{F}$ = point load

$a_x, a_y$ = panel dimensions

Then Equation (4.51) is used to determine the equivalent uniformly distributed load:

$$q_{\text{Ersatz}} = 2\frac{\boldsymbol{F}}{a_x a_y} \qquad (4.51)$$

The equation gives an exact equivalence in the case of point loads applied at the centre or at the centre of the support strip of an interior panel. Equation (4.51) can in fact be applied for a point load applied anywhere along an interior panel support strip. In that case, the equivalent distributed load should be applied only when undertaking calculations for the loaded support strip. In the case of partial yield line failure, the reduced effective dimensions of the panel have to be used resulting in Equation (4.52):

$$q_{\text{Ersatz innerfield}} = 2\frac{\boldsymbol{F}}{a_{x,i} a_{y,i}} \qquad (4.52)$$

In the case of point loads applied onto edge or corner panels, the above procedure may be unconservative and the equivalent distributed load should be increased to 15% above the value produced by Equation (4.51).

## 4.29 Preliminary design

In order to streamline the design procedure, Fig. 4.11 can be used to investigate the sensitivity of design inputs. Figure 4.11 allows the bending strength of an internal panel to be assessed for different combinations of pile grid dimensions, slab thickness, steel fibre type and dosage, and support strip reinforcement.

The following variables are entered:

- in the first quadrant, the stress is entered depending on pile grid size and restraint value — the dead load is taken into account by varying the slab thickness between 200 mm ($p_k = 5\,\text{kN/m}^2$) and 400 mm ($p_k = 10\,\text{kN/m}^2$)
- in the second quadrant the reinforcement required is related to the stress $m_{sd}$ and strength $m_{Rd}$ — in this quadrant, the ultimate load check that $\boldsymbol{R}_{d,m} \geq \boldsymbol{S}_{d,m}$ can be made

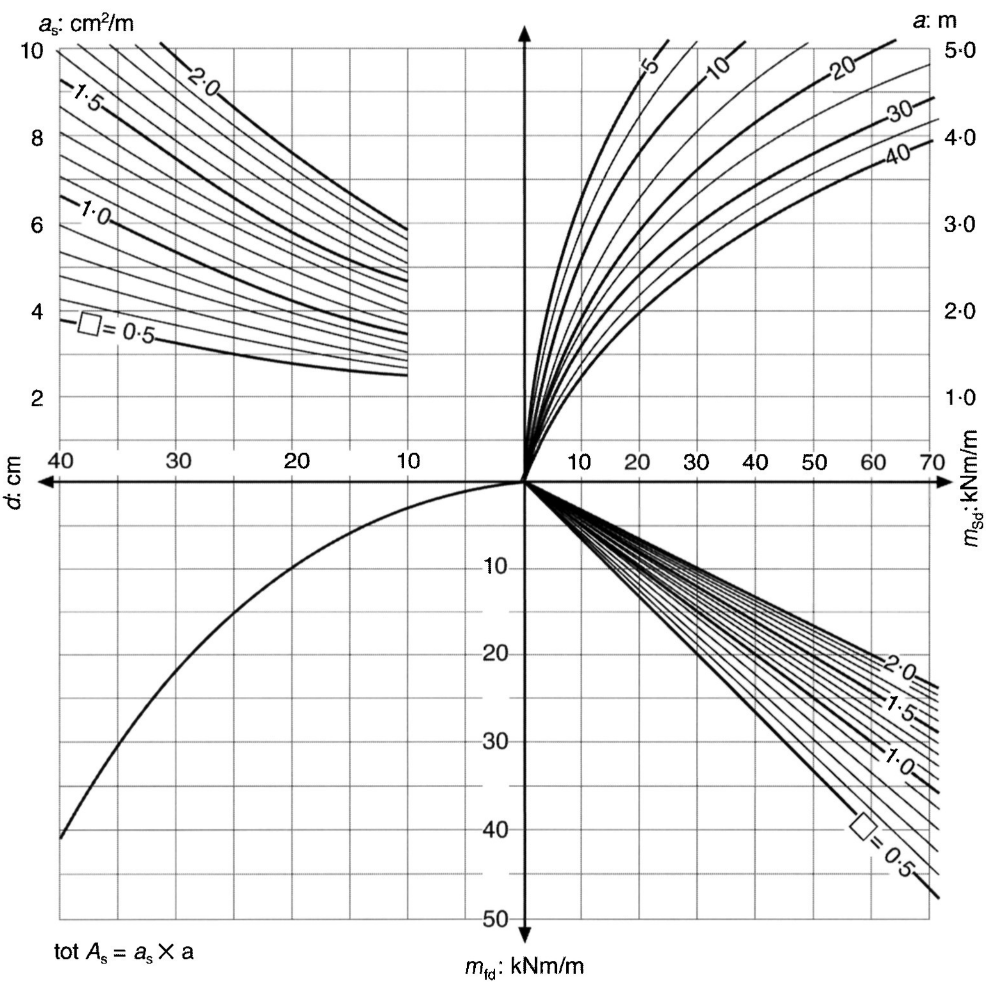



*Fig. 4.11. Preliminary design nomogram*

- in the third quadrant, the moment of resistance of steel fibre reinforced C30/37 concrete with an equivalent bending strength $f_{fct,eq,150} = 2{\cdot}70\,N/mm^2$ can be investigated
- in the fourth quadrant, the reinforcement required can be found.

Figure 4.12 is a flow chart illustrating the use of the nomogram. The procedure is as follows:

1. Select the curve in the first quadrant corresponding with the loading.
2. Select pile spacings in each orthogonal direction.
3. Project a vertical line from the first quadrant to the fourth.
4. Read off slab thickness.
5. Determine the strength corresponding with the steel fibre dosage in the third quadrant and project the line into the fourth quadrant. Read the steel reinforcement requirement ($\rho$) at the intersection with the vertical axis.
6. Select the line corresponding with the $\rho$ value determined in 5.
7. Select the reinforcement required $a_s$ (cm$^2$/m).
8. The total support strip reinforcement required (tot $A_s$) is obtained by multiplying $a_s$ by the vertical distance between the plane of restraint and the point of fixity of the pile.
9. It may be possible to undertake exact calculations to reduce the amount of support strip reinforcement — the reduction is achieved by multiplying the area of steel by the support influence factor, $\kappa$.

## 4.30 Support strip reinforcement detailing

Plastic analysis produces an overall moment to be resisted which can be allocated to the centre of the strip and the pile supports according to the designer's redistribution preferences. Often, the designer provides similar reinforcement at all of the peak moment locations. The total support strip reinforcement (tot $A_s$) can be proportioned according to the following principles:

tot $A_s$ = total area of reinforcement required
$A_{s,S,o}$ = steel reinforcement at top of pile
$A_{s,S,u}$ = steel reinforcement at bottom of pile
$A_{s,F,o}$ = steel reinforcement at top mid-way between piles
$A_{s,F,u}$ = steel reinforcement at bottom mid-way between piles.

### *4.30.1 Support strips surrounding interior panels*

For interior panels, the support strip reinforcement can be proportioned as follows.

Dead load reinforcement:

$A_{s,S,o} = 0{\cdot}7(\text{tot } A_s)$
$A_{s,F,u} = 0{\cdot}3(\text{tot } A_s)$

Imposed load reinforcement:

$$A_{s,S,u} \geq 0{\cdot}5A_{s,F,u}$$
$$A_{s,F,o} \geq 0{\cdot}5A_{s,S,o}$$

All of the bottom steel reinforcement is turned downwards through 90° and anchored to the pile. Also, 50% of the area of steel has to run continuously over the pile. In the case of different panel widths, support strip reinforcement is determined from the panel subjected to the highest stress level.

### 4.30.2 Support strips surrounding edge and corner panels

For edge and corner panels, the support-strip reinforcement is distributed equally between the panel support strip and the first interior panel support strip.

Dead load reinforcement:

$$A_{s,S,o} = 0{\cdot}5(\text{tot } A_s)$$
$$A_{s,F,u} = 0{\cdot}5(\text{tot } A_s)$$

Imposed load reinforcement:

$$A_{s,F,o} > 0{\cdot}5A_{s,s,o}$$

In the case of non-square corner and edge panels, the critical support strip reinforcement is determined by considering the longer span support strip. The bottom dead load panel support strip reinforcement $A_{s,F,u}$ is continued at its full area from the edge support to the first in-board pile. The bottom imposed load reinforcement is calculated by considering the first interior panel and using the reinforcement which that panel requires.

### 4.30.3 Length of support strip dead load reinforcement

The support strip top reinforcement should have sufficient bond length. For both flat-slab and mushroom slab design, the top reinforcement can be determined by elastic section analysis. It can be assumed that a bond length of $0{\cdot}3 \times a$ using an amount of reinforcement equal to $A_{s,S,o}$ is conservative — beyond that length the top reinforcement is reduced to $A_{s,F,o}$.

## 4.31 Piled floor design example

The design method previously explained is now illustrated by an example.

Consider a warehouse pile supported ground floor slab with piles on a $3{\cdot}6 \times 3{\cdot}0$ m grid subjected to a moderately heavy load, $p_k$ of 20 kN/m$^2$. The edge panels are supported on frost-resistant sub-base material whose design is not considered in this example.

There are no specific performance criteria and it is assumed that cracks wider than 0·4 mm are permissible (in practice, such cracks may need to be sealed). It is assumed that the floor comprises a simple rectangle with no structural complications.

## 4.32 Piled floor preliminary design

Figure 4.11 can be used to undertake a preliminary design for an interior panel. Separate calculations can be undertaken for support strip reinforcement in the two orthogonal directions. Assume that C30/37 concrete reinforced with 40 kg/m$^3$ steel fibre is to be used. This material has a flexural tensile strength $f_{fct,eq,150}$ of 2·7 N/mm$^2$. These values are used in the preliminary design chart Fig. 4.11 — see Fig. 4.12.

Selected slab thickness:

$h = 25$ cm

From the preliminary design chart, the total reinforcement values are:

$x$ direction: $a_s = 4{\cdot}7\ \text{cm}^2/\text{m}$, tot $A_{s,x} = 4{\cdot}7\ \text{cm}^2/\text{m}$ For $3{\cdot}00\ \text{m} = 14{\cdot}1\ \text{cm}^2$

$y$ direction: $a_s = 2{\cdot}8\ \text{cm}^2/\text{m}$, tot $A_{s,y} = 2{\cdot}8\ \text{cm}^2/\text{m}$ For $3{\cdot}60\ \text{m} = 10{\cdot}1\ \text{cm}^2$

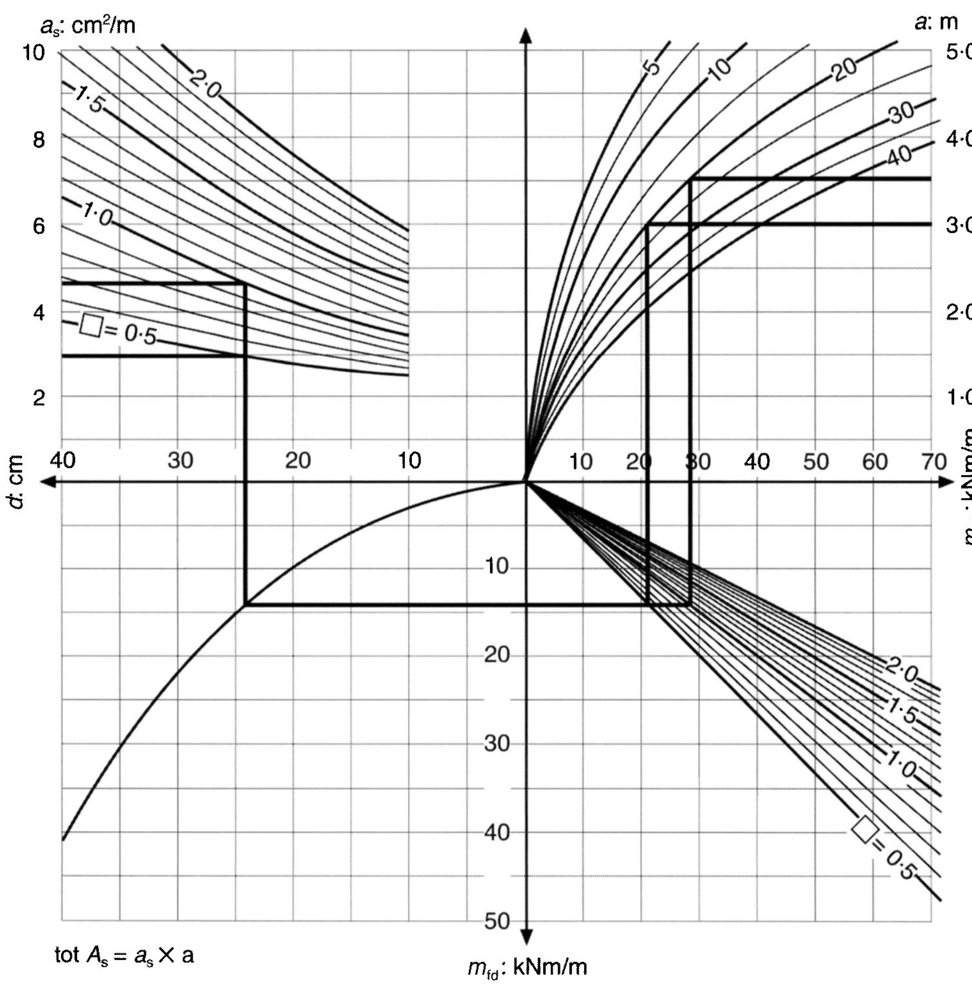



*Fig. 4.12. Typical example using the preliminary design nomogram*

As shown in Fig. 4.12, the trial slab thickness:

$h = 250\,\text{mm}$

Considering the effect of the pile diameter:

$$\varepsilon = \frac{a_x}{a_y} = \frac{3{\cdot}6}{3} = 1{\cdot}2 \leq 1{\cdot}3$$

(this means that the panel is sufficiently square)

$$\xi = \frac{d_\text{p}}{a_x} = \frac{200}{3600} = 0{\cdot}056$$

$$\kappa = \frac{3}{3 - 6\sqrt{2\xi} + 8\sqrt{2\xi^3}} = 1{\cdot}187$$

Therefore, enhance the load by 18·7% when considering the interior panel's local yield line pattern:

$$\lambda_{\text{M},x} = \lambda_{\text{M},y} = \kappa\lambda_\text{M} = 1{\cdot}187 \times 16 = 18{\cdot}99$$

## 4.33 Bending strength calculations for a piled floor interior panel

### *4.33.1 Calculation of stress in slab resulting from applied moment*

$$S_{\text{d,m}} = \boldsymbol{g}\gamma_\text{G} + \boldsymbol{q}\gamma_\text{Q} = 0{\cdot}25\,\text{m} \times 25\,\text{kN/m}^2 \times 1{\cdot}35 + 20\,\text{kN/m}^2 \times 1{\cdot}5$$
$$= 38{\cdot}44\,\text{kN/m}^2$$

Firstly, consider overall yield line failure of the panel.

Determine the flexural strength of the interior panel for reinforcement in the $x$ direction, i.e. bending about the $y$ axis:

$m_{\text{Rd,f}} = 16{\cdot}00\,\text{kN-m/m}$ (Equation (4.28))

$M_{\text{Rd,f},y} = 16{\cdot}00\,\text{kN-m/m} \times 3{\cdot}00\,\text{m} = 48\,\text{kN/m}$ (multiply $m_{\text{Rd,f}}$ by the length of the side of the panel)

tot $A^*_{\text{s},x} = A_{\text{s},x}/\kappa = 14{\cdot}1/1{\cdot}187 = 11{\cdot}9\,\text{cm}^2$ (this accounts for the pile thickness)

$A_\text{s} =$ tot $A^*_{\text{s},y}/2 = 5{\cdot}95\,\text{cm}^2$

$M_{\text{Rd,s},y} = 43{\cdot}3\,\text{kN-m}$ (interpolate from Table (4.10))

$\rho_{\text{d},y} = 43{\cdot}3/48 = 0{\cdot}9$

$$\boldsymbol{R}_{\text{d,my}} = \frac{\lambda_{\text{M},x}}{a_x^2}(1 + \rho_{\text{d},y}) \times m_{\text{Rd,f}} = \frac{18{\cdot}99}{3{\cdot}6^2\,\text{m}^2}(1 + 0{\cdot}9) \times 16{\cdot}00\,\text{kN-m/m}$$
$$= 44{\cdot}54\,\text{kN/m}^2$$

Now, determine the flexural strength of the interior panel for reinforcement in the $y$ direction, i.e. bending about the $x$ axis:

$$m_{\mathrm{Rd,f}} = 16{\cdot}00\,\mathrm{kN\text{-}m/m}\ \text{(Equation (4.28))}$$

$$M_{\mathrm{Rd,f},x} = 16{\cdot}00\,\mathrm{kN\text{-}m/m} \times 3{\cdot}60\,\mathrm{m} = 57{\cdot}6\,\mathrm{kN\text{-}m}$$

$$\mathrm{tot}\ A^{*}_{\mathrm{s},y} = A_{\mathrm{s},y}/\kappa = 10{\cdot}1/1{\cdot}187 = 8{\cdot}5\,\mathrm{cm}^2\ \text{(pile thickness considered)}$$

$$A_{\mathrm{s}} = \mathrm{tot}\ A^{*}_{\mathrm{s},y}/2 = 4{\cdot}25\,\mathrm{cm}^2$$

$$M_{\mathrm{Rd,s},x} = 30{\cdot}8\,\mathrm{kN\text{-}m}\ \text{(interpolate from Table (4.10))}$$

$$\rho_{\mathrm{d},x} = 30{\cdot}8/57{\cdot}6 = 0{\cdot}54$$

$$\boldsymbol{R}_{\mathrm{d,m}x} = \frac{\lambda_{\mathrm{M},y}}{a_y^2}(1+\rho_{\mathrm{d},x}) \times m_{\mathrm{Rd,f}} = \frac{18{\cdot}99}{3{\cdot}00^2\,\mathrm{m}^2}(1+0{\cdot}54) \times 16{\cdot}00\,\mathrm{kN\text{-}m/m}$$
$$= 51{\cdot}99\,\mathrm{kN/m}^2$$

### *4.33.2 Check the bending strength of the interior panel in both directions*

$$\boldsymbol{R}_{\mathrm{d,m}} = \min\,(\boldsymbol{R}_{\mathrm{d,m}x},\ \boldsymbol{R}_{\mathrm{d,m}y}) = 44{\cdot}54\,\mathrm{kN/m}^2 > 38{\cdot}44 = \boldsymbol{S}_{\mathrm{d,m}}$$

Now consider the flexural strength of an interior panel for local yield line failure:

$d_{\mathrm{p}} = 20\,\mathrm{cm}$ (pile diameter)

$b_{\mathrm{b}} = 30\,\mathrm{cm}$ (width of cage of reinforcement)

Relevant value: 30 cm

$$a_{x,\mathrm{i}} = 3{\cdot}60\,\mathrm{m} - 0{\cdot}30\,\mathrm{m} = 3{\cdot}30\,\mathrm{m}$$
$$a_{y,\mathrm{i}} = 3{\cdot}00\,\mathrm{m} - 0{\cdot}30\,\mathrm{m} = 2{\cdot}70\,\mathrm{m}$$

$$\varepsilon_{\mathrm{i}} = \frac{a_{x,\mathrm{i}}}{a_{y,\mathrm{i}}} = \frac{3{\cdot}30}{2{\cdot}70} = 1{\cdot}22$$

$$\lambda_{\mathrm{m,I}} = 39{\cdot}96\ \text{(Table 4.10)}$$

$$\boldsymbol{R}_{\mathrm{d,mi}} = \frac{\lambda_{\mathrm{M,i}}}{a_{\mathrm{x,i}}^2} \times m_{\mathrm{Rd,f}} = \frac{16{\cdot}00}{3{\cdot}30} \times 39{\cdot}96 = 58{\cdot}7\,\mathrm{kN/m}^2 > \boldsymbol{S}_{\mathrm{d}}$$

## 4.34 Bending strength calculations for a piled floor edge panel

In the following calculations edge panels with lengths of 2·8 m and 3·0 m are considered. The reinforcement is calculated for the longer dimension and is not reduced in the case of the other shorter dimension.

### 4.34.1 Calculations for flexural strength of entire edge panel

$$\varepsilon = \frac{a_y}{a_{x,e}} = \frac{3}{2 \cdot 8} = 1 \cdot 07 \text{ (see Fig. 4.6)}$$

Calculation of orthotropic factor $\Lambda$ from characteristic values $\rho_{k,y}$ and $\rho_{k,x}$:

$$m_{k,f} = 24 \cdot 00 \, \text{kN-m/m}$$

$$A_{s,x} = \text{tot } A_{s,x}/2 = 7 \cdot 0 \, \text{cm}^2$$

$$M_{k,s,y} = 58 \cdot 7 \, \text{kN-m (Table 4.11)}$$

$$M_{k,f,y} = m_{k,f} \times a_y = 24 \cdot 00 \, \text{kN-m/m} \times 3 \cdot 00 \, \text{m} = 72 \cdot 0 \, \text{kN-m}$$

$$A_{s,y} = 5 \cdot 75 \, \text{cm}^2$$

$$M_{k,s,x} = 48 \cdot 05 \, \text{kN-m (interpolate from Table 4.11)}$$

$$M_{k,s,x} = m_{k,f} \times (a_{x,E} + a_x) = 24 \cdot 00 \, \text{kN-m/m} \times (2 \cdot 80 \, \text{m} + 3 \cdot 60 \, \text{m}) = 76 \cdot 8 \, \text{kN-m}$$

$$\rho_{k,y} = \frac{M_{k,s,y}}{M_{k,f,y}} = \frac{58 \cdot 7}{72 \cdot 0} = 0 \cdot 82$$

$$\rho_{k,x} = \frac{M_{k,s,x}}{M_{k,f,x}} = \frac{48 \cdot 05}{76 \cdot 8} = 0 \cdot 63$$

$$\Lambda = \frac{\rho_{k,x}}{\rho_{k,y}} = \frac{0 \cdot 63}{0 \cdot 82} = 0 \cdot 77$$

To obtain the value for $\lambda_E$, interpolate from Table 4.11 for $\varepsilon = 1 \cdot 07$, $\rho_{k,y} = 0 \cdot 8$ and $\Lambda = 0 \cdot 75$. This gives $\lambda_E = 11 \cdot 1$.

In order to use Equation (4.43) to obtain the flexural strength of the panel, a value for $\rho_{d,y}$ is required:

$$m_{Rd,f} = 16 \cdot 00 \, \text{kN-m/m}$$

$$A_{s,x} = \text{tot } A_{s,x}/2 = 7 \cdot 0 \, \text{cm}^2$$

$$M_{Rd,s,y} = 50 \cdot 72 \, \text{kN-m (Table 4.11)}$$

$$M_{Rd,f,y} = m_{Rd,f} \times a_y = 16 \cdot 00 \, \text{kN-m/m} \times 3 \cdot 00 \, \text{m} = 48 \cdot 0 \, \text{kN-m}$$

$$\rho_{d,y} = \frac{M_{Rd,s,y}}{M_{Rd,s,y}} = \frac{50 \cdot 72}{48 \cdot 0} = 1 \cdot 05$$

$$\boldsymbol{R}_{d,m,E} = \frac{\lambda_E}{a_{x,E}^2}(1 + \rho_{d,y}) \times m_{Rd,f} = \frac{11 \cdot 1}{2 \cdot 80^2 \, \text{m}^2}(1 + 1 \cdot 05) \times 16 \cdot 00 \, \text{kN-m/m}$$

$$= 46 \cdot 4 \, \text{kN/m}^2 (\boldsymbol{R}_{d,m,E} > \boldsymbol{S}_{d,m} \text{ demonstrated})$$

Calculation of flexural strength of the panel in the case of local yield line failure:

$d_p = 200\,\text{mm}$ (pile diameter)

$b_b = 300\,\text{mm}$ (width of reinforcement assembly)

$a_{x,i} = 2{\cdot}80\,\text{m} - 0{\cdot}30\,\text{m}/2 = 2{\cdot}65\,\text{m}$

$a_{y,i} = 3{\cdot}00\,\text{m} - 0{\cdot}30\,\text{m} = 2{\cdot}70\,\text{m}$

$$\varepsilon_i = \frac{a_{x,i}}{a_{y,i}} = \frac{2{\cdot}70}{2{\cdot}65} = 1{\cdot}02$$

$\lambda_{E,i} = 29{\cdot}35$ (Table 4.11)

$$\mathbf{R}_{d,E,i} = \frac{\lambda_{E,i}}{a_{x,i}^2} \times m_{Rd,f} = \frac{29{\cdot}35}{2{\cdot}65^2} \times 16{\cdot}00 = 66{\cdot}8\,\text{kN/m}^2 > S_d \text{ (proven)}$$

## 4.35 Bending strength calculations for a piled floor corner panel

Consider a square corner panel with a side length of 2·8 m. Length of neighbouring edge panels = max $(a_x,\ a_y) = 3{\cdot}6\,\text{m}$.

Calculations for flexural strength of a corner panel assuming overall yield line failure:

$m_{k,f} = 24{\cdot}00\,\text{kN-m/m}$

$A_{s,x} = A_{s,y} = \text{tot } A_s/2 = 7{\cdot}0\,\text{cm}^2$

$M_{k,s} = 58{\cdot}7\,\text{kN-m}$

$M_{k,f} = m_{k,f} \times (a_c + a) = 24 \times (2{\cdot}80 + 3{\cdot}60) = 76{\cdot}8\,\text{kN-m}$

$$\rho_k = \frac{M_{k,s}}{M_{k,f}} = \frac{58{\cdot}7}{76{\cdot}8} = 0{\cdot}76$$

$\lambda_c = 15{\cdot}4$ (Table 4.12)

$M_{Rd,s} = 50{\cdot}7\,\text{kN/m}$

$M_{Rd,f} = 16{\cdot}00\,\text{kN-m/m}$

$M_{Rd,f} = m_{Rd,f} \times (a_c + a) = 16{\cdot}00 \times (2{\cdot}80 + 3{\cdot}60) = 51{\cdot}2\,\text{kN-m}$

$$\rho_d = \frac{M_{Rd,s}}{M_{Rd,f}} = \frac{50{\cdot}7}{51{\cdot}2} = 0{\cdot}99$$

$$\boldsymbol{R}_{\text{d,m,c}} = \frac{\lambda_c}{a_c}(1+\rho_\text{d}) \times m_{\text{Rd,f}} = \frac{15\cdot4}{2\cdot80^2\,\text{m}^2}(1+0\cdot99) \times 16\cdot00\,\text{kN-m/m}$$

$$= 62\cdot5\,\text{kN/m}^2$$

$\boldsymbol{R}_{\text{d,m,C}} > \boldsymbol{S}_{\text{d,m}}$ (demonstrated)

### 4.35.1 Flexural strength calculations for a corner panel assuming local yield line failure

$$a_\text{i} = a - \max\,(d_\text{p},\ b_\text{b})/2 = 2\cdot80\,\text{m} - 0\cdot30\,\text{m}/2 = 2\cdot65\,\text{m}$$

$$\lambda_{\text{c,i}} = 34\cdot97$$

$$\boldsymbol{R}_{\text{d,m,ci}} = \frac{\lambda_{\text{c,i}}}{a^2_{\text{c,i}}} \times m_{\text{Rd,f}} = \frac{34\cdot97}{2\cdot65^2\,\text{m}^2} \times 16\cdot00\,\text{kN-m/m} = 79\cdot67\,\text{kN/m}^2$$

$\boldsymbol{R}_{\text{d,m,ci}} > \boldsymbol{S}_{\text{d,m}}$ (demonstrated)

## 4.36 Serviceability checks on piled floor design solution

### 4.36.1 Check on minimum reinforcement requirement

The negative moments are to be taken a distance $0\cdot2a$ from the support.

Thickness $h = 250\,\text{mm}$

Concrete C30/37 with $f_{\text{fct,eq,300}} = 2\cdot8\,\text{N/mm}^2$

$f_{\text{ct,eff}} = f_{\text{ctk,0}\cdot95} = 3\cdot8\,\text{N/mm}^2$

$A_{\text{ct}}$: cross-section of panel concrete (Eurocode 2, Clause 4.4.2.2)

$k = 1$, $k_c = 0\cdot4$ (pure bending)

$\boldsymbol{\sigma}_\text{s} = \boldsymbol{f}_{\text{yk}}$

$$A_{\text{s,min}} = k \times k_\text{c} \times (\boldsymbol{f}_{\text{fct,eff}} - \boldsymbol{f}_{\text{fct,eq,300}}) = \frac{A_{\text{ct}}}{\boldsymbol{\sigma}_\text{s}}$$

$x$-direction:

$a_x = 3\cdot60\,\text{m}$

$b_{\text{m},x} = 2 \times 0\cdot2 \times 3\cdot60\,\text{m} = 1\cdot44\,\text{m}$

$A_{\text{s,min},x} = 2\cdot88\,\text{cm}^2$

$y$-direction:

$a_y = 3\cdot00\,\text{m}$

$b_{\text{m},y} = 2 \times 0\cdot2 \times 3\cdot00\,\text{m} = 1\cdot20\,\text{m}$

$A_{\text{s,min},y} = 2\cdot4\,\text{cm}^2$

### 4.36.2 Reinforcement check for imposed load

$$\frac{d_\text{p}}{\text{min length}} = 0\cdot0055$$

$$k^\text{g}_{\text{ss}} = -0\cdot301$$

$k^{p}_{ss} = -0{\cdot}334$

$\varepsilon = 3{\cdot}60/3{\cdot}00 = 1{\cdot}20, \; c = 0{\cdot}92$

$$m_{ss} = (-0{\cdot}301 \times 0{\cdot}92 \times 0{\cdot}25\,\text{m} \times 25\,\text{kN/m}^3 - 0{\cdot}334 \times 0{\cdot}92 \times 20\,\text{kN/m}^2) \times 3{\cdot}60^2\,\text{m}^2 = -102\,\text{kN-m/m}$$

Partial steel fibre effect $m_{k,f} = 24\,\text{kN-m/m}$
Proportion to be taken by reinforcing steel: $m_{ss,eff} = 102 - 24 = 79\,\text{kN-m/m}$
Related to an actual width of $0{\cdot}2a$, the result is:

$M_{k,s} = 0{\cdot}2 \times 3{\cdot}00\,\text{m} \times 78\,\text{kNm/m} = 46{\cdot}8\,\text{kN-m}$

$A_{s,S,o,x} = 6\,\text{cm}^2$

### *4.36.3 Interior support strips running in the y-direction*

$$\frac{d_p}{\text{min length}} = 0{\cdot}0055$$

$\varepsilon = 3{\cdot}00/3{\cdot}60 = 0{\cdot}83, \; c = 1{\cdot}28$

$$m_{ss} = (-0{\cdot}301 \times 1{\cdot}28 \times 0{\cdot}25\,\text{m} \times 25\,\text{kN/m}^3 - 0{\cdot}334 \times 1{\cdot}28 \times 20\,\text{kN/m}^2) \times 3{\cdot}00^2\,\text{m}^2 = -77\,\text{kN/m}$$

Partial steel fibre effect $m_{k,f} = 24\,\text{kN-m/m}$
Proportion to be taken by the reinforcing steel: $m_{ss,eff} = 77 - 24 = 53\,\text{kN-m/m}$
Related to an actual width of $0{\cdot}2a$, the result is:

$M_{k,s} = 0{\cdot}2 \times 3{\cdot}60\,\text{m} \times 53\,\text{kN-m/m} = 38{\cdot}2\,\text{kN-m}$

$A_{s,S,o,y} = 5\,\text{cm}^2$

## 4.37 Piled floor deflexion check

The following calculations show that for the panel sizes in this example, the imposed load does not cause cracking. Determine the moments within panel strips.

$\varepsilon = 3{\cdot}60/3{\cdot}00 = 1{\cdot}20$

$k^{g}_{FF} = 0{\cdot}043$

$k^{p}_{FF} = 0{\cdot}083$

$$m_{FF} = (k^{g}_{FF} \times \boldsymbol{g} + k^{p}_{FF} \times \boldsymbol{p}) \times l^2$$

$$m_{FF} = (0{\cdot}043 \times 0{\cdot}25\,\text{m} \times 25\,\text{kN/m}^3 + 0{\cdot}083 \times 30\,\text{kN/m}^2) \times 3{\cdot}60^2\,\text{m}^2 = 25\,\text{kN-m/m}$$

Characteristic moment capacity from steel fibre effect ($\alpha = 1{\cdot}0$):

$$m_k = f_{fct,eq,300} \times W_{el}$$

$$m_k = 2{\cdot}9\,\text{MN/m}^2 \times 0{\cdot}25^2\,\text{m}^2/6 = 30\,\text{kN-m/m}$$

$$m_k > m_{FF} \text{ (demonstrated)}$$

## 4.38 Selection of piled floor reinforcement

It has been shown that the preliminary design support strip reinforcement levels are satisfactory. It has been shown that slight reductions from those preliminary values are possible but this is not considered in this example. Tables 4.13–4.15 show how the method set out in Section 4.8 is used to select an arrangement of reinforcement.

Figure 4.13 shows a typical detail of the support strip reinforcement. In this case, the reinforcement comprises 12 mm diameter bars throughout.

*Table 4.13. Centre field, reinforcement in x-direction (tot $A_{s,x} = 11{\cdot}50\,\text{cm}^2$)*

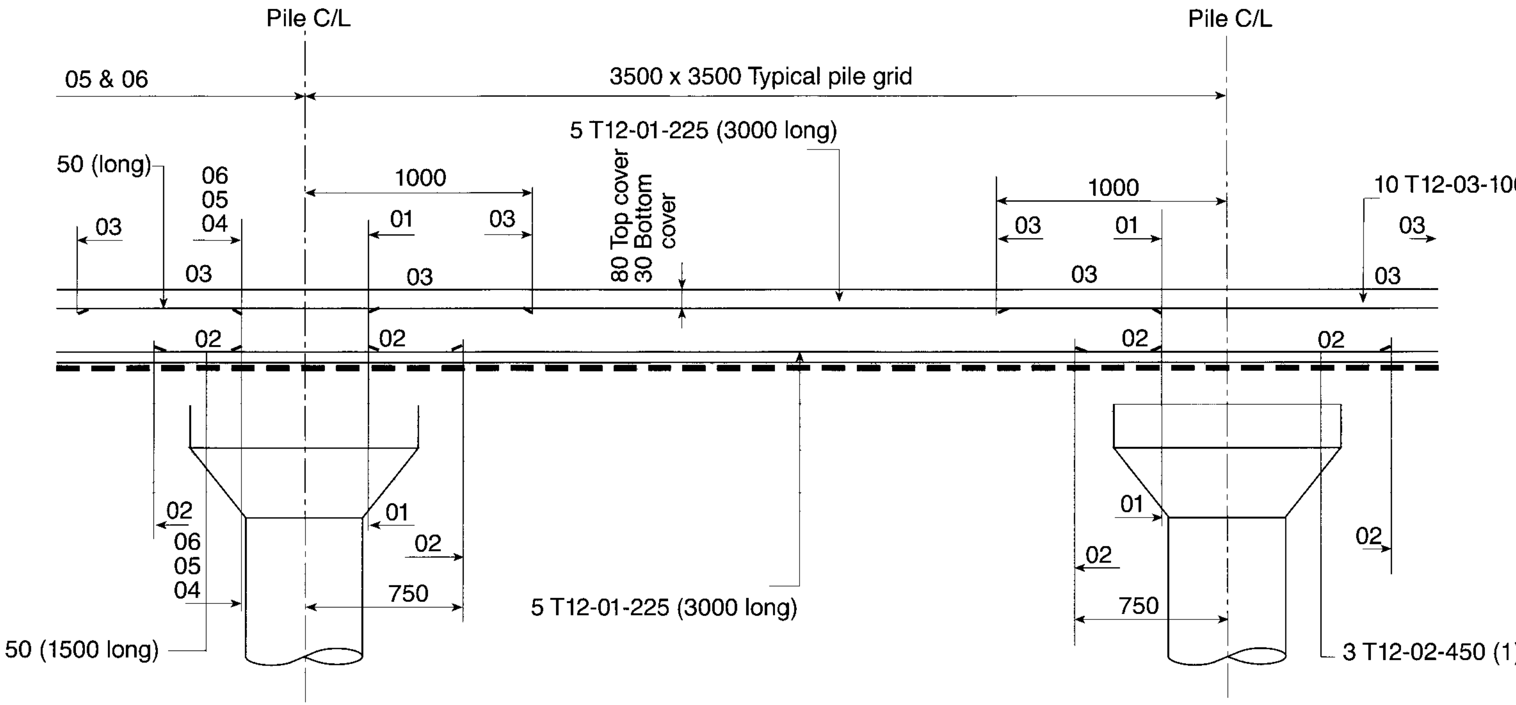
| Indication position | Part $A_s$/tot $A_s$ | Found $A_s$: cm$^2$ | Selected | Existing $A_s$: cm$^2$ |
|---|---|---|---|---|
| $A_{s,S,o}$ | 0·7 | 8·05 | 4 Ø 16 | 8·04 |
| $A_{s,F,o}$ | 0·5 × 0·7 | 4·03 | 2 Ø 16 | 4·02 |
| $A_{s,F,u}$ | 0·3 | 3·45 | 4 Ø 14 | 6·16 |
| $A_{s,S,u}$ | 0·5 × 0·3 | 1·73 | 2 Ø 16 | 3·08 |

*Table 4.14. Centre field, reinforcement in y-direction (tot $A_{s,y} = 8{\cdot}20\,\text{cm}^2$)*

| Indication position | Part $A_s$/tot $A_s$ | Found $A_s$: cm$^2$ | Selected | Existing $A_s$: cm$^2$ |
|---|---|---|---|---|
| $A_{s,S,o}$ | 0·7 | 5·74 | 4 Ø 14 | 6·16 |
| $A_{s,F,o}$ | 0·5 × 0·7 | 2·87 | 2 Ø 14 | 3·08 |
| $A_{s,f,u}$ | 0·3 | 2·46 | 4 Ø 12 | 4·52 |
| $A_{s,S,u}$ | 0·5 × 0·3 | 1·23 | 2 Ø 12 | 2·26 |

*Table 4.15. Centre field, reinforcement to corner (tot $A_{s,x} = 14{\cdot}00\,\text{cm}^2$)*

| Indication position | Part $A_s$/tot $A_s$ | Found $A_s$: cm$^2$ | Selected | Existing $A_s$: cm$^2$ |
|---|---|---|---|---|
| $A_{s,S,o}$ | 0·5 | 7·00 | 4 Ø 16 | 8·04 |
| $A_{s,F,o}$ | 0·5 × 0·5 | 3·50 | 2 Ø 16 | 4·02 |
| $A_{s,f,u}$ | 0·5 | 7·00 | 4 Ø 16 | 8·04 |
| $A_{s,S,u}$ | 0·5 | 7·00 | 4 Ø 16 | 8·04 |




*Typical bar marks shown to denote location only. All bars central on pile centre or mid-span line*

*Fig. 4.13. Detail of steel fibre piled floor support strips. The numbers indicate bar marks. Note how the top of the pile is separated from the underside of the floor by a 100 mm thick layer of crushed rock*

## 4.39 Piled floor punching shear calculations

Punching shear can be considered either by ignoring the steel fibres or by using the method set out in Eurocode 2:

$\tau_{Rd}$ = $0{\cdot}34\,N/mm^2$ (Eurocode 2 Table 4.8, original values)
$h$ = 25 cm
$d$ = 20 cm
$d_p$ = 20 cm
$d_{crit}$ = $0{\cdot}5 \times d_p + 1{\cdot}5 \times d = 40\,cm$
$k$ = $1{\cdot}6 - d \geq 1{\cdot}0$ (German design formula Eurocode 2, Clause 4.56)
$k$ = $1{\cdot}4$
$\rho_l$ = $\sqrt{\rho_{lx}\rho_{ly}} \leq 0{\cdot}015$

$$\rho_l = \frac{\sqrt{8{\cdot}05 \times 5{\cdot}74}}{2{\cdot}40 \times 20} = 0{\cdot}0042$$

### 4.39.1 Stress

$V_{Sd}$ = $(g \times \gamma_G + q \times \gamma_Q) \times A$
$V_{Sd}$ = $(0{\cdot}25 \times 25 \times 1{\cdot}35 + 20{\cdot}0 \times 1{\cdot}50)3{\cdot}60 \times 3{\cdot}00 = 415{\cdot}1\,kN$
$\beta$ = $1{\cdot}15$
$\nu_{Sd}$ = $V_{Sd} \times \beta/u$ with $u = 2\pi d_{crit}$
$\nu_{Sd}$ = $0{\cdot}4151 \times 1{\cdot}15/(2\pi \times 0{\cdot}40) = 0{\cdot}190\,MN/m$

### 4.39.2 Shear stresses

$\nu_{Rd1}$ = $\tau_{Rd} \times k(1{\cdot}2 + 40\,\rho_l) \times d$
$\nu_{Rd1}$ = $0{\cdot}34 \times 1{\cdot}4(1{\cdot}2 + 0{\cdot}042) \times 0{\cdot}20 = 0{\cdot}130\,MN/m$
$\nu_{Sd} > \nu_{Rd1} \rightarrow$ shear reinforcement required
$\nu_{Rd2}$ = $1{\cdot}6\,\nu_{Rd1} = 1{\cdot}6 \times 0{\cdot}130 = 0{\cdot}208\,MN/m > \nu_{Sd}$

### 4.39.3 Required shear reinforcement (without steel fibre effect)

$$\Sigma A_{sw} = \frac{V_{Sd} - V_{Rd1}}{f_{yk}/\gamma_s} \times u$$

$$\Sigma A_{sw} = \frac{0{\cdot}190 - 0{\cdot}130}{500/1{\cdot}15} \times 2{\cdot}51 = 3{\cdot}46\,cm^2$$

Selected: 2 stirrups Ø 8 mm, $e$ = 10 cm, existing $A_{sw} = 4{\cdot}02\,cm^2$

## 4.40 Design Charts

This section shows a finite element derived Design Chart (Fig. 4.14) and ten floor design charts.

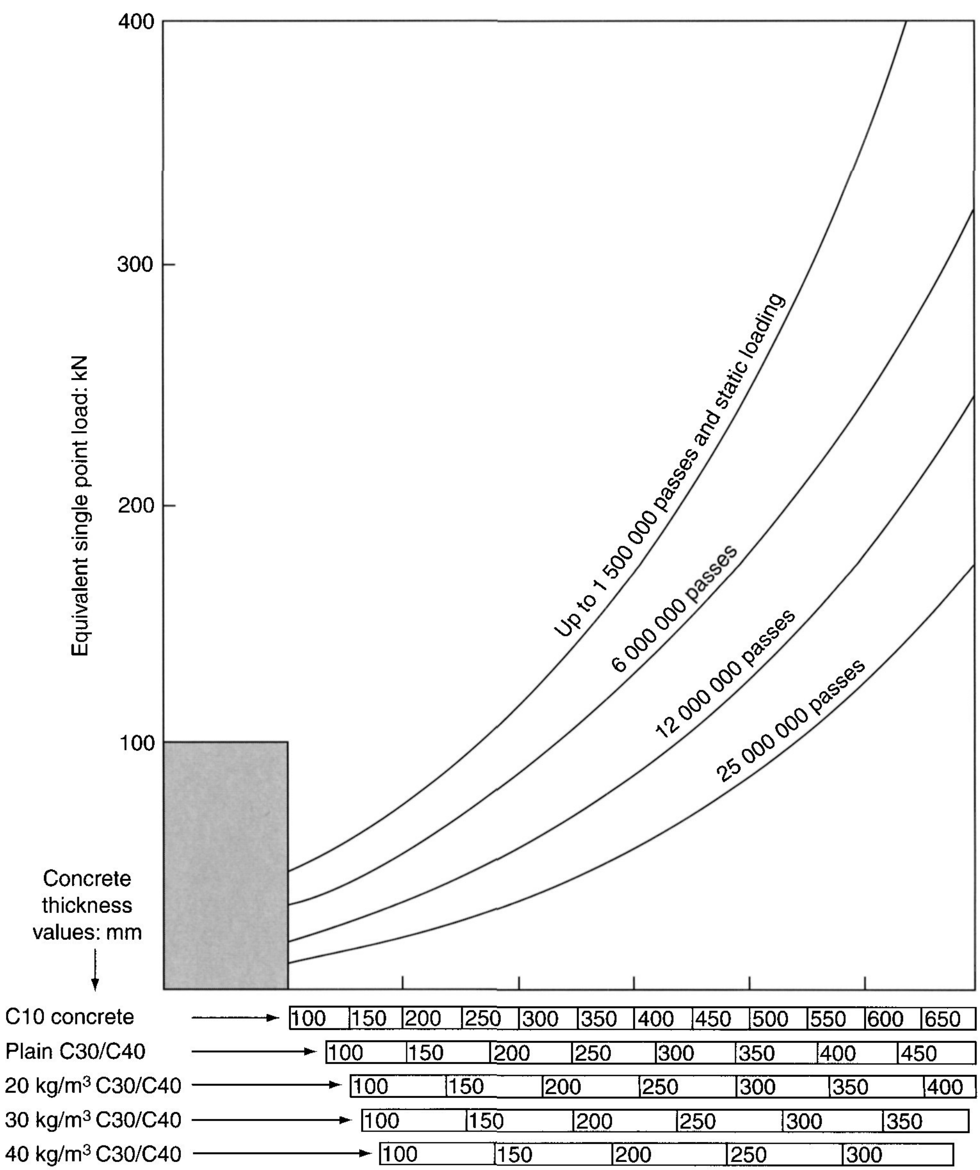
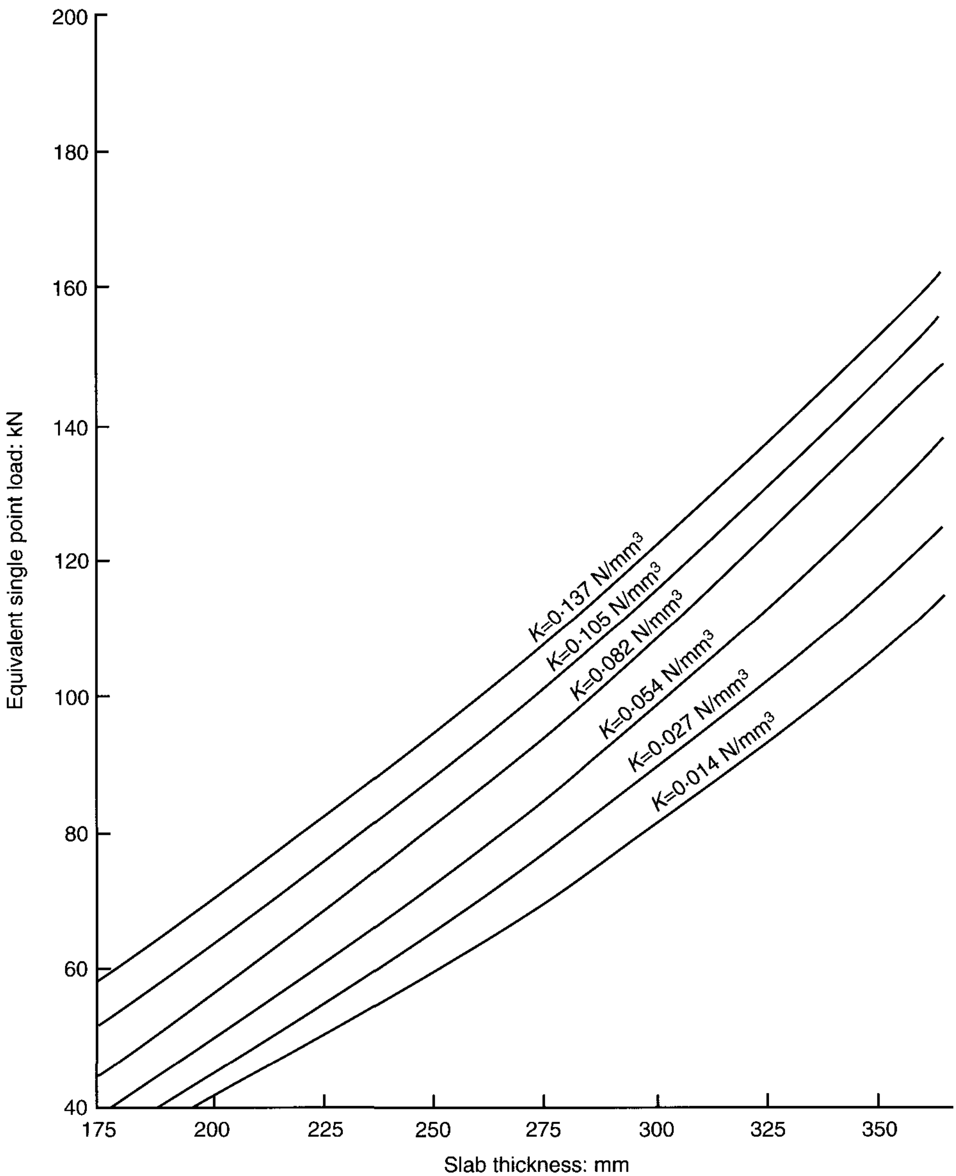



*Fig. 4.14. Point load design chart*

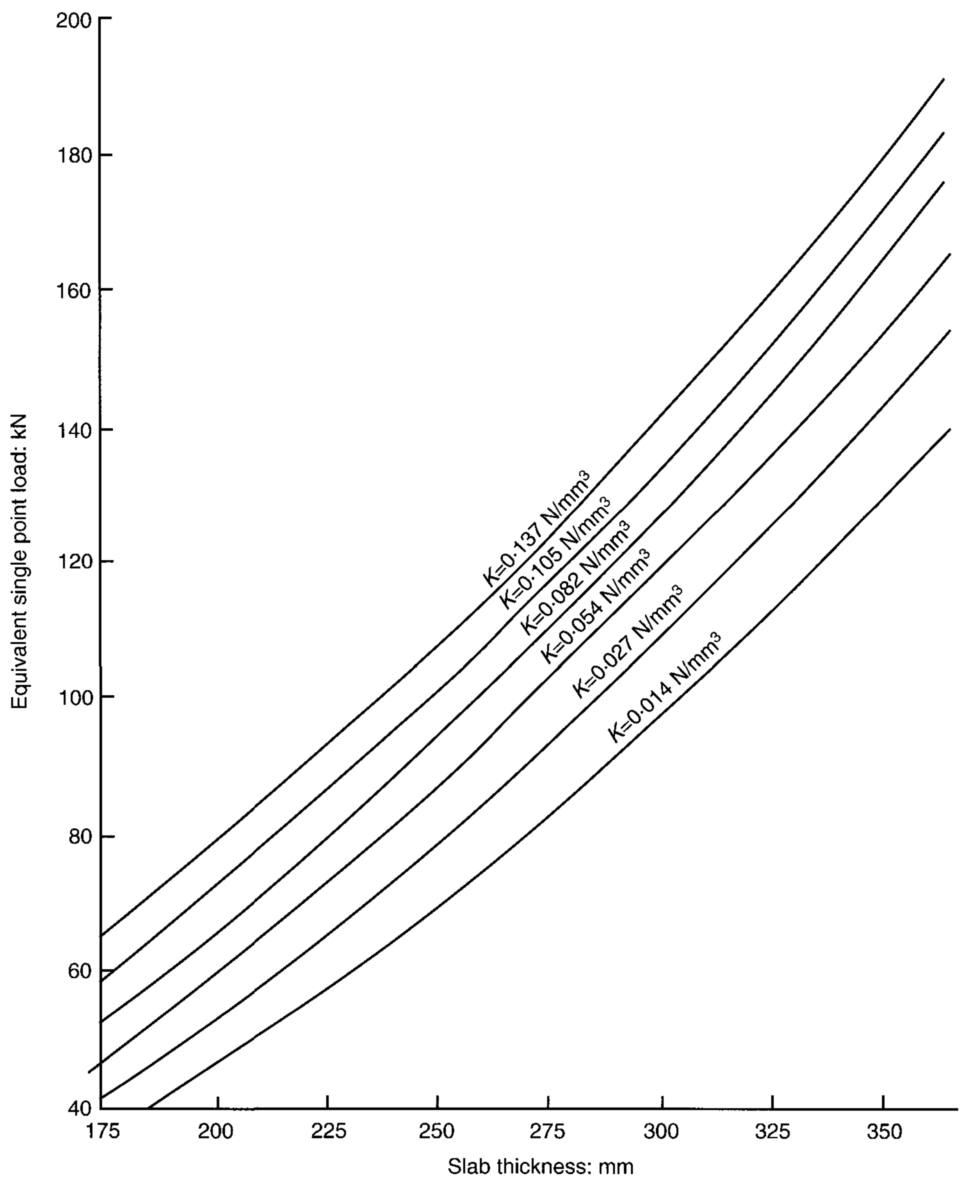
# Design Chart 1
# Plain C30 concrete

Flexural strength = $2{\cdot}0\ N/mm^2$



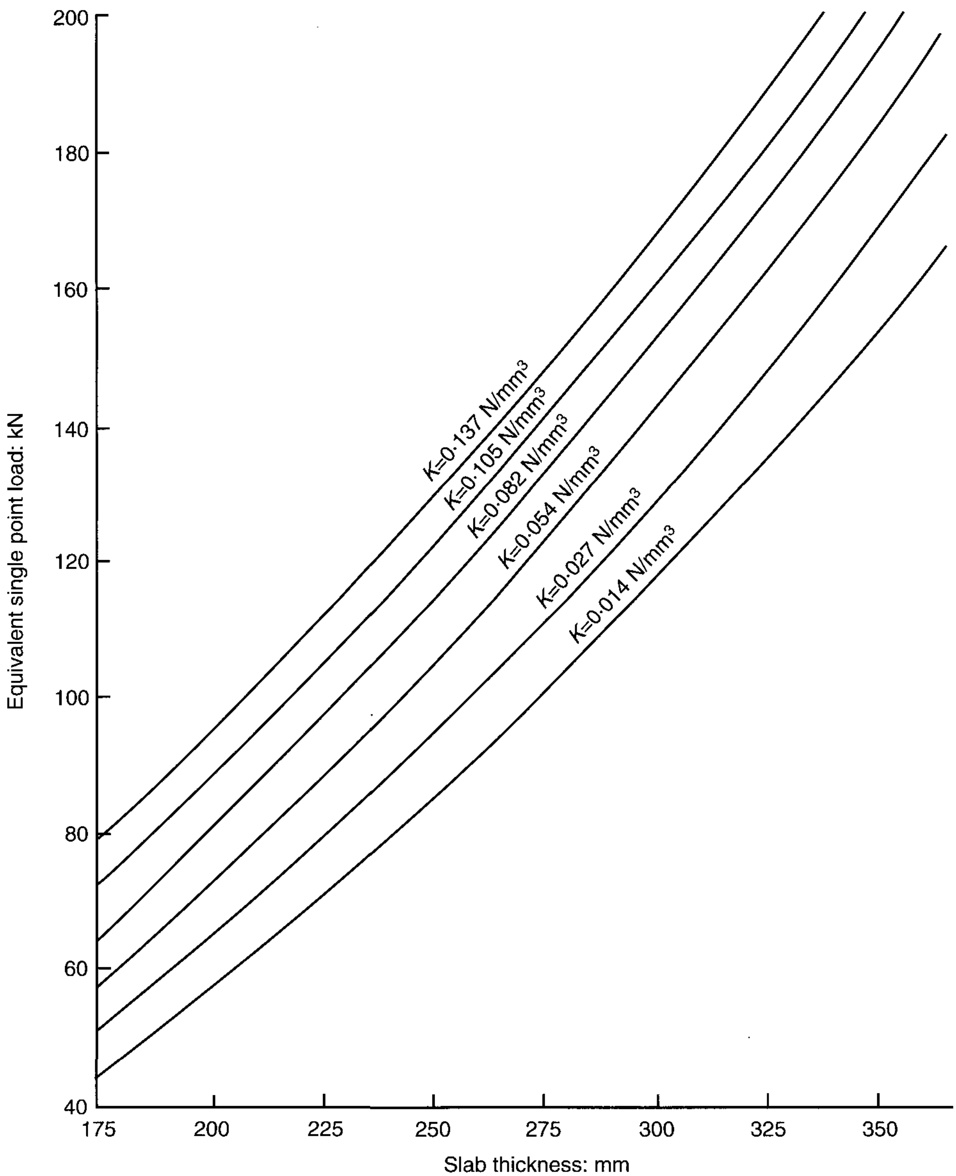
# Design Chart 2
# Micro-silica C30 concrete

Flexural strength = 2·4 N/mm$^2$



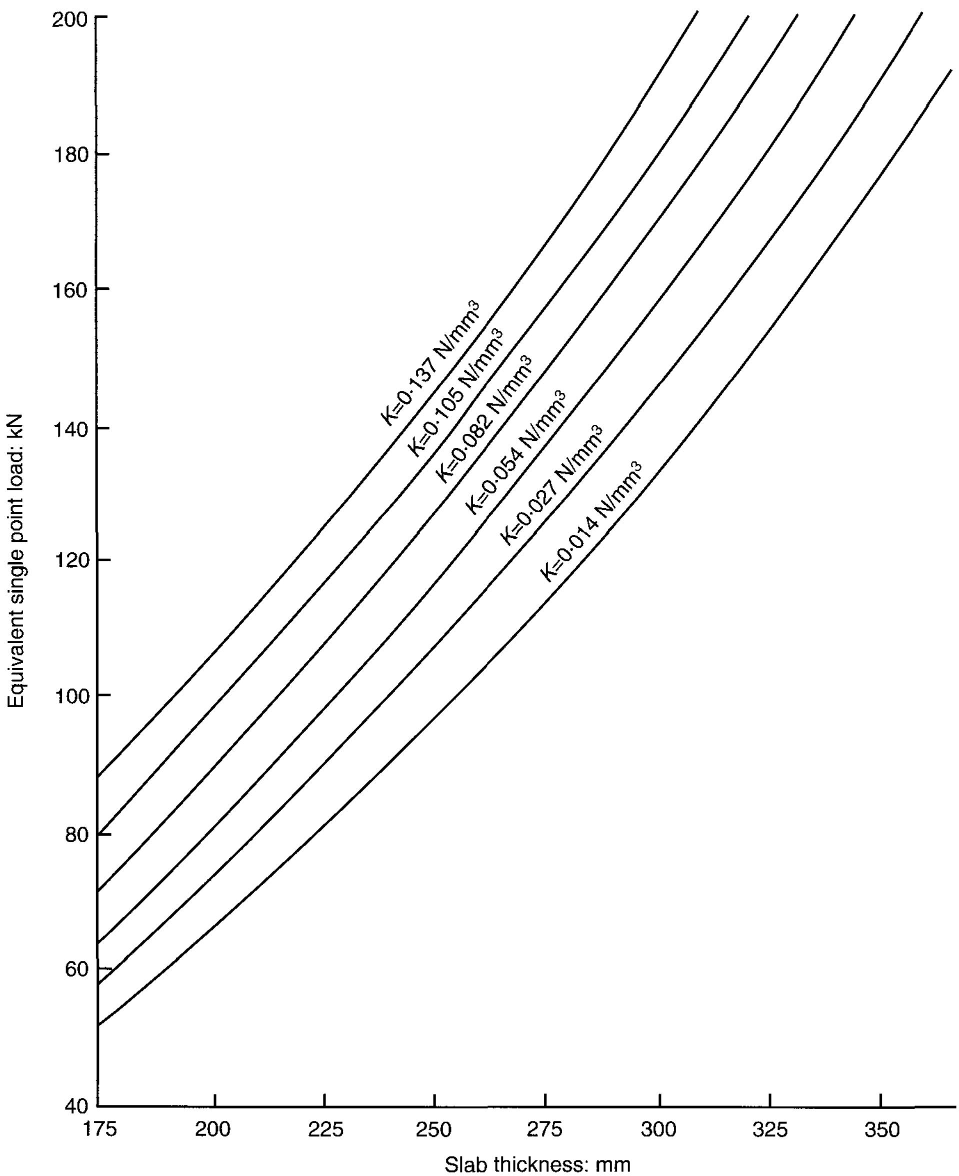
# Design Chart 3
# 20 kg/m³ ZC 60/1·00 steel fibre
# C30 concrete

Flexural strength = 2·8 N/mm²



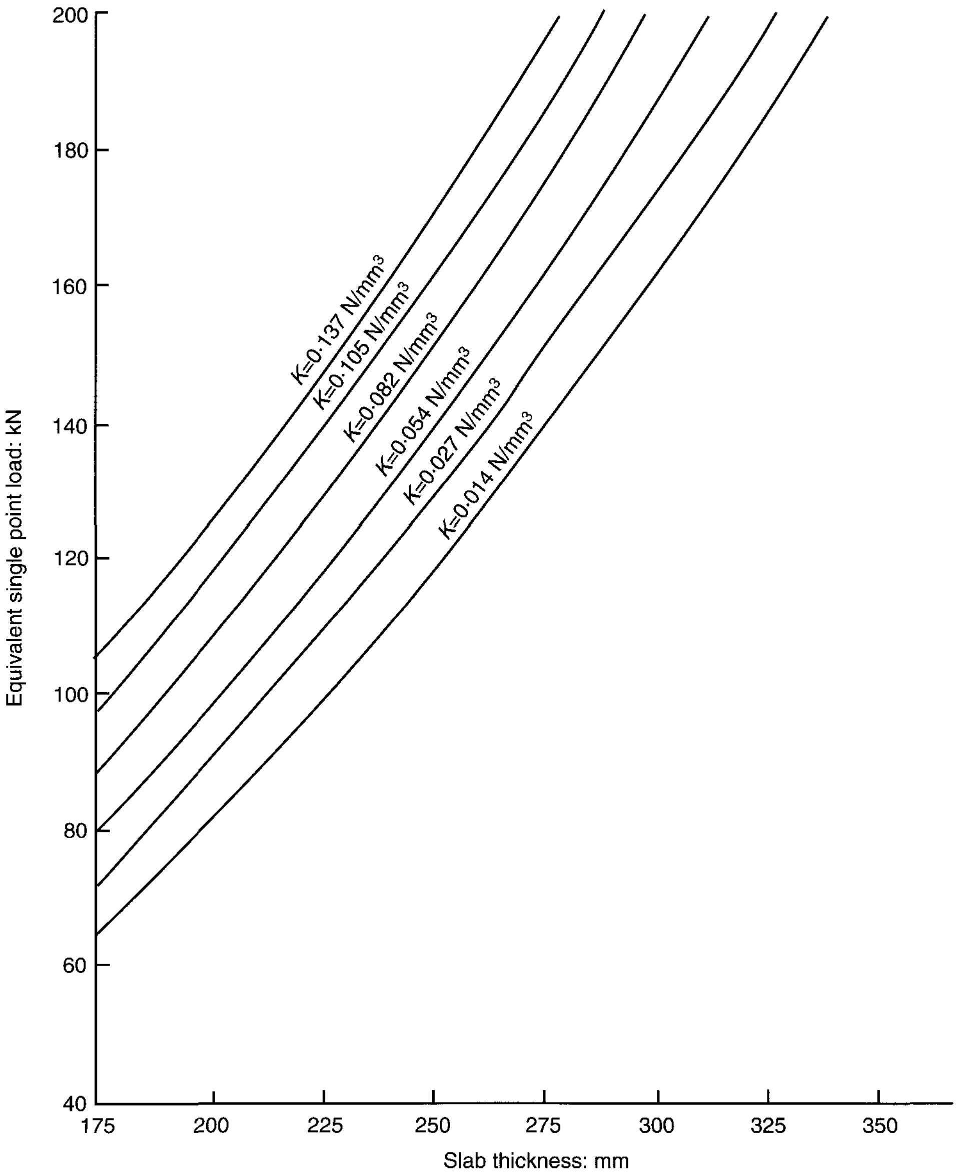
# Design Chart 4
# 30 kg/m$^3$ ZC 60/1·00 steel fibre
# C30 concrete

Flexural strength = 3·2 N/mm$^2$



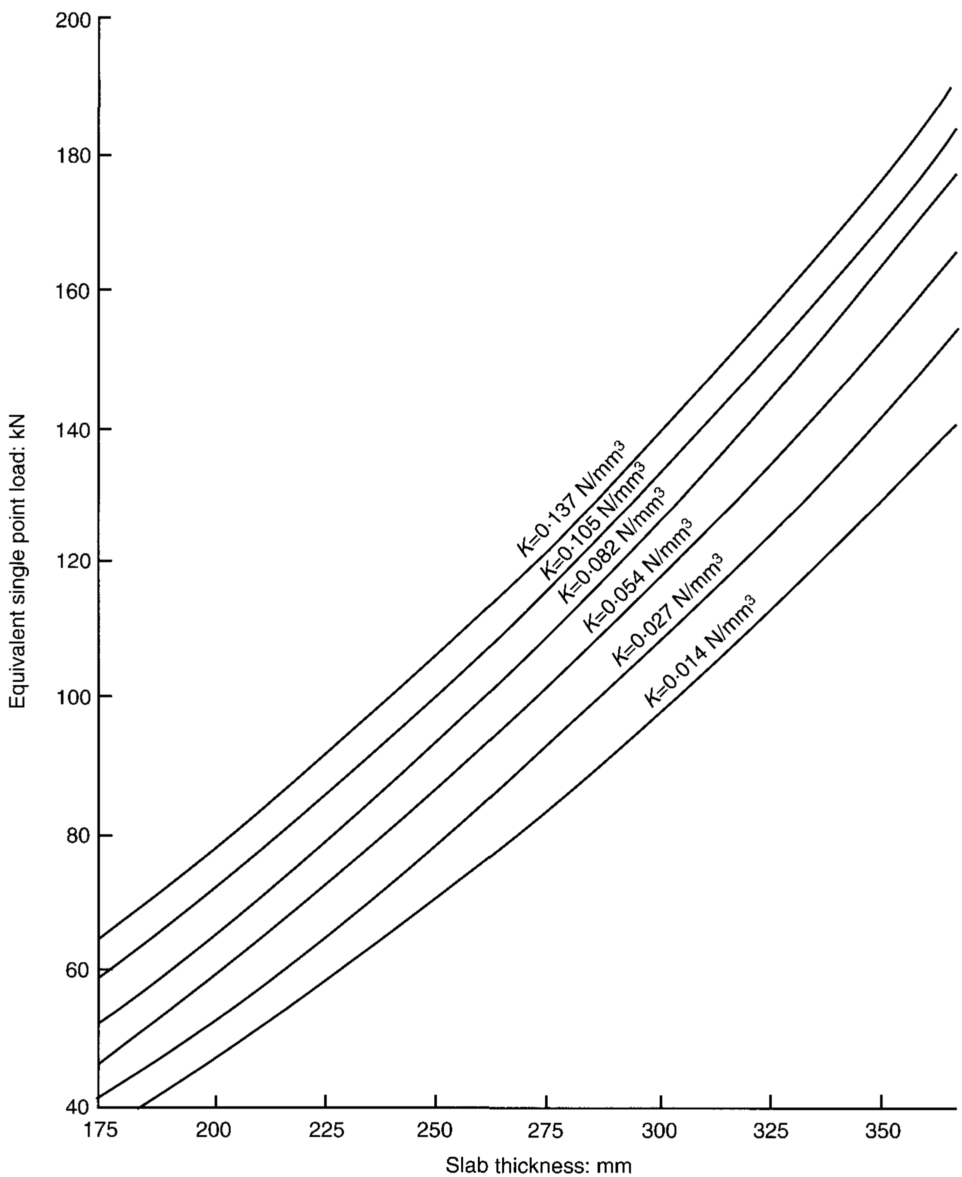
# Design Chart 5
# 40 kg/m$^3$ ZC 60/1·00 steel fibre
# C30 concrete

Flexural strength = 3·8 N/mm$^2$



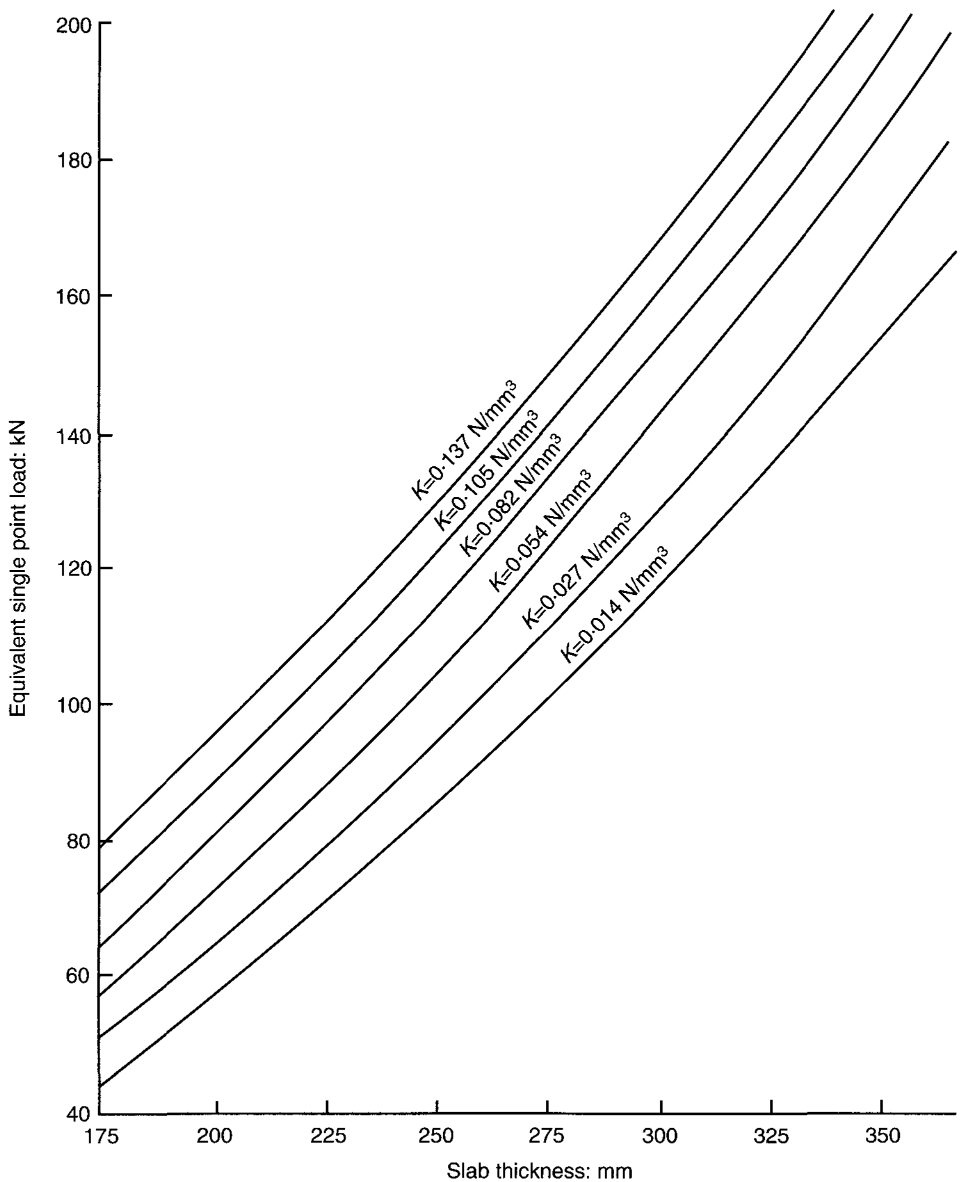
# Design Chart 6
# Plain C40 concrete

Flexural strength = $2{\cdot}4\ N/mm^2$



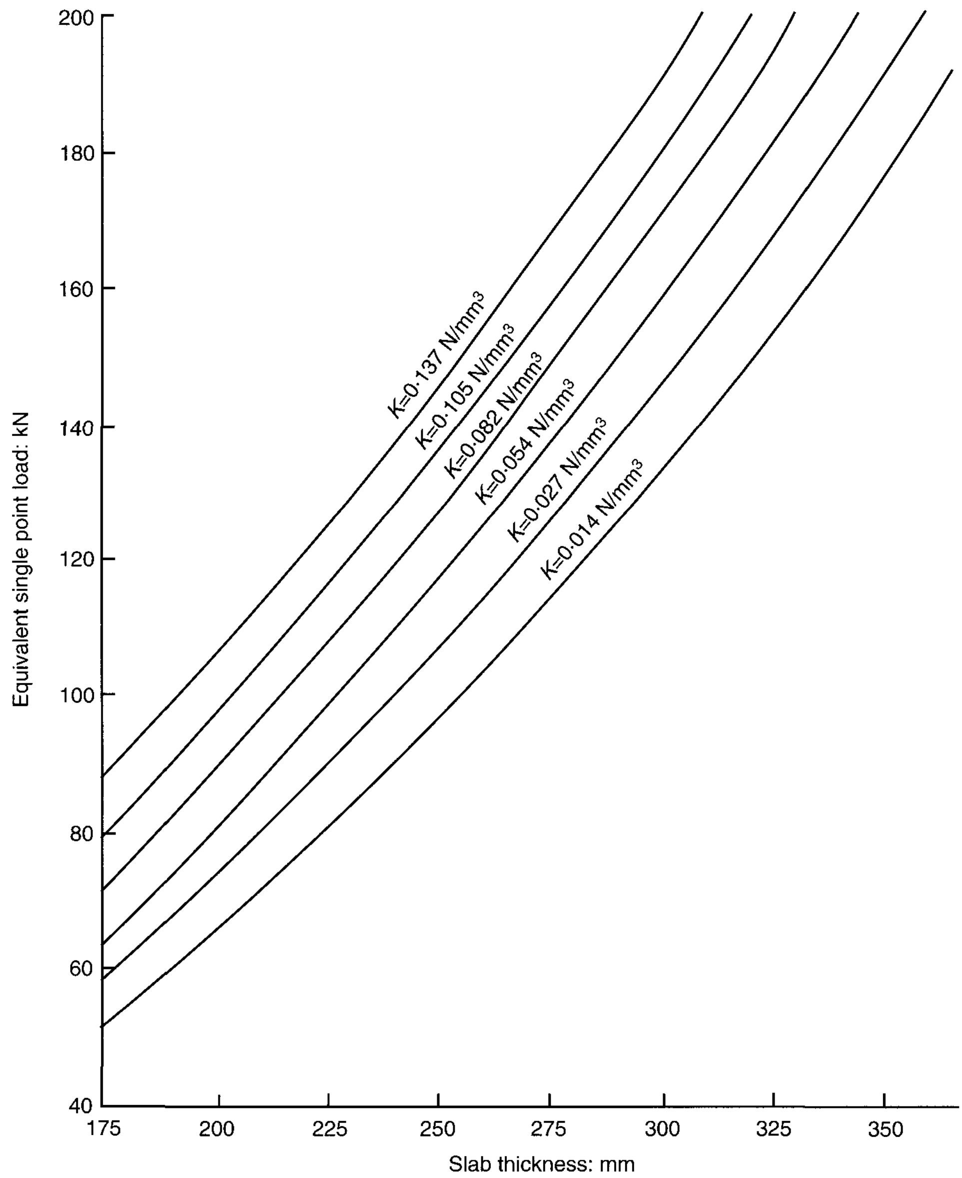
# Design Chart 7
# Micro-silicaC40 concrete

Flexural strength = $2{\cdot}8\ N/mm^2$



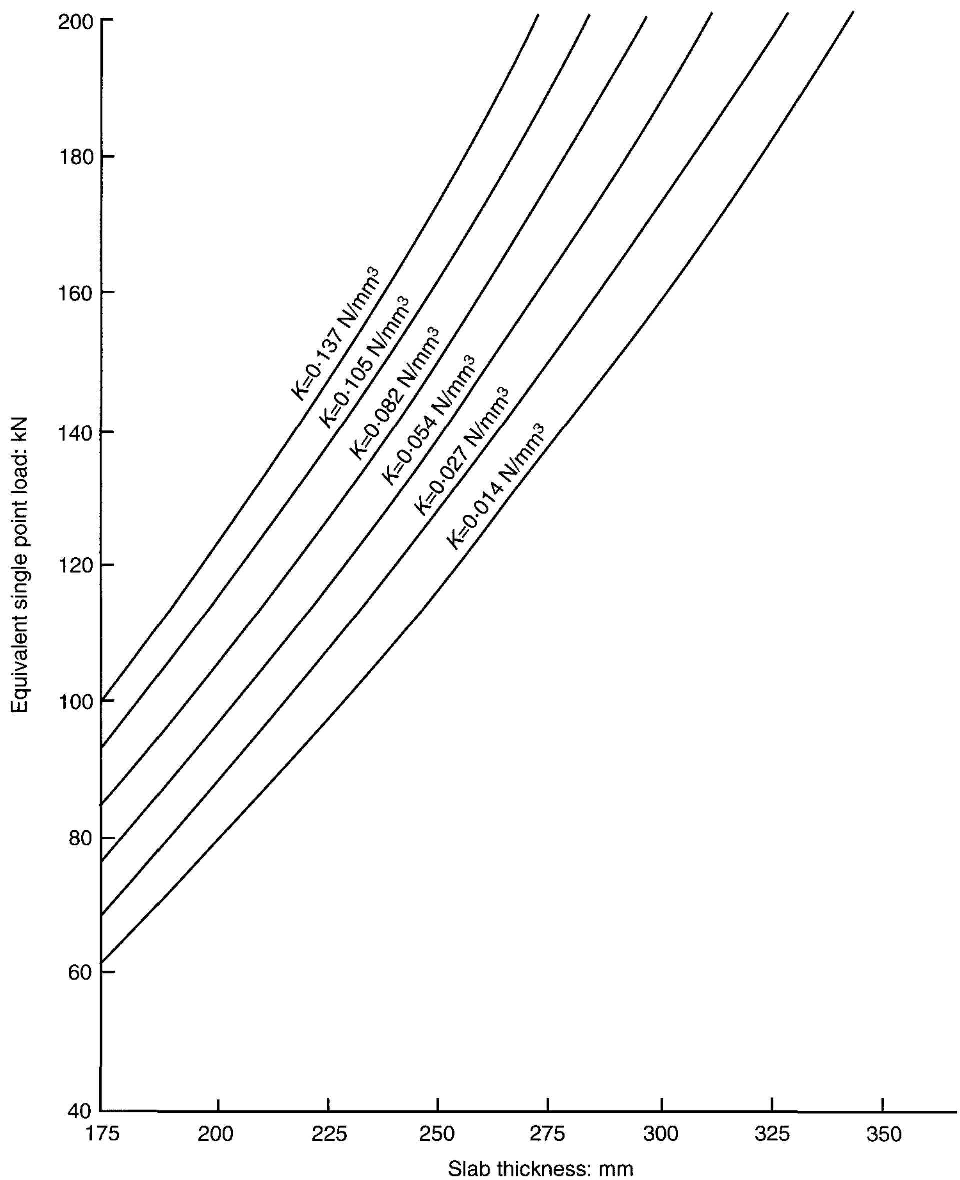
# Design Chart 8
# 20 kg/m$^3$ ZC 60/1·00 steel fibre
# C40 concrete

Flexural strength = 3·2 N/mm$^3$



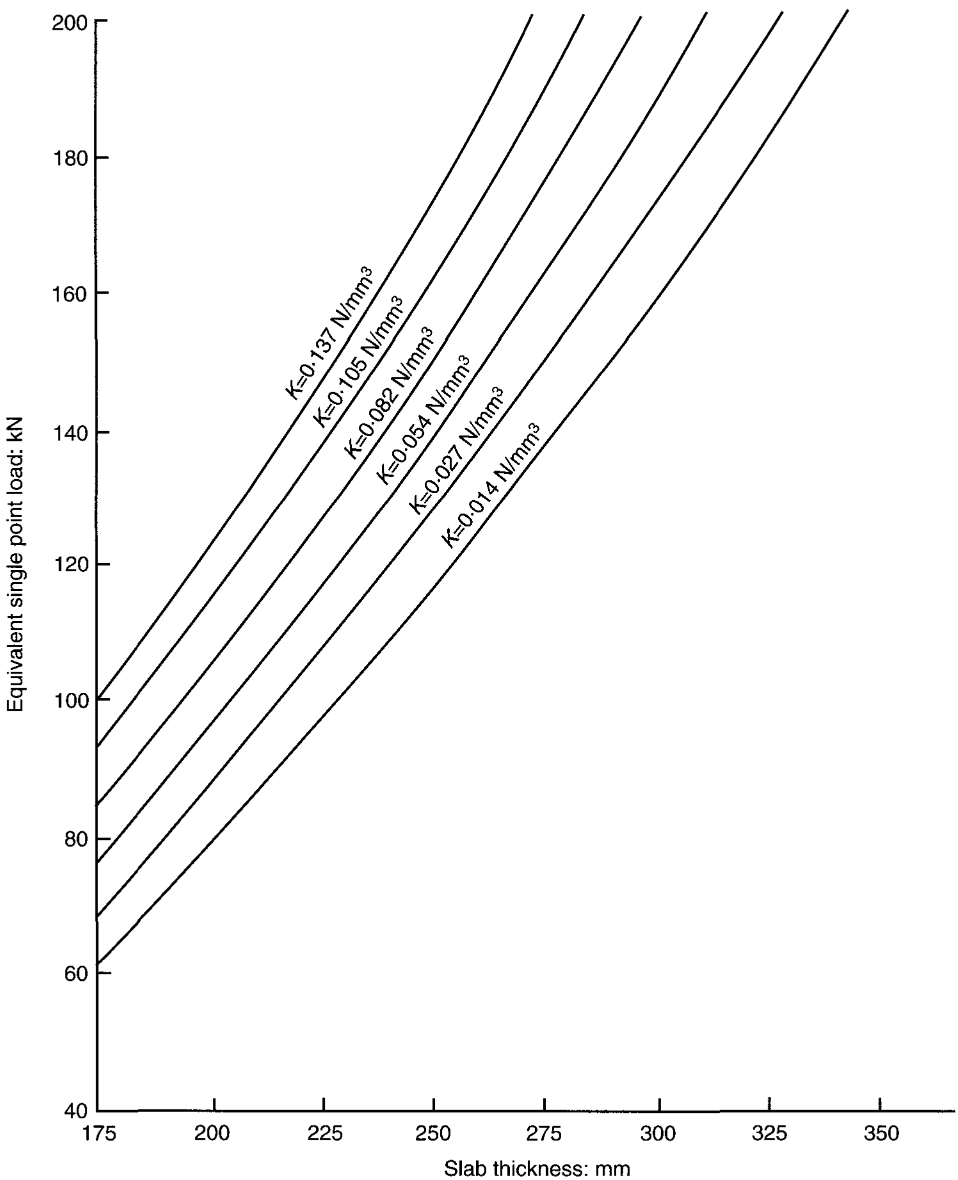
# Design Chart 9
# 30 kg/m$^3$ ZC 60/1·00 steel fibre
# C40 concrete

Flexural strength = 3·6 N/mm$^2$



# Design Chart 10
# 40 kg/m$^3$ ZC 60/1·00 steel fibre
# C40 concrete

Flexural strength = 4·2 N/mm$^2$



# 5 Case studies and related data

## 5.1 Joint movement monitoring exercise

Because concrete is susceptible to the time-dependent behaviour of shrinkage, creep and thermal movement, joints are required in concrete ground floor slabs. These factors result in changes in internal stress if the slab is restrained from moving. The result, if left unchecked, would be shrinkage, cracking and curling of the concrete slab. There are methods of predicting the shrinkage and creep of concrete[5.1] which are dependent on a number of variables.

### 5.1.1 Site investigation

A site investigation of joint performance has been undertaken of two ground supported concrete industrial floors. Each 1000 m$^2$ floor was constructed using polypropylene fibre reinforced concrete by laser screeding. The two floors were Unit 114/14 and Unit 114/10 of the Boldon Business Park, Tyne and Wear, UK. Once the floor slabs had been concreted and the saw cut joints made, the arrangement of studs shown in Figs 5.1 and 5.2 was established on the concrete surface to permit joint movements to be monitored.

### 5.1.2 Site observations

Using an extensometer with an accuracy of one hundredth of a millimetre, weekly measurements were taken during a period of nine months commencing December 1993.

### 5.1.3 Interpretation of results

Figures 5.3–5.12 show the movement of each of the measured joints from week to week. The larger movements at the beginning of the slabs' life represent joints cracking and the influence of cracked joints on neighbouring joints can be seen. When all the joints had cracked and were working, a more uniform movement became evident throughout the slab. The data were used to calculate the cumulative movement of each joint with time in both floors. There are some sudden larger movements during the early stages of each slab's life which corresponds with the active joint movement period. Once all the joints have cracked and are working, the movements become more uniform. The cumulative movements are of the same order, demonstrating uniform slab movement.

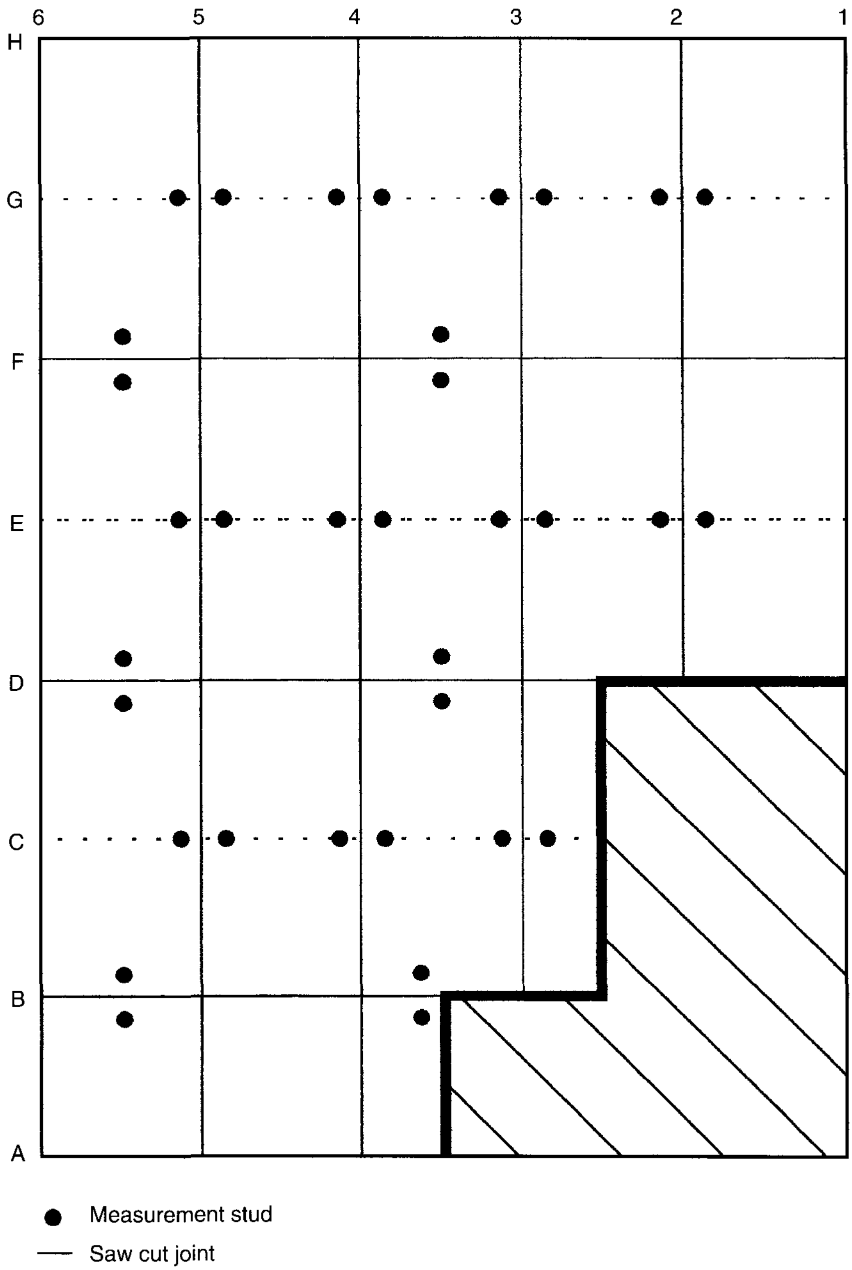



*Fig. 5.1. Plan of joints in Unit 114/14*

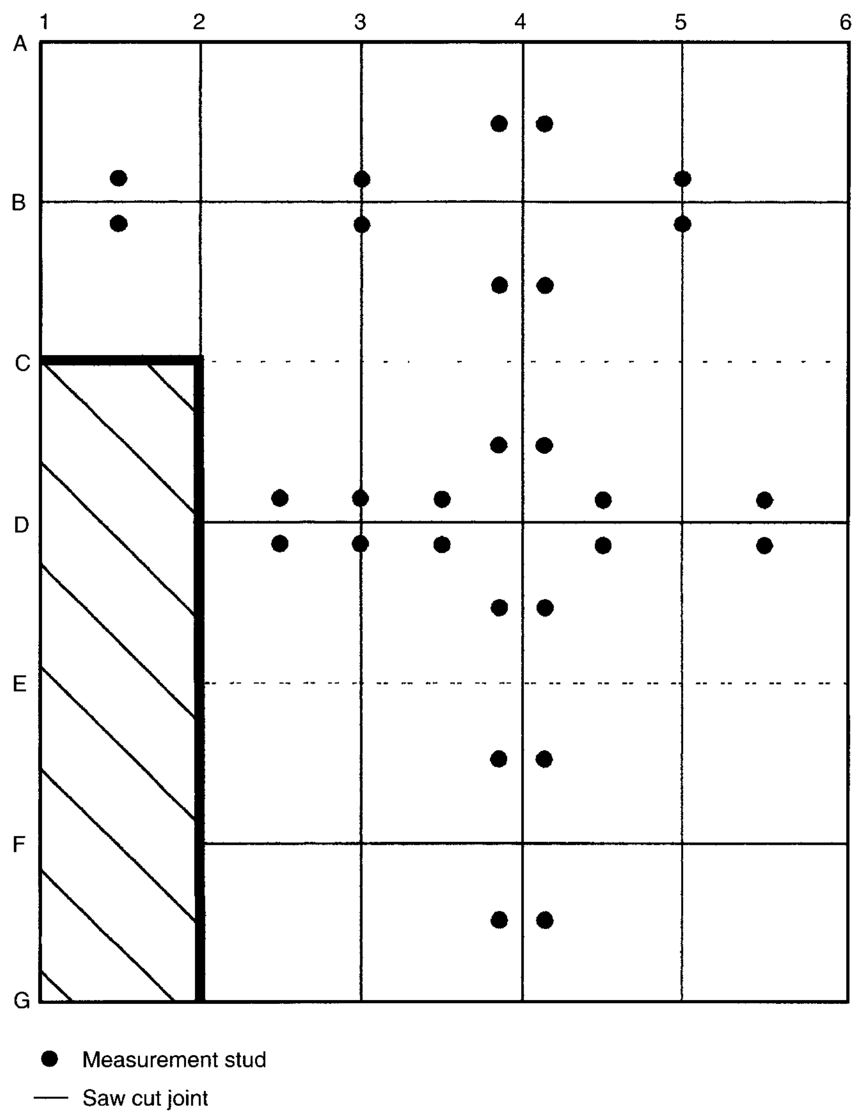



*Fig. 5.2. Plan of joints in Unit 114/10*

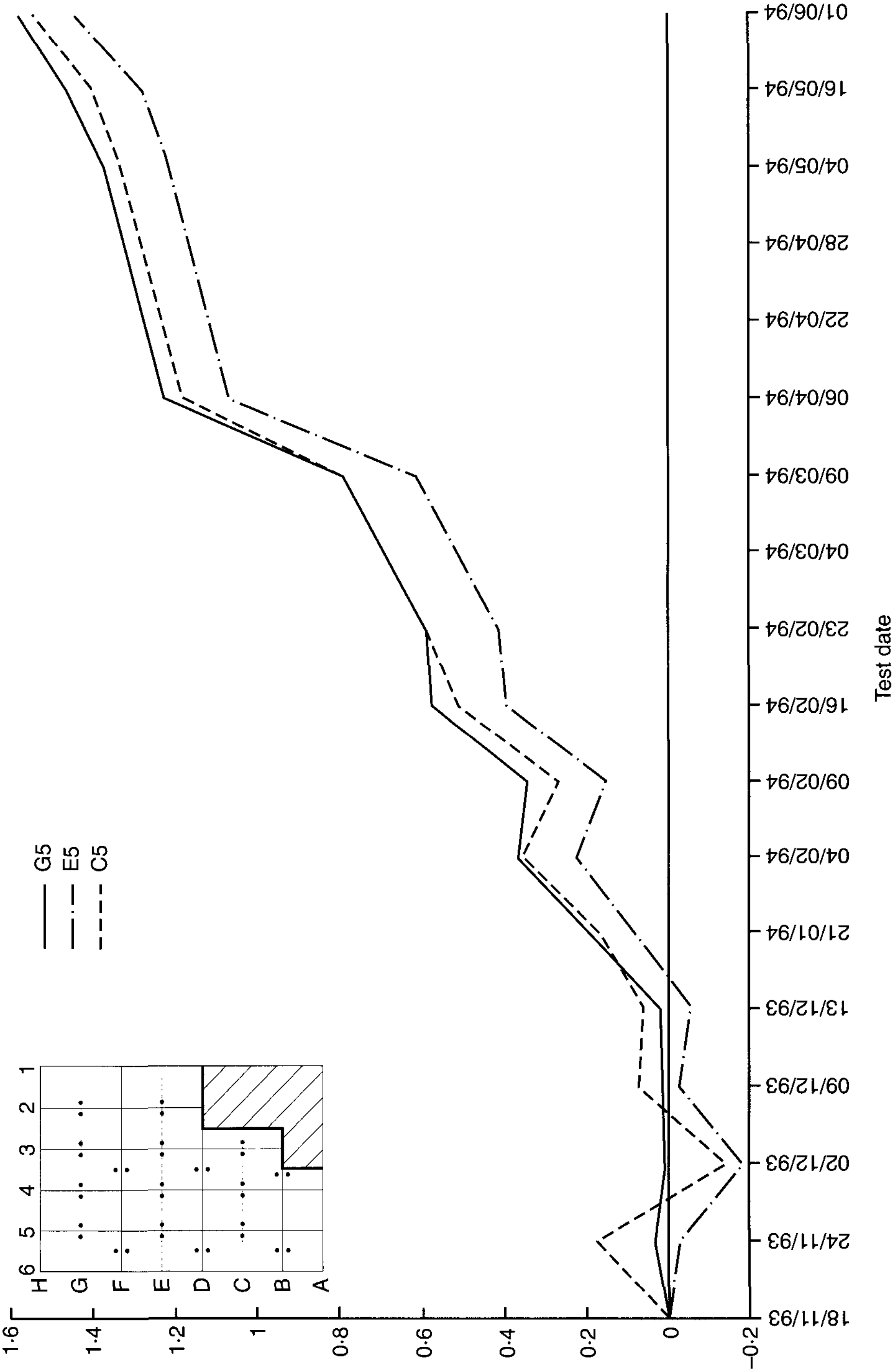



*Fig. 5.3. Cumulative movements of joint 5 in Unit 114/14*

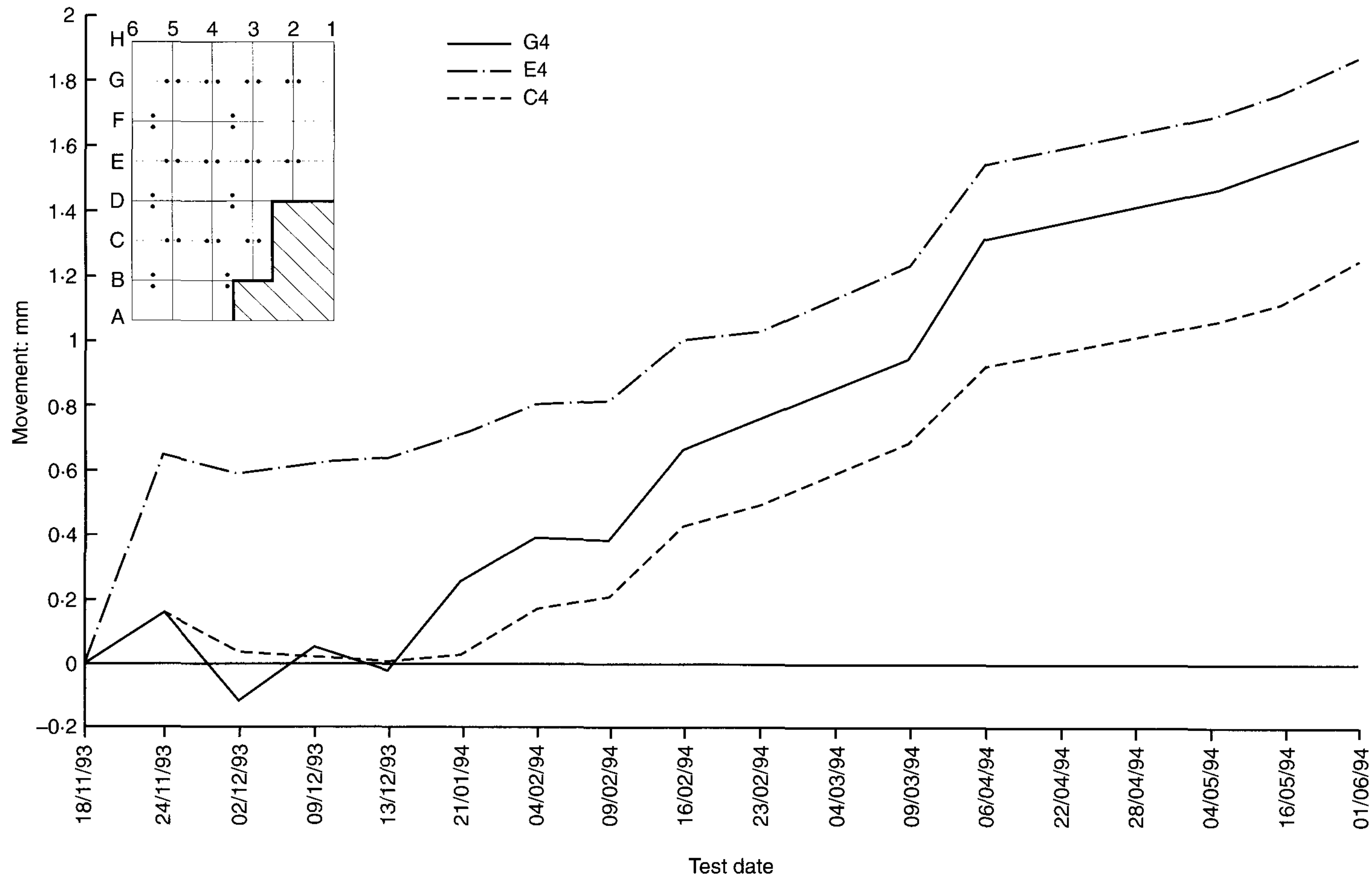



*Fig. 5.4. Cumulative movements of joint 4 in Unit 114/14*

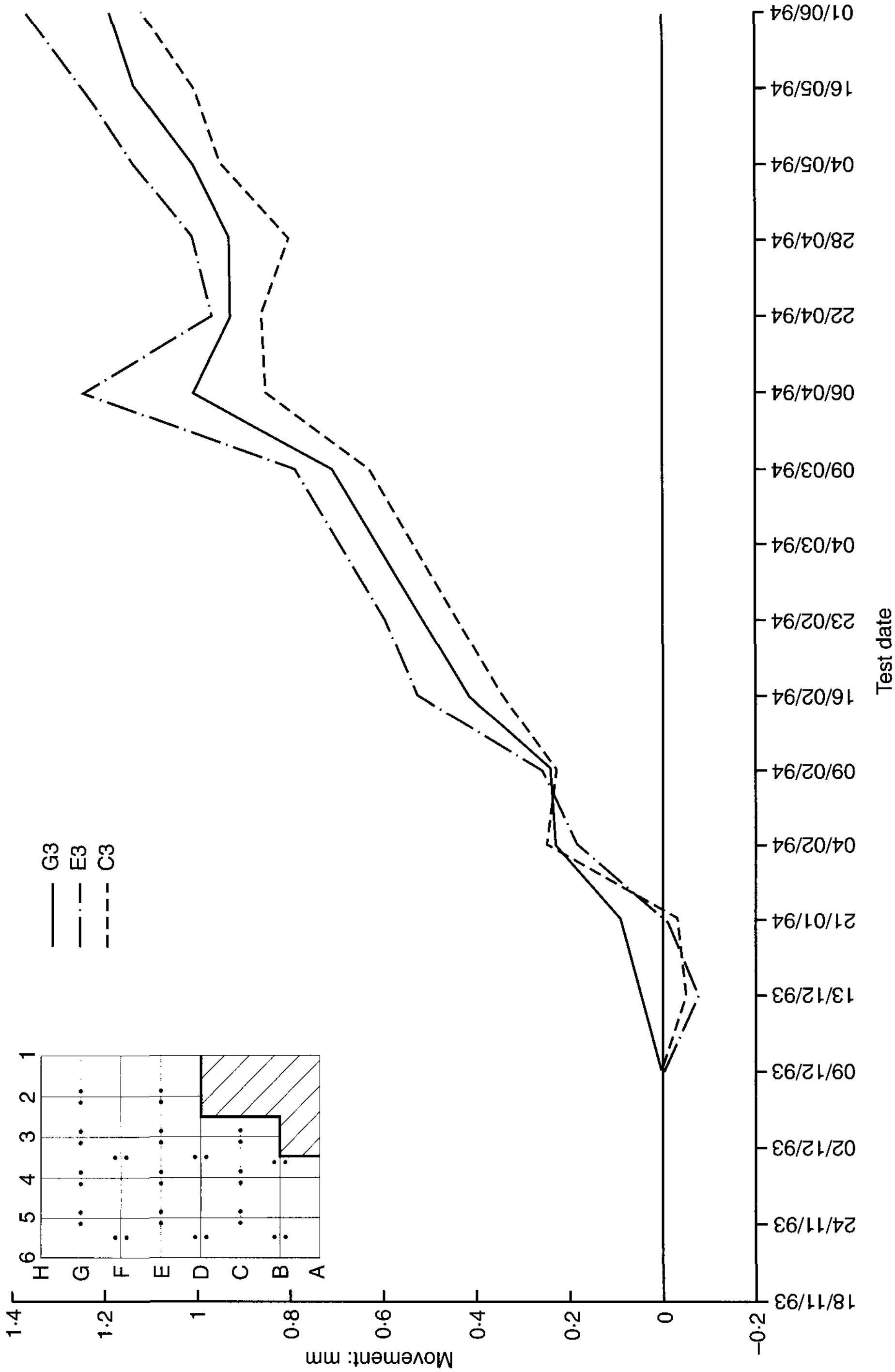



*Fig. 5.5. Cumulative movements of joint 3 in Unit 114/14*

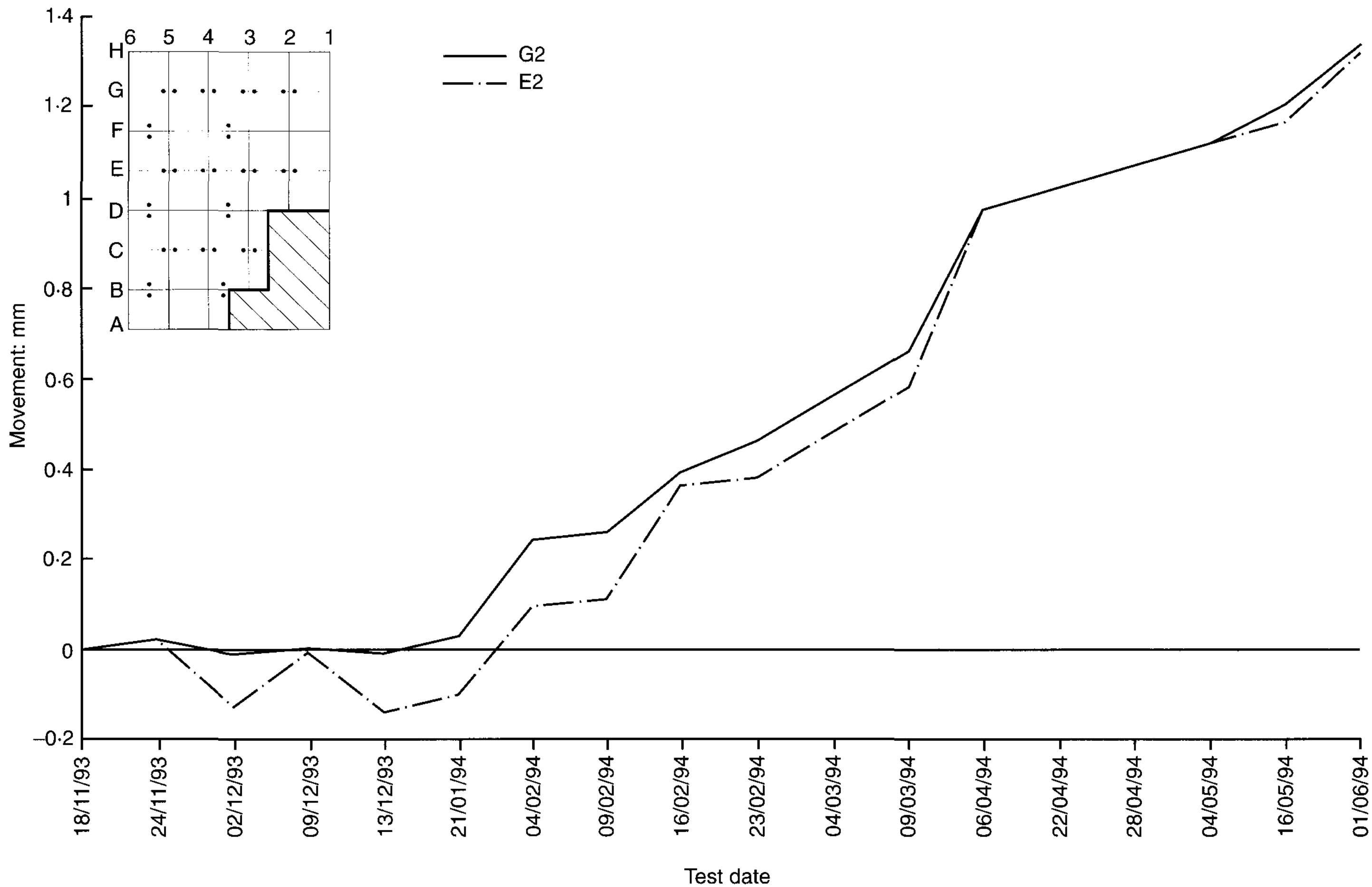



*Fig. 5.6. Cumulative movements of joint 2 in Unit 114/14*

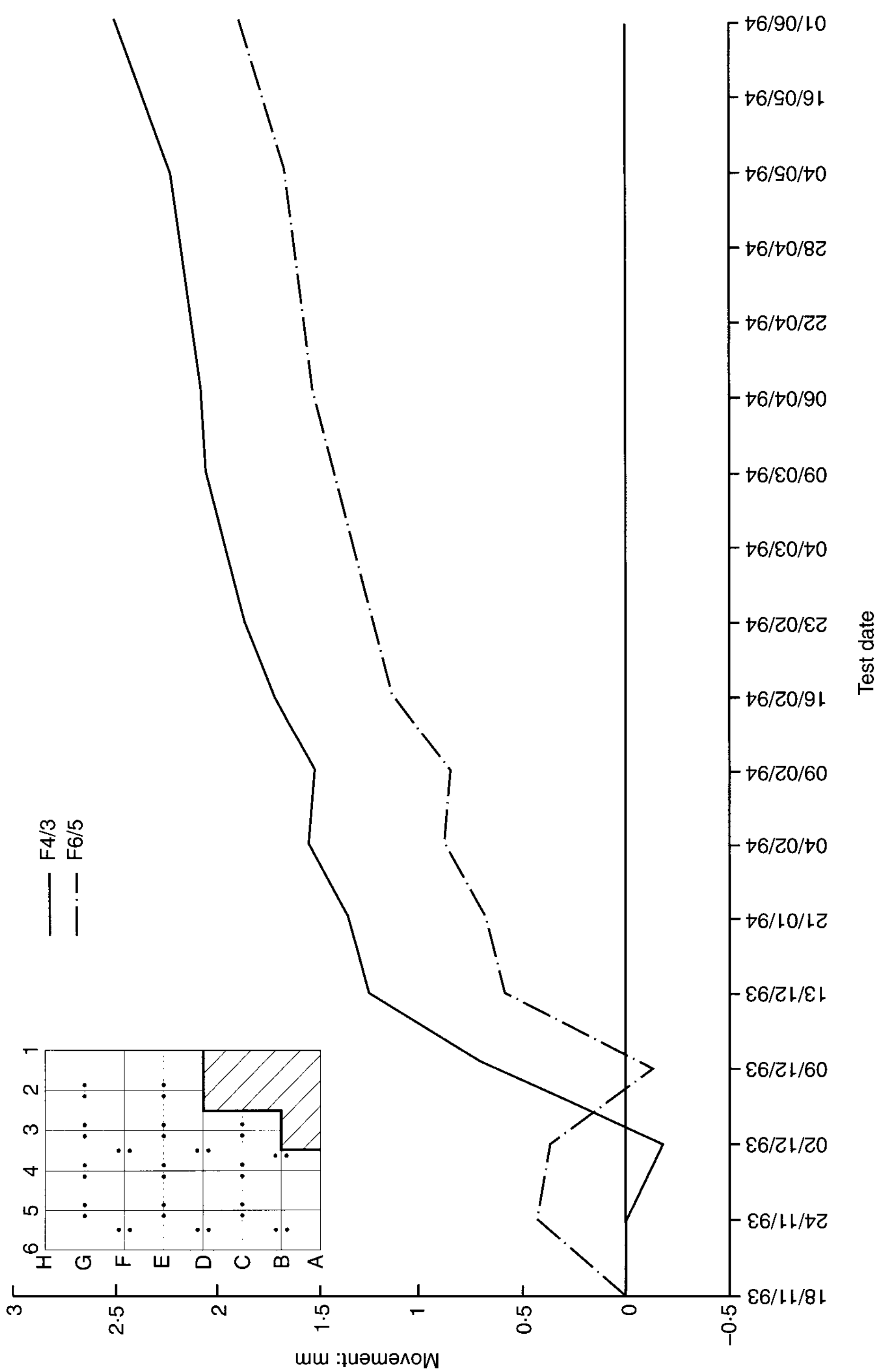



*Fig. 5.7. Cumulative movements of joint F in Unit 114/14*

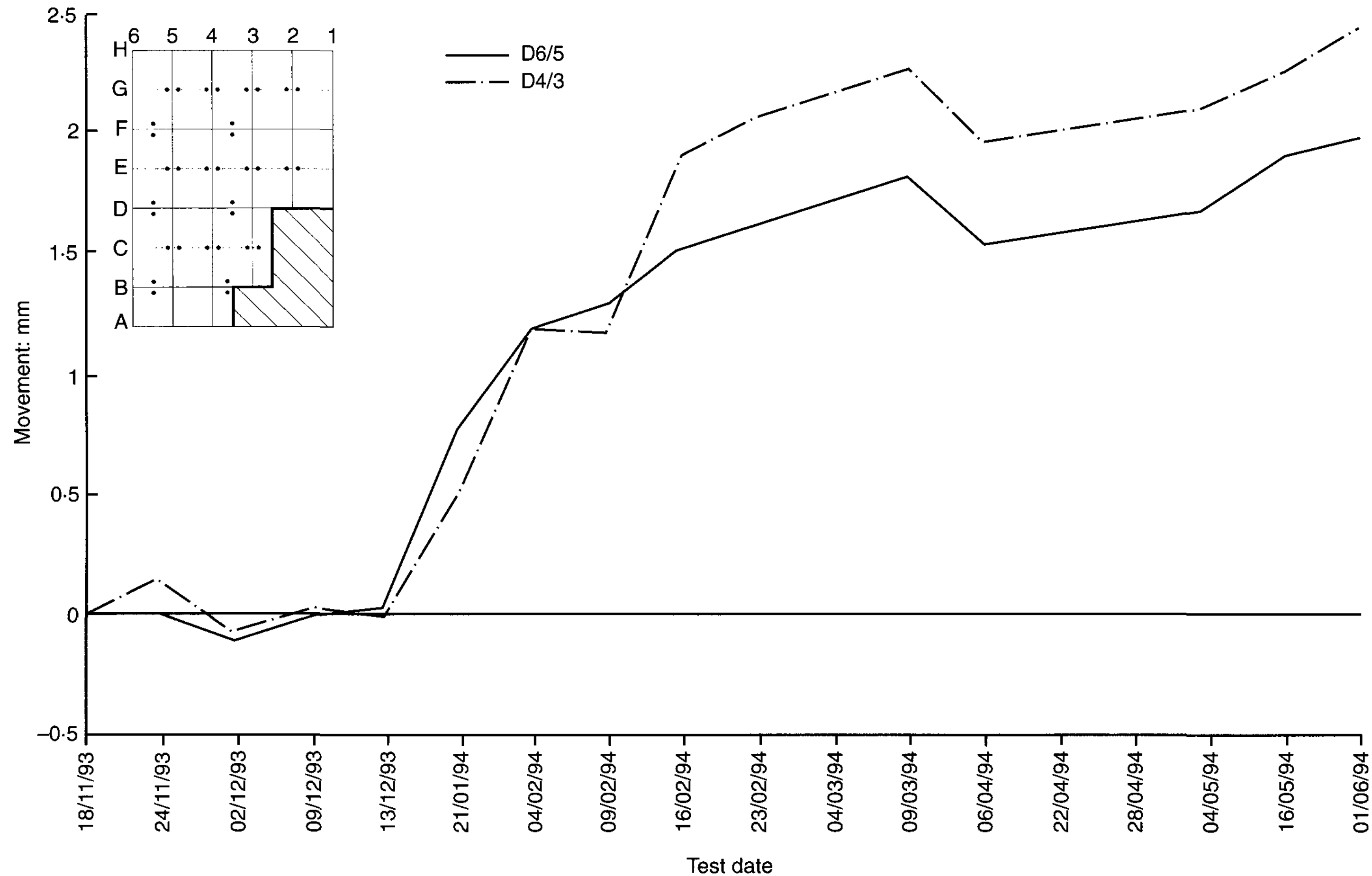



*Fig. 5.8. Cumulative movements of joint D in Unit 114/14*

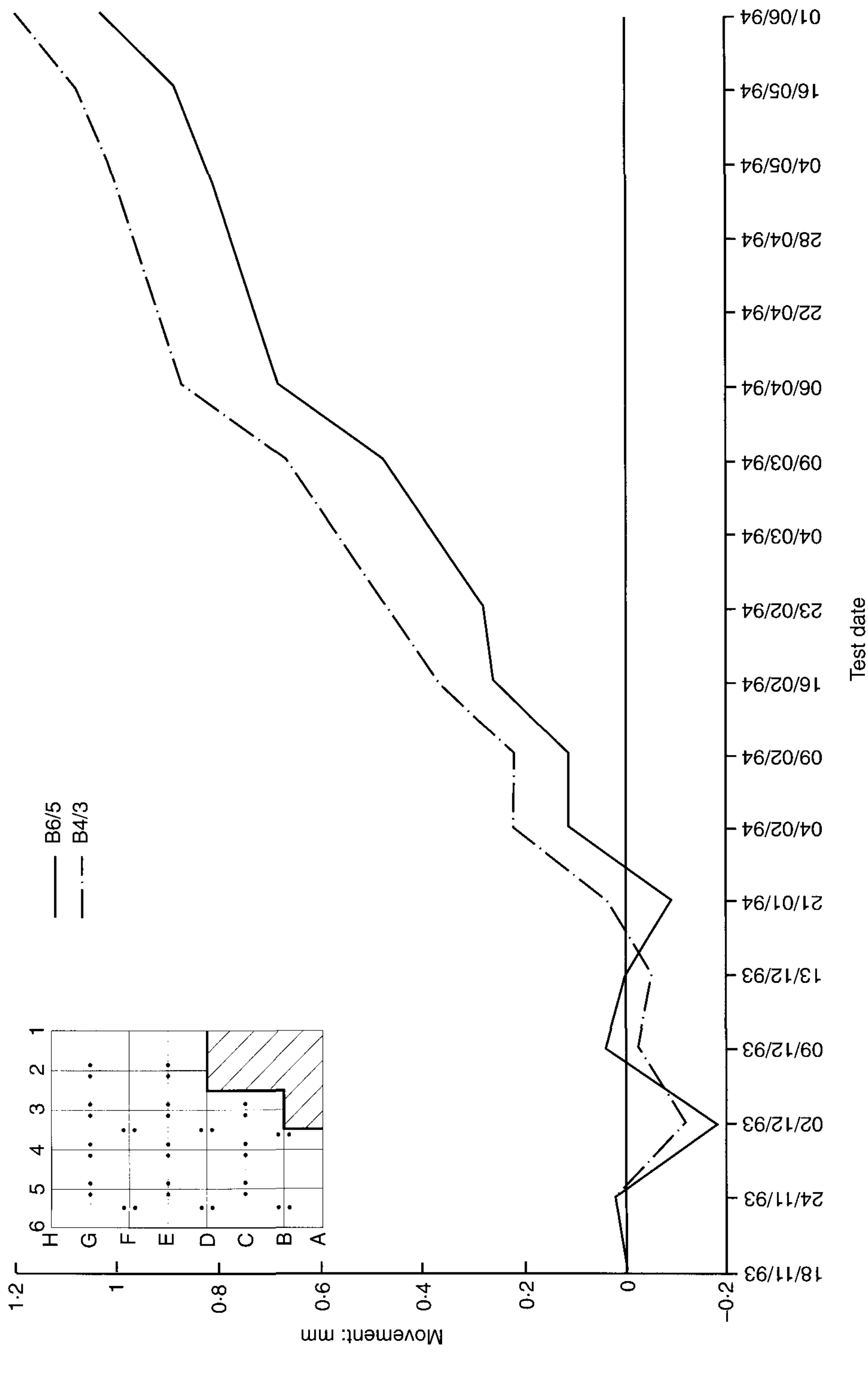



*Fig. 5.9. Cumulative movements of joint B in Unit 114/14*

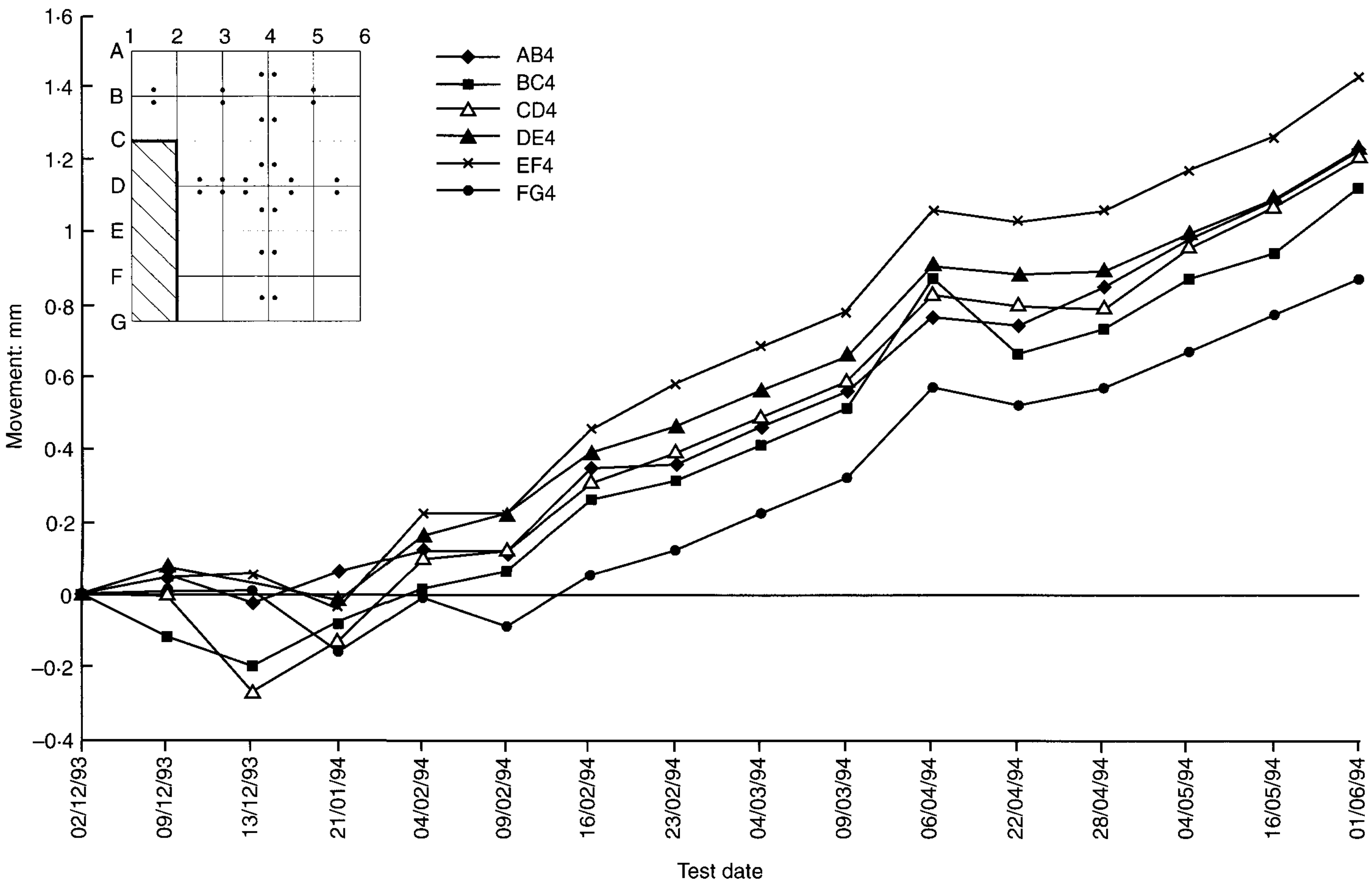



*Fig. 5.10. Cumulative movements of joint 4 in Unit 114/10*

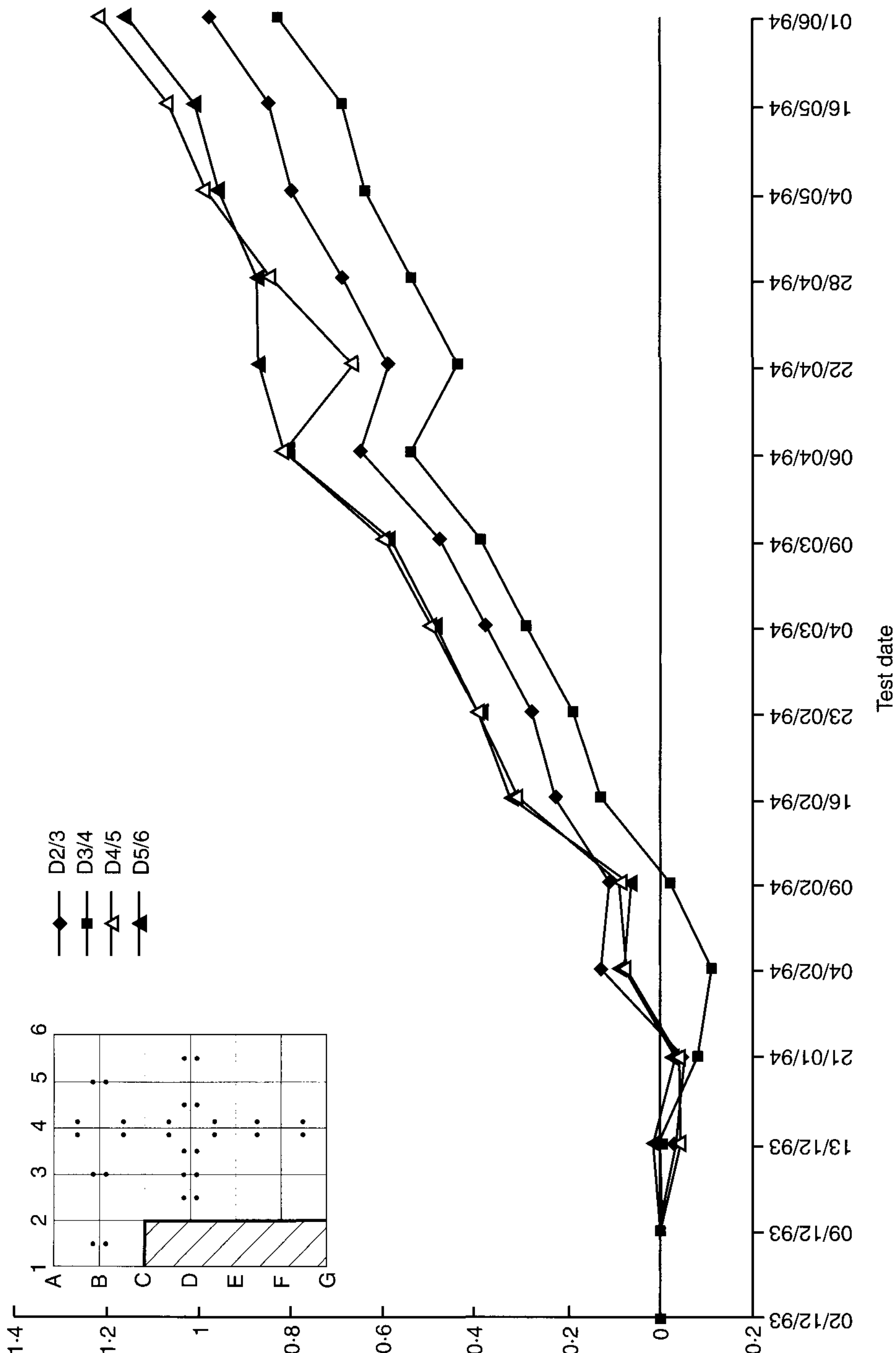



*Fig. 5.11. Cumulative movements of joint D in Unit 114/10*

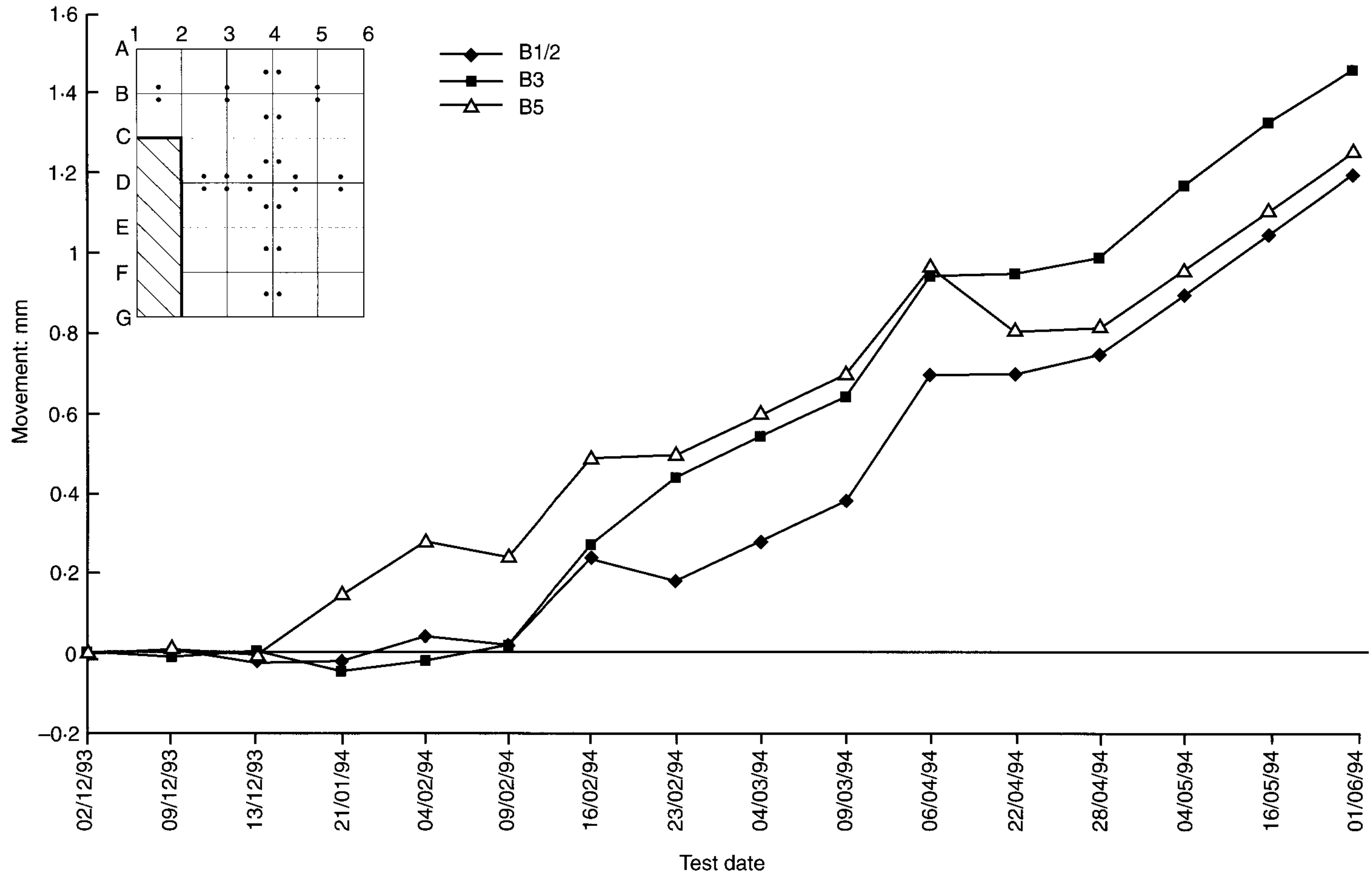



Fig. 5.12. *Cumulative movements of joint B in Unit 114/10*

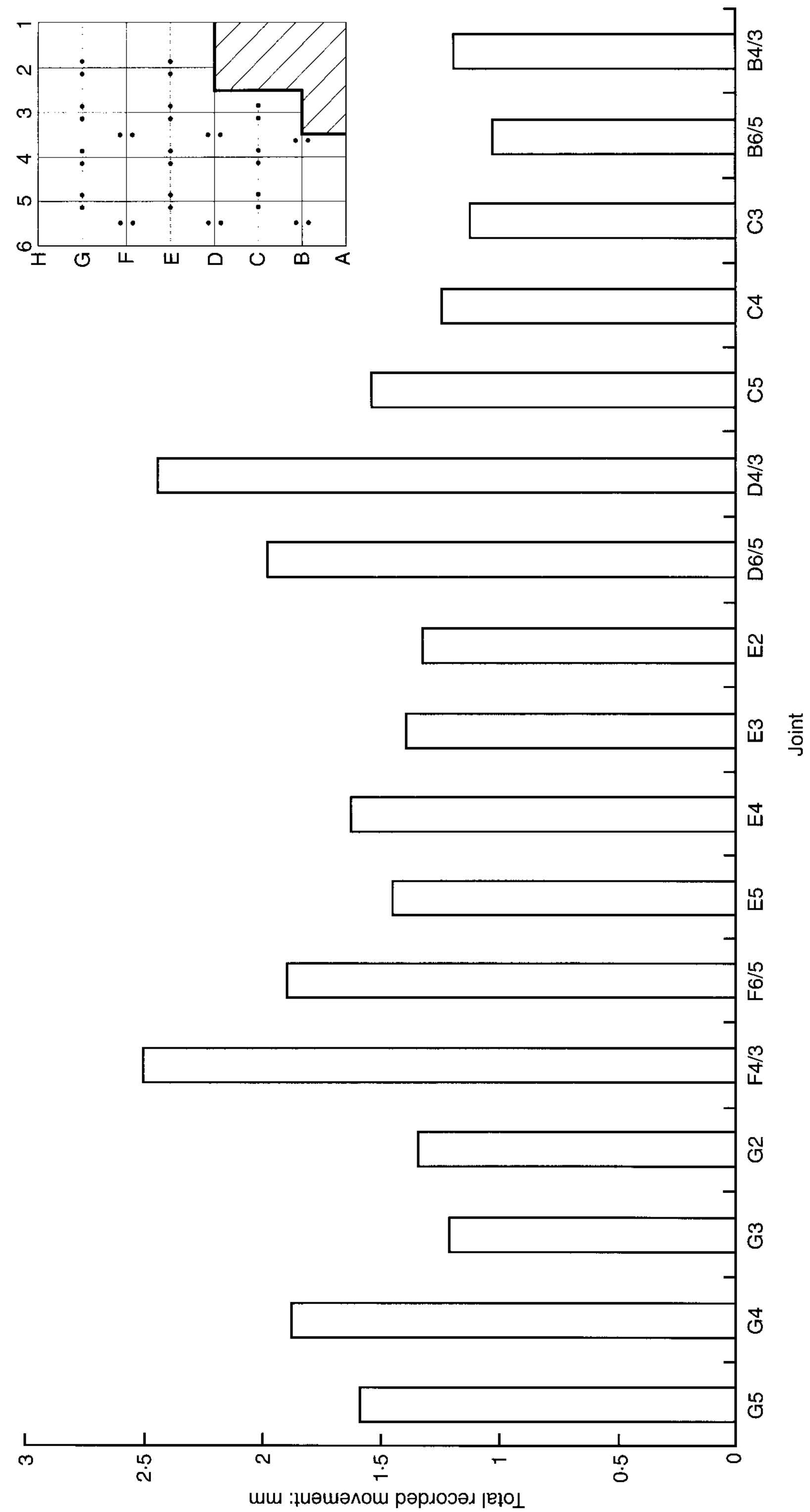



*Fig. 5.13. Total recorded movements in Unit 114/14*

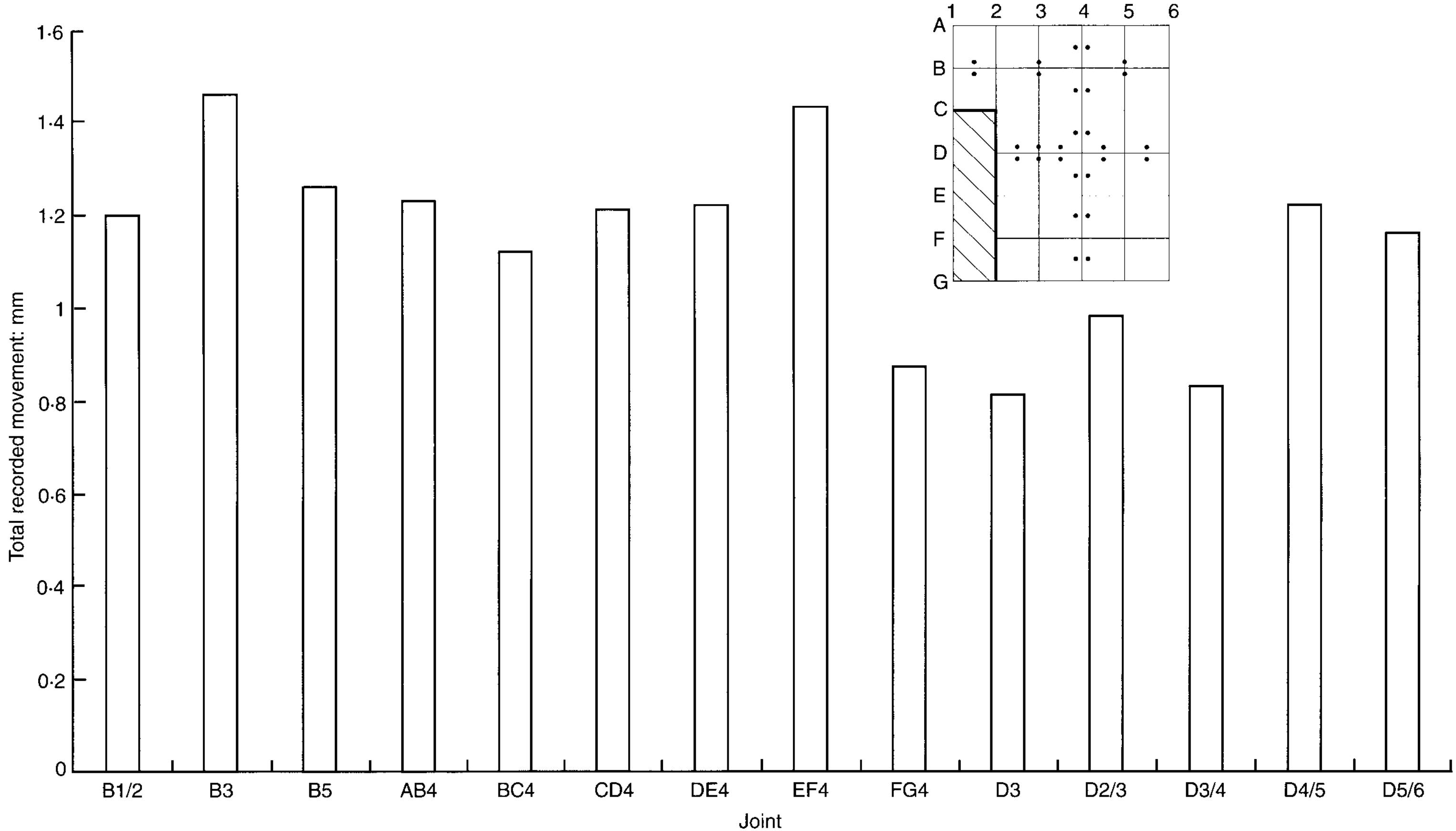



*Fig. 5.14. Total recorded movements in Unit 114/10*

Figures 5.13 and 5.14 show the total movement observed at each location for both slabs. All of the joints opened gradually and by the same degree. On average, each joint opened by between 1 mm and 2 mm, which is the optimum joint movement ensuring a working joint while at the same time maintaining aggregate interlock.

The monitoring of the two floors shows that the laser screeding process is capable of producing floors that behave in a manner which should ensure long-term successful performance. Monitoring of the two floors is continuing and ultimately, a full life data set will be available.

The joint movements recorded have occurred as a result of drying shrinkage (temperature related shrinkage is usually of secondary importance in concrete floors). The three effects of drying shrinkage are now described.

## 5.2 Structural design example

The following example illustrates the design process on a floor constructed using a laser guided screeding machine. Figure 2.16 showed a plan of an industrial unit for which a ground bearing floor design is required. The floor is to be installed by a laser guided screeding machine as one pour. The internal walls are to be constructed directly off the floor and the columns supporting the mezzanine floor are to fixed onto the floor. The floor is to be constructed over poor ground with a modulus of subgrade reaction $K$ of 0·027 N/mm$^3$. The floor area of 475 m$^2$ is well within the daily capacity of a laser screed, so it will be unnecessary to provide construction joints. The floor is to comprise a steel fibre reinforced concrete slab constructed over a granular sub-base. The joint spacings, slab thickness, slab strength, fibre dosage and sub-base thickness are determined as follows.

The first consideration is joint spacing. A 600 mm perimeter beam will be cast around the building and a full movement joint will be constructed separating the floor slab from this beam. See Section 5.9 for the construction of a perimeter beam. The only joints within the body of the floor slab will be induced joints formed by saw cutting 40 mm deep slots into the floor as soon as the concrete has gained sufficient strength to permit sawing without damaging the concrete. It is good practice for both aesthetic and structural reasons to align joints with column centres. In this case, joints could be

*Table 2.1. Common concretes and suggested joint spacings*

| Concrete type | Joint spacing: m |
|---|---|
| Plain C30 concrete | 6 |
| Micro-silica C30 concrete | 6 |
| 20 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C30 concrete | 6 |
| 30 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C30 concrete | 8 |
| 40 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C30 concrete | 10 |
| Plain C40 concrete | 6 |
| Micro-silica C40 concrete | 6 |
| 20 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C40 concrete | 6 |
| 20 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C40 concrete | 10 |
| 20 kg/m$^3$ ZC 60/1·00 steel fibre reinforcement C40 concrete | 12 |

provided on grid lines 2, 3, 4 and B or alternatively on 3 and B. By providing the additional joints on grid lines 2 and 4, the maximum joint spacing is reduced from 12 m to 9 m. Reducing joint spacing allows a greater range of flooring materials to be considered. A third alternative, and the one which will be developed henceforth is to provide joints on gridlines 2, 3, 4 and B and to provide additional joints half-way between A–B and B–C. This reduces the maximum distance between joints to 6 m and has the benefit of providing bays which are nearly square, which is the most efficient

*Table 4.2. Mean and characteristic 28 day flexural strength values for various concrete mixes — see Table 1.1 for other properties of concrete*

| | Flexural Strength: N/mm² | |
|---|---|---|
| | Mean | Characteristic |
| Plain C30 concrete | 2·0 | 1·8 |
| Micro-silica C30 concrete | 2·4 | 1·9 |
| C30 concrete 20 kg/m³ ZC 60/1·00 steel fibre[a] | 2·8 | 2·0 |
| C30 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 3·2 | 2·2 |
| C30 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 3·8 | 2·7 |
| Plain C35 concrete | 2·2 | 1·95 |
| Micro-silica C35 concrete | 2.6 | 2.0 |
| C35 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·0 | 2·1 |
| C35 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 3·4 | 2·3 |
| C35 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 4·0 | 2·9 |
| Plain C40 Concrete | 2·4 | 2·1 |
| Micro-silica C40 Concrete | 2·8 | 2·15 |
| C40 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·2 | 2·2 |
| C40 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 3·6 | 2·5 |
| C40 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 4·2 | 3·2 |
| Plain C45 Concrete | 2·7 | 2·3 |
| Micro-silica C45 Concrete | 3·1 | 2·4 |
| C45 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·5 | 2·5 |
| C45 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 4·0 | 2·8 |
| C45 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 4·8 | 3·6 |
| Plain C50 Concrete | 3·0 | 2·5 |
| Micro-silica C50 Concrete | 3·4 | 2·6 |
| C50 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 3·8 | 2·7 |
| C50 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 4·2 | 3·0 |
| C50 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 5·0 | 3·8 |
| Plain C55 Concrete | 3·4 | 2·7 |
| Micro-silica C55 Concrete | 3·8 | 2·8 |
| C55 concrete 20 kg/m³ ZC 60/1·00 steel fibre | 4·2 | 2·9 |
| C55 concrete 30 kg/m³ ZC 60/1·00 steel fibre | 4·6 | 3·2 |
| C55 concrete 40 kg/m³ ZC 60/1·00 steel fibre | 5·4 | 4·1 |

[a] ZC 60/1·00 refers to a commonly used anchored bright wire fibre of total length 60 mm and wire diameter 1·00 mm.

approach structurally. Table 2.1, reproduced here, shows that all of the commonly specified concretes are suitable.

A $20\,kg/m^3$ steel fibre C40 concrete will now be investigated. Table 4.2, reproduced here, shows that this fibre reinforced concrete has a characteristic flexural strength of $2{\cdot}2N/mm^2$.

In order to account for fatigue, the three categories of loading are to be increased by the following factors:

- mezzanine floor leg loads: $44 \times 1{\cdot}5 = 66\,kN$
- vehicle wheel load: $45 \times 2 = 90\,kN$
- material storage load: $60 \times 1{\cdot}25 = 75\,kN/m^2$.

The above three factors (1·5, 2 and 1·25) are based upon conventional fatigue relationships. Table 5.1 shows how increasing the load by various factors will extend the life of the floor. The system presented here allows different categories of loads to have different fatigue factors.

In the case of the mezzanine leg loads, the effect of load proximity needs to be taken into account (it is assumed that the vehicle tyres are sufficiently well separated for proximity to be ignored — this is not always the case). Figure 4.1 is used to determine the ratio $M_t/\boldsymbol{P}$. To do this, first, calculate the radius of relative stiffness, $l$, from:

$$l = \left(\frac{Eh^3}{12(1-\mu^2)K}\right)^{0{\cdot}25} \tag{5.1}$$

Usually, the elastic modulus of the fibre reinforced concrete, $E$, is taken as 20 000 $N/mm^2$ and Poisson's ratio, $\mu$, is 0·15. At this stage, it is necessary to assume a slab thickness with no real guidance. As a trial, consider 225 mm. Equation (5.1) can now be evaluated:

$$l = (20\,000 \times 225^3/12(1 - 0{\cdot}0225) \times 0{\cdot}027)^{1/4}$$

$$l = 921\,\text{mm}$$

*Table 5.1. Relationship between number of load repetitions and fatigue factor. By multiplying the actual load by the fatigue factor, a higher load is produced which can be used to ensure a floor of the specified life*

| Number of load repetitions | Fatigue factor |
|---|---|
| 1 | 1 |
| 50 | 1·25 |
| 5000 | 1·5 |
| 50,000 | 1·75 |
| Infinity | 2 |

The distance between the mezzanine leg loads is 1500 mm so the ratio *s/l* is 1500/921 = 1·63. From Figure 4.1, $M_t/\boldsymbol{P} = 0{\cdot}028$.

Equation (5.2) can now be used to determine the additional stress caused by one leg beneath its neighbour:

$$\sigma_{\text{add}} = \frac{M_t}{\boldsymbol{P}}\frac{6}{h^2}\boldsymbol{P}_2 \tag{5.2}$$

In this case, each leg load is 66 kN, so:

$$\sigma_{\text{add}} = 0{\cdot}028\frac{6}{225^2}66\,000 = 0{\cdot}22\,\text{N/mm}^2$$

This stress is in fact added twice because the leg loads are in line, which means one of the inner legs is subjected to the proximity effect twice.

Now use Equation (5.3):

$$\sigma_{\text{max}} = \sigma_{\text{flex}} - \sigma_{\text{add}} \tag{5.3}$$

to give:

$$\sigma_{\text{max}} = 2{\cdot}2 - 2 \times 0{\cdot}22 = 1{\cdot}76\,\text{N/mm}^2$$

Now use Equation (5.4) to determine the Single Point Load (SPL) which in this case accounts for the proximity of two neighbouring legs as well as the fatigue effect:

$$SPL = APL\left(\frac{\sigma_{\text{flex}}}{\sigma_{\text{max}}}\right) \tag{5.4}$$

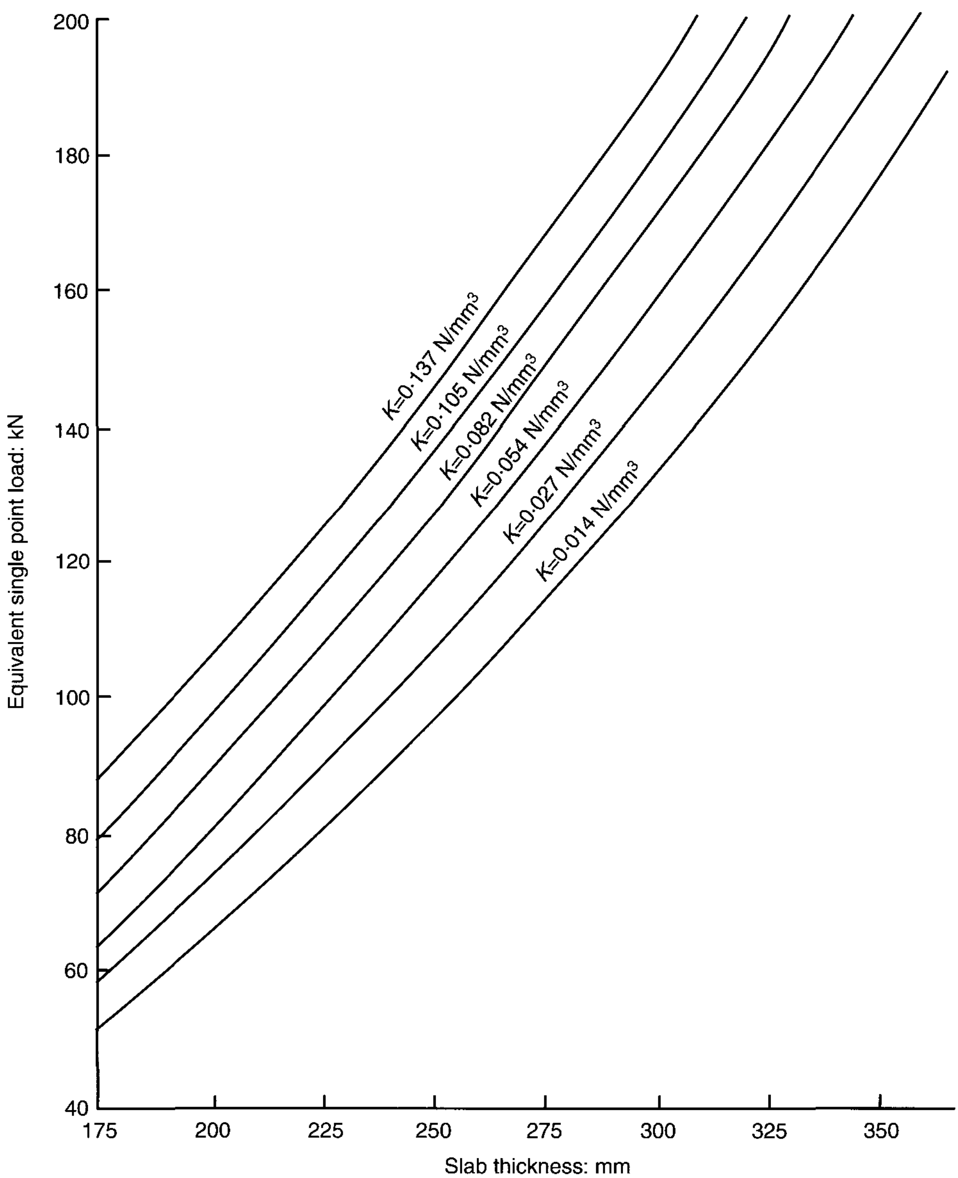
$$SPL = 66\left(\frac{2{\cdot}20}{1{\cdot}76}\right) = 82{\cdot}5\,\text{kN}$$

*Table 1.5 Enhanced value of K when a sub-base is used*

| *K* value of subgrade alone: N/mm$^3$ | Enhanced value of *K* when used in conjunction with: | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Granular sub-base of thickness: mm | | | | Cement-bound sub-base of thickness: mm | | | |
| | 150 | 200 | 250 | 300 | 100 | 150 | 200 | 250 |
| 0·014 | 0·018 | 0·022 | 0·027 | 0·033 | 0·045 | 0·063 | 0·081 | 0·106 |
| 0·027 | 0·034 | 0·038 | 0·044 | 0·051 | 0·075 | 0·104 | 0·137 | — |
| 0·054 | 0·059 | 0·065 | 0·072 | 0·081 | 0·125 | 0·175 | | |
| 0·082 | 0·089 | 0·096 | 0·105 | 0·114 | | | | |

# Design Chart 8
# 20 kg/m$^3$ ZC 60/1·00 steel fibre

Flexural strength = 3·2 N/mm$^2$



If the effective radius of contact of the leg loads falls outside the range 150–250 mm, Table 4.5 is now used to adjust the SPL to obtain the Equivalent Single Point Load (ESPL). In this case, it is assumed that the ESPL and the SPL are similar.

Having determined the ESPL, Design Chart 8 can now be used to determine the thickness of the floor. Before using the Design Chart, the effect of the granular sub-base in enhancing the modulus of subgrade reaction must be determined. Table 1.5, reproduced here, is used to do this.

In this case, the underlined value applies for a 250 mm thick sub-base and a subgrade $K$ value of $0{\cdot}027\,\text{N/mm}^3$. Using this value with the ESPL of 66 kN, Design Chart 8 confirms that the assumed slab thickness of 225 mm is satisfactory in the case of the mezzanine floor leg loads. Design Chart 8 is reproduced here with the 82·5 kN value brought across to the 0·044 $K$ value curve (0·044 is interpolated between the 0·027 and 0·054 curves).

Now consider the highway vehicle and calculate the stresses in the floor when subjected to a patch load of 90 kN.

The three Westergaard equations are reproduced here:

*(a) Patch load within slab:*

$$\boldsymbol{\sigma}_{\max} = \frac{0{\cdot}275(1+\mu)}{h^2}\mathbf{P}\log\left(\frac{0{\cdot}36Eh^3}{Kb^4}\right) \tag{5.5}$$

*(b) Patch load at edge of slab:*

$$\boldsymbol{\sigma}_{\max} = 0{\cdot}529(1+0{\cdot}54\mu)\frac{\mathbf{P}}{h^2}\log\left(\frac{0{\cdot}20Eh^3}{Kb^4}\right) \tag{5.6}$$

*(c) Patch load at slab corner:*

$$\boldsymbol{\sigma}_{\max} = \frac{3\mathbf{P}}{h^2}\left(1-\left(\frac{1{\cdot}41b}{\left(\dfrac{Eh^3}{12(1-\mu^2)K}\right)^{0{\cdot}25}}\right)^{0{\cdot}6}\right) \tag{5.7}$$

where $\boldsymbol{\sigma}_{\max}$ is the flexural stress (N/mm$^2$); $\boldsymbol{P}$ is the patch load (N), i.e. characteristic wheel load × load factor, 90 000 N; $\mu$ is Poisson's ratio, 0·15; $h$ is the slab thickness (mm), 225 mm; $E$ is the elastic modulus, 20 000 N/mm$^2$; $K$ is the modulus of subgrade reaction, 0·044 N/mm$^3$; $b$ is the radius of tyre contact zone (mm); $\mathbf{p}$ is the contact stress between wheel and floor, 0·8 N/mm$^2$; and $b$ is $(W/\pi\mathbf{p})^{1/2} = \sqrt{90\,000/0{\cdot}8\pi} = 190$ mm.

Tables 5.2–5.7 can be used as an aid to evaluating the three Westergaard equations. They apply to the common $\mu$ and $E$ values above. Westergaard stresses are obtained by multiplying together two values read from the appropriate tables.

## 5.2.1 Tables for patch load within slab

*Table 5.2. Values for* $[0{\cdot}275(1+\mu)/h^2]\boldsymbol{P}$. *The bold figure relates to this example*

| Load: N | Slab thickness: mm | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 | 275 | 300 | 325 | 350 |
| 10 000 | 0·140 | 0·103 | 0·079 | 0·062 | 0·051 | 0·042 | 0·035 | 0·030 | 0·026 |
| 20 000 | 0·280 | 0·206 | 0·158 | 0·124 | 0·102 | 0·084 | 0·070 | 0·060 | 0·052 |
| 30 000 | 0·420 | 0·309 | 0·237 | 0·186 | 0·153 | 0·126 | 0·105 | 0·090 | 0·078 |
| 40 000 | 0·562 | 0·413 | 0·316 | 0·250 | 0·202 | 0·167 | 0·140 | 0·120 | 0·103 |
| 50 000 | 0·703 | 0·516 | 0·395 | 0·312 | 0·253 | 0·209 | 0·176 | 0·150 | 0·129 |
| 60 000 | 0·840 | 0·618 | 0·474 | 0·375 | 0·304 | 0·251 | 0·211 | 0·180 | 0·155 |
| 70 000 | 0·984 | 0·723 | 0·553 | 0·437 | 0·354 | 0·293 | 0·246 | 0·210 | 0·181 |
| 80 000 | 1·124 | 0·826 | 0·632 | 0·500 | 0·404 | 0·334 | 0·280 | 0·240 | 0·206 |
| 90 000 | 1·265 | 0·929 | 0·712 | **0·562** | 0·455 | 0·376 | 0·316 | 0·269 | 0·232 |
| 100 000 | 1·406 | 1·032 | 0·790 | 0·624 | 0·506 | 0·418 | 0·352 | 0·300 | 0·258 |
| 150 000 | 2·109 | 1·548 | 1·185 | 0·936 | 0·759 | 0·627 | 0·487 | 0·450 | 0·387 |
| 200 000 | 2·812 | 2·064 | 1·580 | 1·248 | 1·012 | 0·836 | 0·704 | 0·600 | 0·516 |

*Table 5.3. Values for log* $(0{\cdot}36\ Eh^3/Kb^4)$. *The bold figure relates to this example*

| Load: N | Slab thickness: mm | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 | 275 | 300 | 325 | 350 |
| 10 000 | 4·544 | 4·746 | 4·920 | 5·073 | 5·210 | 5·334 | 5·447 | 5·552 | 5·648 |
| 20 000 | 3·944 | 4·145 | 4·319 | 4·472 | 4·610 | 4·734 | 4·847 | 4·952 | 5·048 |
| 30 000 | 3·592 | 3·793 | 3·967 | 4·121 | 4·258 | 4·382 | 4·495 | 4·600 | 4·696 |
| 40 000 | 3·341 | 3·541 | 3·715 | 3·869 | 4·006 | 4·130 | 4·244 | 4·348 | 4·444 |
| 50 000 | 3·145 | 3·346 | 3·520 | 3·673 | 3·811 | 3·935 | 4·048 | 4·153 | 4·249 |
| 60 000 | 2·992 | 3·193 | 3·366 | 3·520 | 3·657 | 3·782 | 3·895 | 4·000 | 4·096 |
| 70 000 | 2·831 | 3·031 | 3·205 | 3·359 | 3·496 | 3·620 | 3·733 | 3·838 | 3·934 |
| 80 000 | 2·740 | 2·941 | 3·115 | 3·269 | 3·406 | 3·530 | 3·643 | 3·748 | 3·844 |
| 90 000 | 2·627 | 2·825 | 3·002 | **3·155** | 3·292 | 3·417 | 3·530 | 3·634 | 3·731 |
| 100 000 | 2·538 | 2·738 | 2·913 | 3·714 | 3·203 | 3·327 | 3·441 | 3·545 | 3·642 |
| 150 000 | 2·193 | 2·393 | 2·567 | 2·721 | 2·858 | 2·982 | 3·096 | 3·199 | 3·296 |
| 200 000 | 1·941 | 2·142 | 2·316 | 2·469 | 2·607 | 2·731 | 2·844 | 2·948 | 3·045 |

The figures in this table apply in the case of poor subgrade ($K = 0{\cdot}027$ N/mm$^3$) with a 250 mm thick granular sub-base so the effective $K$ value is 0·044 N/mm$^3$. In the case of very poor subgrade on 150 mm granular sub-base material, increase the values in the table by 12%. For floors constructed over good ground with 150 mm thickness of sub-base, decrease the values by 6%.

## 5.2.2 Tables for patch load at edge of slab

*Table 5.4. Values for* $[0{\cdot}529(1+0{\cdot}54\mu)/h^2]\boldsymbol{P}$. *The bold figure relates to this example*

| Load: N | Slab thickness: mm) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 | 275 | 300 | 325 | 350 |
| 10 000 | 0·254 | 0·187 | 0·143 | 0·113 | 0·091 | 0·076 | 0·063 | 0·054 | 0·047 |
| 20 000 | 0·508 | 0·374 | 0·286 | 0·226 | 0·182 | 0·152 | 0·126 | 0·108 | 0·094 |
| 30 000 | 0·762 | 0·560 | 0·429 | 0·339 | 0·275 | 0·227 | 0·191 | 0·162 | 0·140 |
| 40 000 | 1·016 | 0·748 | 0·572 | 0·452 | 0·364 | 0·304 | 0·252 | 0·216 | 0·188 |
| 50 000 | 1·270 | 0·933 | 0·715 | 0·565 | 0·457 | 0·378 | 0·317 | 0·271 | 0·233 |
| 60 000 | 1·524 | 1·120 | 0·858 | 0·678 | 0·550 | 0·454 | 0·382 | 0·324 | 0·280 |
| 70 000 | 1·780 | 1·307 | 1·000 | 0·790 | 0·640 | 0·529 | 0·444 | 0·379 | 0·327 |
| 80 000 | 2·212 | 1·496 | 1·144 | 0·904 | 0·728 | 0·608 | 0·504 | 0·432 | 0·376 |
| 90 000 | 2·287 | 1·680 | 1·287 | **1·017** | 0·823 | 0·681 | 0·572 | 0·487 | 0·420 |
| 100 000 | 2·540 | 1·866 | 1·430 | 1·130 | 0·914 | 0·756 | 0·634 | 0·542 | 0·466 |
| 150 000 | 3·810 | 2·799 | 2·145 | 1·695 | 1·371 | 1·134 | 0·951 | 0·813 | 0·699 |
| 200 000 | 5·080 | 3·732 | 2·860 | 2·260 | 1·828 | 1·512 | 1·268 | 1·084 | 0·932 |

*Table 5.5. Values for log* $(0{\cdot}20\ Eh^3/Kb^4)$. *The bold figure relates to this example*

| Load: N | Slab thickness: mm | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 | 275 | 300 | 325 | 350 |
| 10 000 | 4·290 | 4·490 | 4·664 | 4·818 | 4·955 | 5·079 | 5·193 | 5·297 | 5·393 |
| 20 000 | 3·689 | 3·890 | 4·064 | 4·217 | 4·355 | 4·479 | 4·592 | 4·697 | 4·793 |
| 30 000 | 3·337 | 3·538 | 3·712 | 3·865 | 4·003 | 4·127 | 4·240 | 4·344 | 4·441 |
| 40 000 | 3·085 | 3·286 | 3·460 | 3·613 | 3·751 | 3·875 | 3·988 | 4·093 | 4·189 |
| 50,000 | 2·890 | 3·091 | 3·265 | 3·418 | 3·555 | 3·679 | 3·793 | 3·897 | 3·994 |
| 60 000 | 2·737 | 2·938 | 3·111 | 3·265 | 3·402 | 3·526 | 3·640 | 3·744 | 3·840 |
| 70 000 | 2·575 | 2·776 | 2·950 | 3·104 | 3·241 | 3·365 | 3·478 | 4·071 | 3·679 |
| 80 000 | 2·250 | 2·686 | 2·860 | 3·013 | 3·151 | 3·275 | 3·388 | 3·492 | 3·589 |
| 90 000 | 2·372 | 2·572 | 2·747 | **2·900** | 3·037 | 3·162 | 3·275 | 3·379 | 3·475 |
| 100 000 | 2·282 | 2·483 | 2·658 | 2·811 | 2·948 | 3·072 | 3·186 | 3·290 | 3·387 |
| 150 000 | 1·937 | 2·138 | 2·312 | 2·466 | 2·603 | 2·727 | 2·840 | 2·944 | 3·041 |
| 200 000 | 1·686 | 1·886 | 2·061 | 2·214 | 2·351 | 2·476 | 2·589 | 2·693 | 2·790 |

The figures in this table apply in the case of poor subgrade ($K = 0{\cdot}027\ \mathrm{N/mm^3}$) with a 250 mm thick granular sub-base so the effective $K$ value is $0{\cdot}044\ \mathrm{N/mm^3}$. In the case of very poor subgrade on 150 mm granular sub-base material, increase the values in the table by 12%. For floors constructed over good ground with 150 mm thickness of sub-base, decrease the values by 6%.

### 5.2.3 Tables for patch load at corner of slab

*Table 5.6. Values for* $1 - \{1{\cdot}41b/[Eh^3/12(1-\mu^2)K]^{0{\cdot}25}\}^{0{\cdot}6}$. *The bold figure relates to this example*

| Load: N | Slab thickness: mm) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 | 275 | 300 | 325 | 350 |
| 10 000 | 0·682 | 0·704 | 0·721 | 0·735 | 0·748 | 0·758 | 0·768 | 0·776 | 0·783 |
| 20 000 | 0·609 | 0·636 | 0·657 | 0·674 | 0·689 | 0·703 | 0·714 | 0·724 | 0·733 |
| 30 000 | 0·559 | 0·588 | 0·612 | 0·632 | 0·649 | 0·664 | 0·677 | 0·688 | 0·699 |
| 40 000 | 0·519 | 0·551 | 0·577 | 0·599 | 0·618 | 0·634 | 0·648 | 0·660 | 0·671 |
| 50 000 | 0·485 | 0·520 | 0·548 | 0·571 | 0·591 | 0·608 | 0·624 | 0·636 | 0·648 |
| 60 000 | 0·457 | 0·494 | 0·523 | 0·547 | 0·568 | 0·586 | 0·603 | 0·617 | 0·629 |
| 70 000 | 0·426 | 0·464 | 0·496 | 0·522 | 0·544 | 0·563 | 0·580 | 0·594 | 0·608 |
| 80 000 | 0·407 | 0·448 | 0·480 | 0·507 | 0·529 | 0·549 | 0·560 | 0·582 | 0·596 |
| 90 000 | 0·384 | 0·426 | 0·459 | **0·487** | 0·511 | 0·531 | 0·550 | 0·565 | 0·579 |
| 100 000 | 0·365 | 0·407 | 0·442 | 0·471 | 0·495 | 0·516 | 0·536 | 0·551 | 0·566 |
| 150 000 | 0·284 | 0·333 | 0·371 | 0·404 | 0·432 | 0·455 | 0·477 | 0·495 | 0·511 |
| 200 000 | 0·219 | 0·272 | 0·314 | 0·350 | 0·380 | 0·406 | 0·429 | 0·449 | 0·467 |

*Table 5.7. Values for* $3\mathbf{P}/h^2$. *The bold figure relates to this example*

| Load: N | Slab thickness: mm | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 | 275 | 300 | 325 | 350 |
| 10 000 | 1·333 | 0·980 | 0·750 | 0·593 | 0·480 | 0·397 | 0·333 | 0·284 | 0·245 |
| 20 000 | 2·667 | 1·959 | 1·500 | 1·185 | 0·960 | 0·793 | 0·667 | 0·568 | 0·490 |
| 30 000 | 4·000 | 2·938 | 2·250 | 1·778 | 1·440 | 1·190 | 1·000 | 0·850 | 0·735 |
| 40 000 | 5·333 | 3·918 | 3·000 | 2·370 | 1·920 | 1·587 | 1·333 | 1·136 | 0·980 |
| 50 000 | 6·667 | 4·898 | 3·750 | 2·963 | 2·400 | 1·983 | 1·667 | 1·420 | 1·224 |
| 60 000 | 8·000 | 5·877 | 4·500 | 3·555 | 2·880 | 2·380 | 2·000 | 1·704 | 1·469 |
| 70 000 | 9·333 | 6·857 | 5·250 | 4·148 | 3·360 | 2·777 | 2·333 | 1·988 | 1·714 |
| 80 000 | 10·667 | 7·836 | 6·000 | 4·740 | 3·840 | 3·174 | 2·667 | 2·272 | 1·959 |
| 90 000 | 12·000 | 8·816 | 6·750 | **5·333** | 4·320 | 3·570 | 3·000 | 2·556 | 2·204 |
| 100 000 | 13·333 | 9·796 | 7·500 | 5·926 | 4·800 | 3·967 | 3·333 | 2·840 | 2·449 |
| 150 000 | 20·000 | 14·694 | 11·250 | 8·889 | 7·200 | 5·950 | 5·000 | 4·260 | 3·673 |
| 200 000 | 26·667 | 19·592 | 15·000 | 11·852 | 9·600 | 7·934 | 6·667 | 5·680 | 4·898 |

### 5.2.4 Solution

The stresses developed in the slab by a patch load of 90 kN can now be calculated using the values in bold in Tables 5.2–5.7 as follows.

(*a*) Maximum stress within the slab: $0{\cdot}562 \times 3{\cdot}155 = 1{\cdot}77\,\text{N/mm}^2$
(*b*) Maximum stress at slab edge: $1{\cdot}017 \times 2{\cdot}900 = 2{\cdot}95\,\text{N/mm}^2$
(*c*) Maximum stress at corner of slab: $0{\cdot}487 \times 5{\cdot}333 = 2{\cdot}60\,\text{N/mm}^2$

In the case of edge and corner stresses, aggregate interlock and fibre load transfer may allow the sharing of patch loads by neighbouring slabs. This transfer reduces edge and corner stresses by an amount that depends upon the amount by which the joints open. The true joint opening depends upon many factors, some of which will be unknown at design stage. Experience indicates that significant load transfer occurs when joints open by up to 1 mm. When joint openings exceed 2 mm, there is little or no load transfer. In the absence of other data, Table 5.8 may be used to assess load transfer. The table has been developed from the author's experience in the design and maintenance of floors.

In this example, joints are at 6 m spacings in both directions, so the maximum 30% load transfer can be assumed. This applies to both edge and corner loading. Therefore, the three critical stresses are as follows:

(*a*) Maximum stress within the slab $= 1{\cdot}77\,\text{N/mm}^2$
(*b*) Maximum stress at slab edge $= 2{\cdot}07\,\text{N/mm}^2$
(*c*) Maximum stress at corner of slab $= 1{\cdot}82\,\text{N/mm}^2$

The characteristic strength of the concrete is $2{\cdot}2\,\text{N/mm}^2$ so the proposed slab is satisfactory in the case of patch loading.

Now consider the materials storage load of $75\,\text{kN/m}^2$ or $0{\cdot}075\,\text{N/mm}^2$. The stress produced by the most onerous combination of storage space and aisles is given by:

$$\sigma_{\max} = 1008q/(\lambda^2 h^2)(\text{N/mm}^2)$$

Table 4.3 which shows values of $\lambda^2 h^2$ for common combinations of modulus of subgrade reaction ($K$) and slab thickness, is reproduced here with an additional line showing values for $K = 0{\cdot}44\,\text{N/mm}^3$ which applies to this example. The appropriate value of $\lambda^2 h^2$ is shown in bold.

Substituting known values into the stress equation (Equation (5.8)) yields:

$$\sigma_{\max} = 1008 \times 0{\cdot}075/0{\cdot}053 = 1{\cdot}426\,\text{N/mm}^2$$

*Table 5.8. Dowel bar load transfer efficiency values*

| Joint spacing: m | Load transfer: % |
|---|---|
| 6 or less | 30 |
| 8 | 20 |
| 10 | 10 |
| 12 | 0 |

*Table 4.3. Values of $\lambda^2 h^2$ for combinations of slab thickness and modulus of subgrade reaction*

| Modulus of subgrade reaction $K$: N/mm$^3$ | $\lambda^2 h^2$ for slab thickness (mm): | | | | |
|---|---|---|---|---|---|
| | 150 | 175 | 200 | 225 | 250 |
| 0·082 | 0·061 | 0·066 | 0·070 | 0·074 | 0·078 |
| 0·054 | 0·049 | 0·053 | 0·057 | 0·060 | 0·063 |
| 0·044 | 0·043 | 0·047 | 0·050 | **0·053** | 0·056 |
| 0·027 | 0·035 | 0·038 | 0·040 | 0·043 | 0·045 |
| 0·013 | 0·024 | 0·026 | 0·028 | 0·029 | 0·031 |

This is less than the characteristic strength of 2·2 N/mm$^2$ so the floor is satisfactory for the distributed load.

See Section 2.9 for an outline of the construction procedure for this example.

## 5.3 Single pour floor construction case study

Figures 5.15–5.22 illustrate the installation of a single pour distribution warehouse floor using a laser guided screeding machine to place, compact and finish the surface of the concrete. The floor was constructed during 2001 in Dublin. Errors in the thickness of the

*Fig. 5.15. Laser guided screeding machine spreading, compacting and finishing a large pour concrete slab. As the machine places the concrete, stabilising legs are extended so that the wheels no longer touch the ground. Here, the carriage is fully extended. Note the absence of reinforcement mesh. Steel fibres have been added to the concrete*


*Fig. 5.16. Details of a laser guided screeding machine's screed carriage. A rotating vibrating tube places, compacts and finishes the concrete as the carriage is moved progressively towards the machine operator*

*Fig. 5.17. Power trowelling machines ensure a durable abrasion resistant finish. The operator rides on larger machines (see Fig. 5.19) but smaller machines are operated as shown in this figure*

*Fig. 5.18. The management of the delivery of concrete is a crucial element in single pour floors. It is common for over 500 m$^3$ of concrete to be placed in one day*

*Fig. 5.19. Ride-on power trowelling machines allow greater areas to be finished and are commonly used on floors of area exceeding 500 m$^2$*

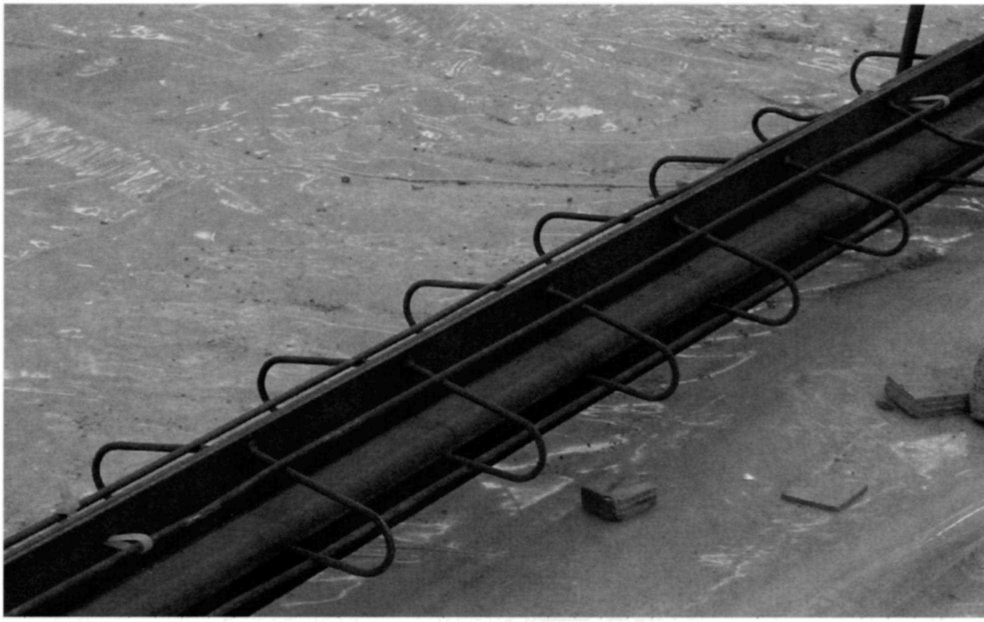


*Fig. 5.20. Floors larger than 2000 m$^2$ may require a formed daywork joint. Specialist flooring contractors have developed steel fabrications that strengthen such joints, eliminating spalling and providing load transfer. This 'Delta' joint being installed in a large distribution warehouse floor in Dublin has proven successful*

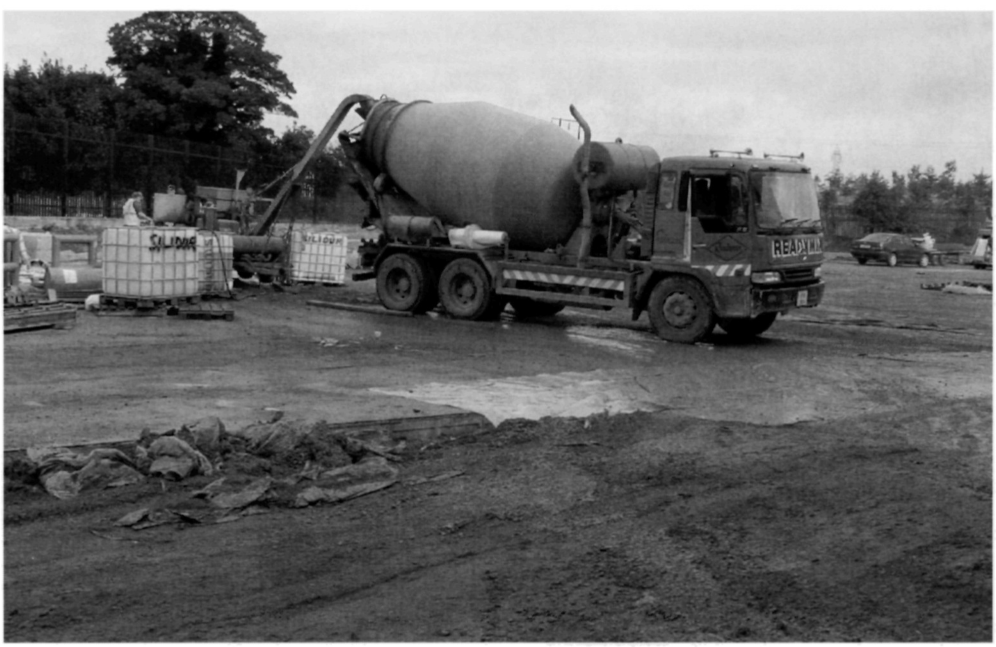


*Fig. 5.21. Steel fibres are frequently included in single pour floors to reduce or eliminate intermediate sawn joints. The steel fibres can be added to the concrete at the production plant or, as here, at the construction site*

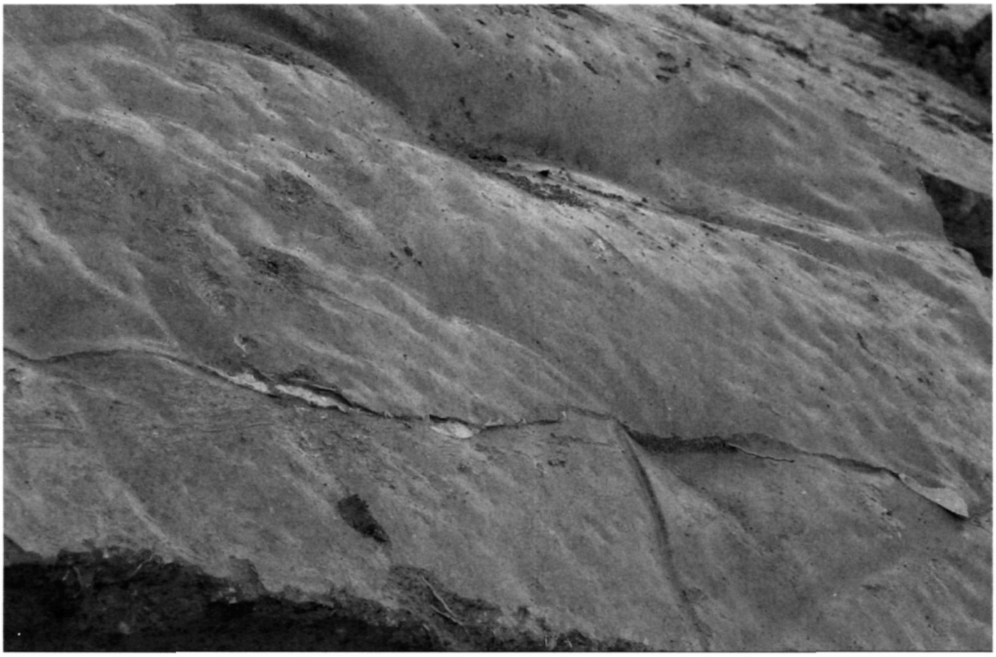


*Fig. 5.22. A recently constructed slab has been removed to reveal its underside. A slip membrane had been provided. The figure illustrates that the underside has a significant degree of undulation. This is common and is a result of a concrete delivery vehicles disturbing the sub-base material*

concrete led to the removal of the floor and its reconstruction to the correct thickness. Figure 5.22, showing the underside of the concrete, was taken following the removal of the original floor. By including steel fibres, the designer chose to eliminate intermediate joints and to specify only formed construction joints. This is beneficial in a distribution warehouse where flatness is important.

## 5.4 Long strip floor construction case study

This floor was constructed during 1998 at the Silverlink Industrial Estate in North Tyneside. A conventionally reinforced 150 mm thick concrete slab was constructed over a dolomitic limestone 150 mm thick sub-base. The floor was constructed over a period of two weeks. One or two strips running from one side of the building to the other were constructed each working day. The reinforcement was placed near the upper surface of the slab and three types of joint were provided. The joints between neighbouring strips comprised conventional ties connecting the strips. The transverse joints comprised dowelled joints and tied joints. In the case of the dowelled joints, sleaves were placed over the bars at one side of the joint to allow contraction. The ties comprised a short length of mesh reinforcement placed at the underside of the slab.

This case study is similar in some respects to the one shown in Section 2.9, except no perimeter beam is included and the surrounds to the columns are square. Details of the project are included in the captions to Figs 5.23–5.41.

*Fig. 5.23. The propped portal frame has been completed and the sub-base has been laid. Floor construction cannot start until the roof sheets have been fixed otherwise the sun could lead to rapid moisture loss from the concrete. This in turn could cause plastic cracking in the floor*

*Fig. 5.24. The prop columns along the centre of the building have been bolted to their concrete foundations which are below the level of the underside of the floor slab. The floor slab will surround these columns*

*Fig. 5.25. Lightweight steel mesh reinforcement has been fixed around the prop columns through the depth of the floor slab. A baseplate has been welded to the end of the column. The baseplate has been bolted to the foundation. Wind may cause the column to develop tension so it has to be bolted positively to its foundation*

*Fig. 5.26. Concrete has been cast around the column through the depth of the floor slab. A strip of compressible material will be fixed around the perimeter of this concrete to the full depth of the floor to ensure that the floor and the structural frame can move independently of each other*

*Fig. 5.27. A slip membrane has been laid and formwork has been fixed prior to concreting. The concrete truck reverses along the strip to the discharge position. Care has to be taken to avoid rutting the sub-base and disturbing the slip membrane. The mesh reinforcement cannot be installed prior to concreting as it would interfere with access*

*Fig. 5.28. The concrete is discharged between the formwork and, in this case, a previously constructed strip. A piece of mesh reinforcement has been installed. The concrete is discharged as close as possible to its required position and is moved by hand as required*

*Fig. 5.29. A standard size* (4·8 m × 2·4 m) *piece of mesh is ready to be placed in the fresh concrete. The mesh is orientated so the main reinforcement is in the longitudinal direction. The mesh is positioned vertically, then allowed to rotate onto the surface of the concrete*

*Fig. 5.30. The mesh is pressed into the concrete by members of the concrete gang walking over it. The mesh can be located with a surprising degree of accuracy by a skilled team. Care is taken to ensure sufficient overlap with neighbouring pieces of mesh. A cropping tool is on hand to deal with obstructions*

*Fig. 5.31. A double vibrating beam is pulled along the formwork to compact the concrete. A member of the team ensures that there is sufficient concrete along the length of the beam. Tie bars have been inserted in holes in the formwork*

*Fig. 5.32. A short length of mesh is placed near the underside of the concrete to act as the tie at a transverse tied joint. The main mesh reinforcement near the upper surface of the concrete is stopped short of the tied joint. Note that the main reinforcement runs normal to the longitudinal long strip direction. A transverse groove has already been cut in the neighbouring bay to complete the tied joint*

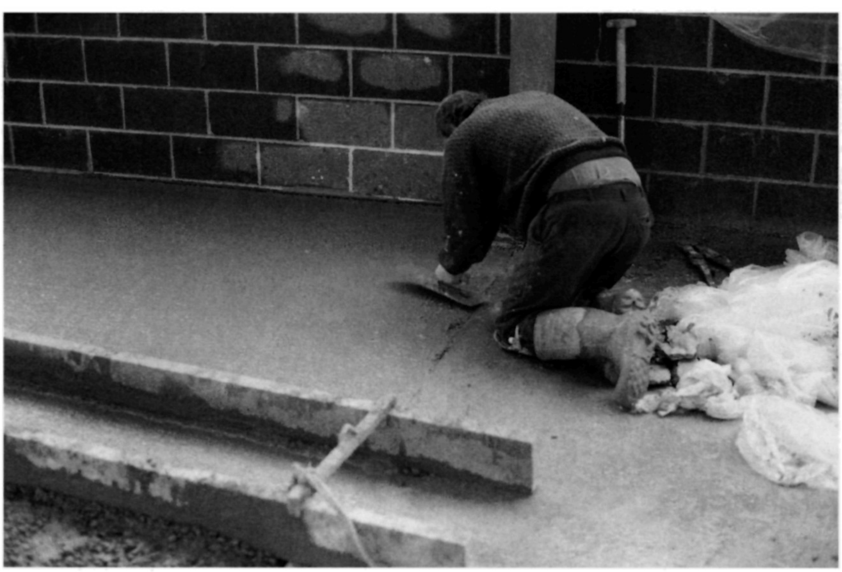


*Fig. 5.33. The edges of the slab are trowelled to improve the quality of the longitudinal joint. This trowelling takes place immediately after the concrete has been compacted. Additional concrete may be placed to make up levels*

*Fig. 5.34. The concrete is left to set. Note the excess slip membrane material and the tie bars. Because the roof sheets are fixed and the weather is cool, there is no need to take special measures to reduce moisture loss from the upper surface of the concrete*

*Fig. 5.35. The floor slab has been cast around an internal column. Compressible material has been fixed around the square concrete surrounding the column. The upper part of the compressible material will be replaced by a protective strip*

*Fig. 5.36. These power trowels are used to improve the quality of the surface of the concrete. They are used when the concrete has gained sufficient strength to support their weight and the weight of the operator, but before the concrete is fully hard. In this case, concrete mixed at 9 a.m. was power trowelled at 1 p.m.*

*Fig. 5.37. The effect of power trowelling is to create a smooth and durable surface. There must be just the correct amount of surface laitance. Too little and the surface will remain open. Too much and a thin layer may eventually spall at the surface*

*Fig. 5.38. The finished slab with the formwork removed showing the tie bars ready for the neighbouring long strip. This concrete can be used as formwork for the next long strip. Sometimes the alternate strip method is used whereby the initial long strips require formwork at each side and the intermediate strips use the previously cast concrete. In this case, one central long strip was cast using formwork at each side. All of the subsequent strips were cast using one existing concrete long strip and one side of formwork*

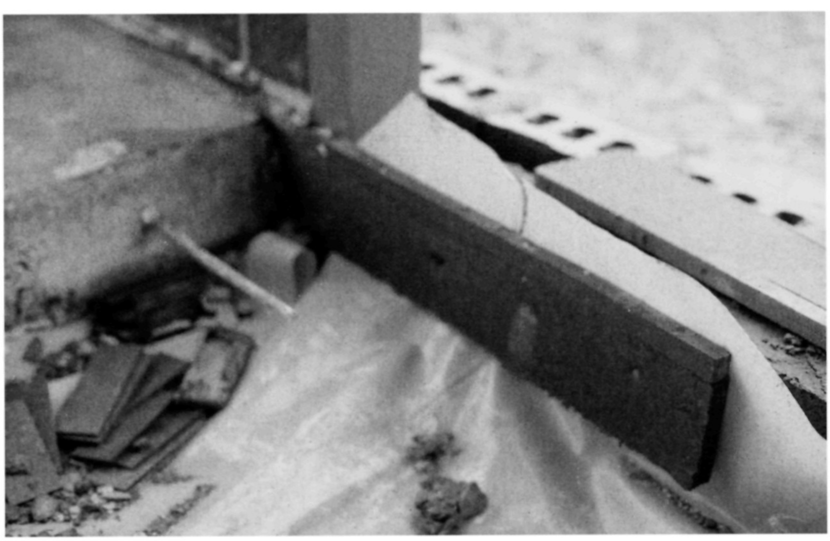


*Fig. 5.39. An isolation joint is formed at the building perimeter using the compressible material shown. In this case, the floor slab is constructed right up to the internal wall without a perimeter strip. The slip membrane is brought to the slab surface to contain the moisture in the concrete during curing*

*Fig. 5.40. At the external columns, the concrete block internal leaf is constructed round each column and a compressible strip is fixed to the concrete blocks prior to concreting. This isolates the floor from the internal leaf and from the main columns*

*Fig. 5.41. A 3 mm wide by 40 mm deep groove is cut into the concrete using a hand-held saw. Care has to be exercised to ensure a straight cut. The reduction in cross-sectional area and second moment of area ensures that cracking occurs here. Note that the main mesh is stopped here and a shorter piece having its main bars running along the line of the cut contributes to the weakening at such joints*

## 5.5 Long strip external industrial hardstanding construction case study

This example of a hardstanding constructed conventionally by the long strip method is presented in Figs 5.42–5.58. The hardstanding was constructed during 1999. A conventionally reinforced 150 mm thick concrete slab was constructed over a dolomitic limestone 150 mm thick sub-base. The hardstanding was constructed over a period of three months. One or two strips running from one side of the building to the other were constructed each working day. The reinforcement was placed near the upper surface of the slab and three types of joint were provided. The joints between neighbouring strips comprised conventional ties connecting the strips. The transverse joints comprised dowelled joints and tied joints. In the case of the dowelled joints, sleaves were placed over the bars at one side of the joint to allow contraction. The ties comprised a short length of mesh reinforcement placed at the underside of the slab.

*Fig. 5.42. Typical construction site. Note that the sub-base must support the weight of the readymix concrete delivery truck*

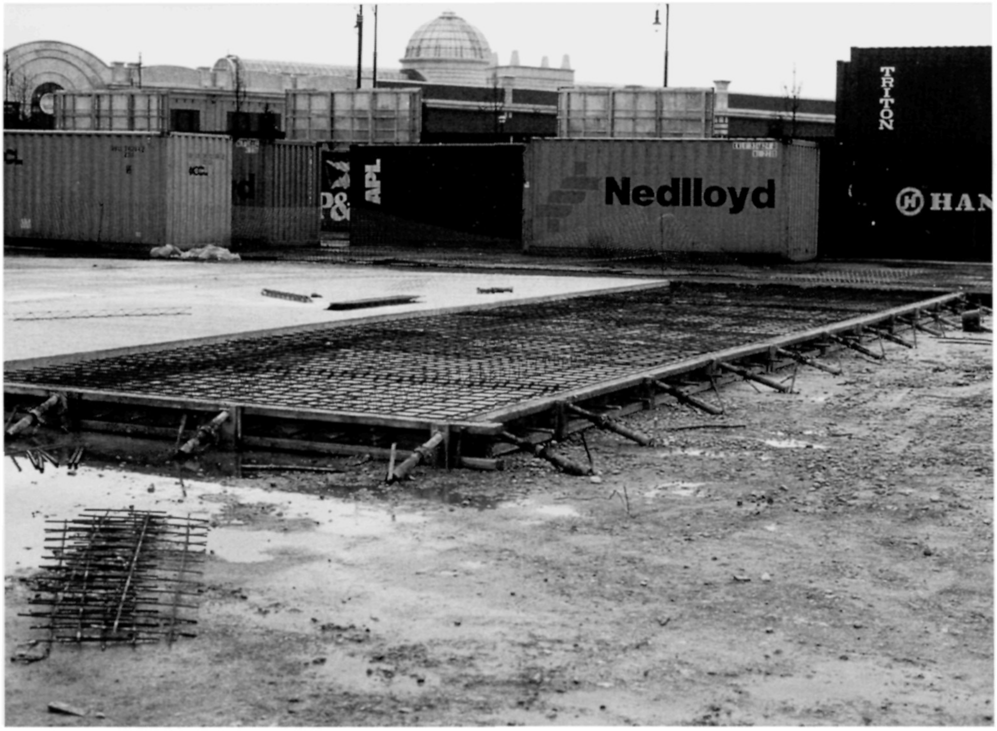



*Fig. 5.43. This bay is ready for concreting*

*Fig. 5.44. The concrete is being compacted while the operative ensures that there is a little concrete surcharge in front of the twin beam compactor*


*Fig. 5.45. Making good the concrete surface prior to applying the brushed finish*

*Fig. 5.46. Finishing the surface with the float prior to applying the brushed finish*

*Fig. 5.47. The next bay is ready for concreting as finishing continues on the previous bay*

*Fig. 5.48. A small excavator helps in placing the concrete. Because hardstanding concrete is of low workability, this saves much labour*


*Fig. 5.49. The excavator allows concrete to be placed at a rate of 15 m$^3$ per hour*


*Fig. 5.50. It is common for industrial hardstandings to be extended and care needs to be exercised to balance the needs of the contractor with those of the hardstanding operator*

*Fig. 5.51. A poker vibrator ensures that compaction is achieved to full depth. Care must be undertaken to avoid over vibrating which will lead to segregation*

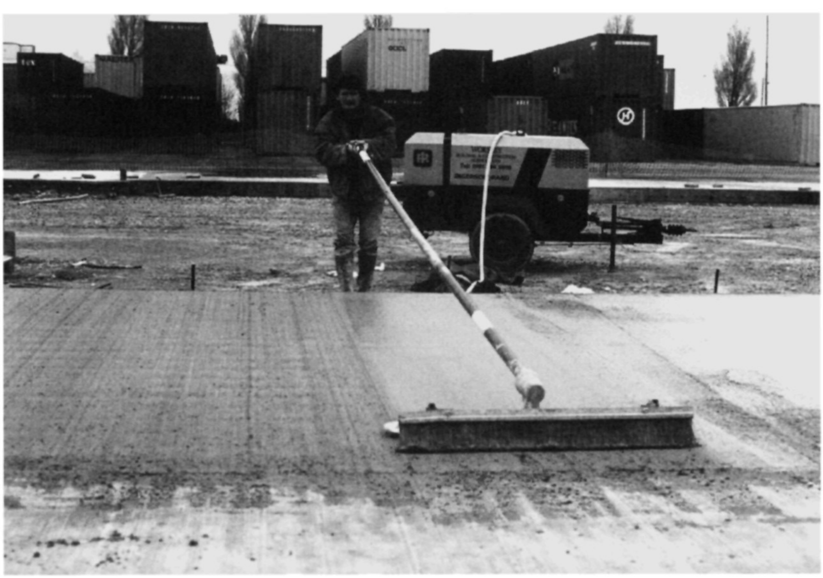


*Fig. 5.52. A brushed finish is achieved by this combination float/brush device*

*Fig. 5.53. The twin beam compactor is winched along progressively. The winch is attched to a heavy item of plant at each side of the twin beam compactor*

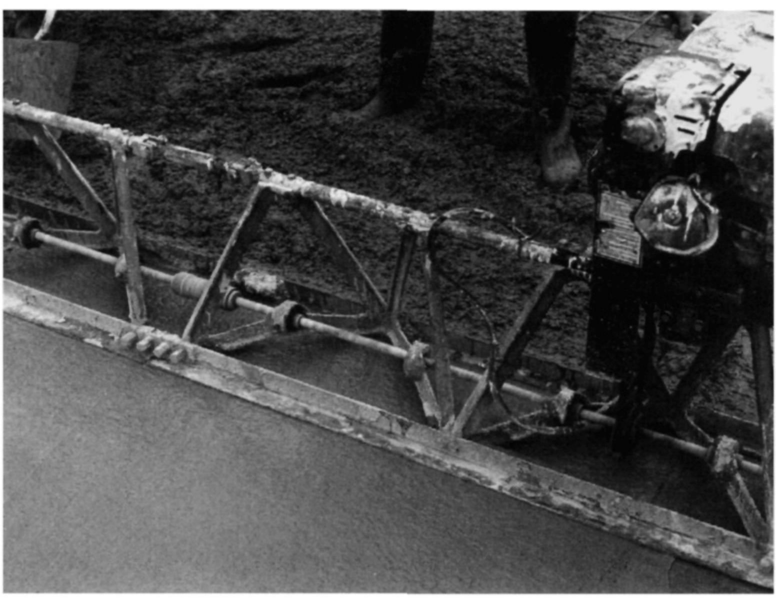


*Fig. 5.54. The motor powers a rotating shaft which has eccentric weights positioned regularly to ensure compaction takes place uniformly across the bay*

*Fig. 5.55. The winching of the twin beam compactor is coordinated with the maintaining of surcharge to ensure uniform compaction*

*Fig. 5.56. Before the readymix truck returns to the public road, the concrete adhering to its drum and discharge chute is washed away using the truck's on-board water supply. It is important to undertake this in an environmentally acceptable manner. The residue must not be washed into the drainage system. Also, any surplus concrete must be disposed of carefully*

*Fig. 5.57. It is common for safety fencing to be fixed to the surface of the concrete using expanding bolts*

*Fig. 5.58. It is inevitable that trenches will be formed in the hardstanding during its life. To attempt to minimise this, ducts should be included where possible to permit the later installation of services with as little disturbance as possible to the surface*

## 5.6 Unreinforced concrete road construction case study

The project illustrated in Figs 5.59–5.73 comprises an unreinforced concrete road in which transverse joints were spaced at 6 m centres. Such close centres allowed loads to be transferred via aggregate interlock, so dispensing with the need for dowel bars. The concrete comprised 200 mm thick C40 pavement quality concrete in which frost resistance was achieved through the use of polypropylene fibres rather than the more traditional air entrainment. Polypropylene fibres are being used commonly in external concrete ground bearing slabs to provide frost resistance. The road is used to transport coal won at an open cast site in Northumberland to a railhead where it is delivered to power stations. It was constructed in 2002. By eliminating steel entirely from the concrete, the owner of the road was given permission to eventually break up the road and place the resulting hardcore in the open cast excavation.

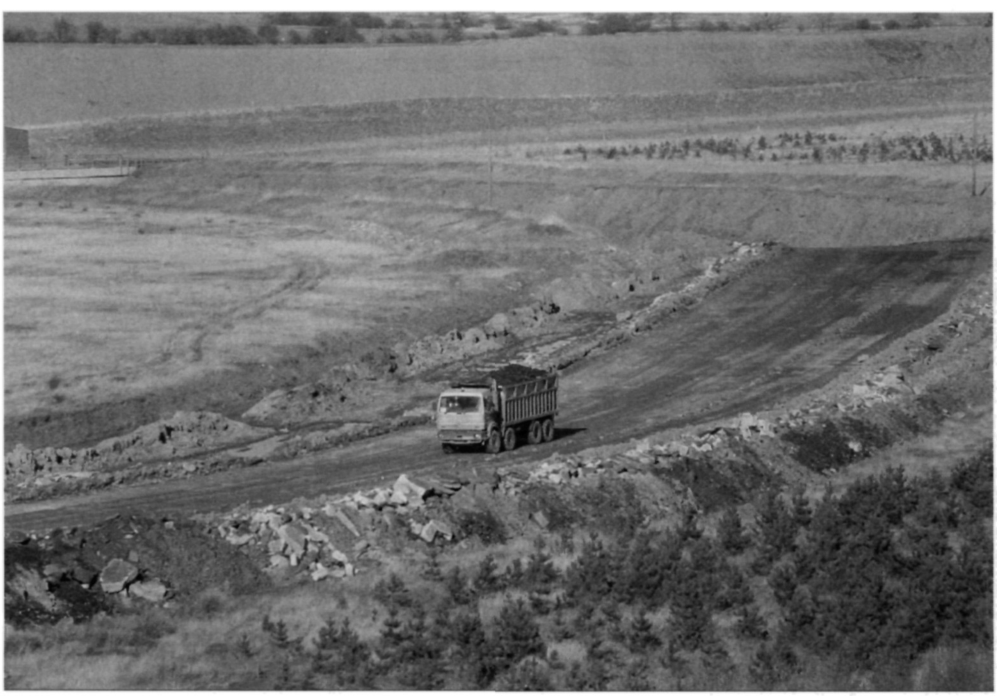


*Fig. 5.59. Until the construction of the road, coal from an open cast site had been transported to a railhead over unmade roads. Production of coal was frequently lost as these roads became unserviceable in wet weather*

*Fig. 5.60. Vegetation has been stripped and subgrade fill has been installed. Timber profiles have been fixed along the length of the road to allow levels to be established and verified. The granular sub-base material will be installed following the drying and final preparation of the subgrade. Note the open drainage ditch to the left of the line of the road*

*Fig. 5.61. The 150 mm thick granular sub-base material has been installed and formwork has been pinned to the sub-base. The sub-base has been compacted using a vibrating roller — see Section 1.2*

*Fig. 5.62. Here the road runs over an embankment. Large mudstone boulders are used to stabilise the sides of the embankment*

*Fig. 5.63. A woven geotextile fabric separates the sub-base from the underlying mudstone fill. This prevents the sub-base material from penetrating the fill, while allowing the free drainage of water*

*Fig. 5.64. The formwork has been located at the correct level over a piece of mudstone. Although this will allow the escape of concrete from the gap beneath the formwork, it represents a pragmatic solution. Note the sliding formwork connector*

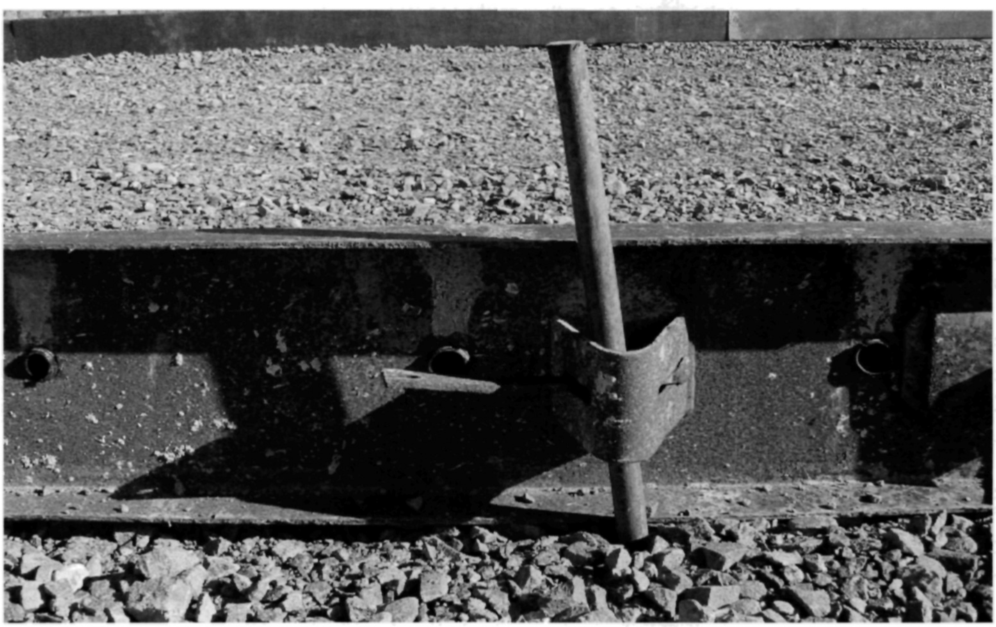


*Fig. 5.65. The formwork is located firmly using steel pins so as to resist the horizontal forces which the concrete will apply and to prevent the formwork from being disturbed accidentally. Note the holes in the formwork that allow dowel bars to be inserted*

*Fig. 5.66. Dowel bars have been fixed along the edge of the road where a passing lay-by is to be concreted. Note the difficulty in ensuring that the dowel bars are all parallel. Even one misaligned dowel bar can lead to the joint locking, which can cause cracking of the concrete*


*Fig. 5.67. Concrete being delivered. This narrow road required the concrete delivery vehicle to reverse the whole length of the road. This can lead to rutting of the sub-base and it is sometimes necessary to recompact the sub-base material between deliveries. The rotating tubular beam compacts the concrete*


*Fig. 5.68. The vibrating tube compacts the concrete as it is pulled along the formwork. Note the daywork joint in the foreground with holes in the formwork for the introduction of dowel bars*


*Fig. 5.69. Additional vibration is introduced using a poker vibrator to ensure that full density is achieved throughout the full depth of the concrete. The vibrating tube is most effective close to the surface of the concrete*

*Fig. 5.70. The concrete has been compacted and a tamped finish has been introduced. This will ensure sufficient skid resistance. Curing compound is about to be sprayed on to the surface of the concrete*

*Fig. 5.71. 4 mm wide by 60 mm deep saw cuts are formed at 6 m centres to reduce stresses resulting from temperature related volume changes. The saw cuts will be sealed to prevent the ingress of detritus. The saw cuts are made as soon as the concrete can be cut without damage. Waiting too long could lead to mid-bay cracking*

*Fig. 5.72. Even with good levels of workmanship and supervision, lack of compaction can occur. Whether such material should be removed will depend upon the anticipated loading in the vicinity of the uncompacted concrete and upon the need to protect the dowel bars from corrosion*

*Fig. 5.73. The elimination of steel mesh and dowel bars led to rapid progress, an important factor in many industrial situations*

## 5.7 Warehouse floor failure case study

A warehouse floor in Kent failed as a floor load of 35 kN/m$^2$ led to consolidation settlement of underlying layers of peat. After five years, differential settlement had attained a maximum value of 250 mm and appeared to have stabilised at that level. The structural frame of the building was piled. The floor was constructed over the pile caps at the location of the columns and therefore remained at its constructed level. Between the columns, the floor settled differentially, forming a catenary shape. This led to high stacks of newsprint stored in the warehouse leaning and becoming unstable. Also, the ensuing cracks led to loose stones that damaged the underside of reels of newsprint on several occasions.

The floor should have been installed independently of the pile caps so as to allow it to settle uniformly. While this would have introduced a maintenance liability at the entrances to the warehouse, it would have proven workable. A cost effective solution was to cut the floor free of the pile caps and to load it more heavily close to those pile caps and thereby allow it to stabilise at a uniform level. While this would have proven a hindrance to the operation of the warehouse, it nonetheless was considered by several parties to be the best way forward. Other solutions, all far more expensive, involved the removal of the floor and the provision of an alternative piled floor. Details of the case study are shown in Figs 5.74–5.78.

*Fig. 5.74. Newsprint reels stored on this floor are leaning as the floor falls away from hard points at the internal columns. The columns had been piled and the floor has been concreted over the pile caps. The subgrade comprises fill material overlying peat. Consolidation settlement of the peat led to the floor settling differentially and cracking. This could have been avoided by allowing the floor to move vertically relative to the pile caps at the columns*

*Fig. 5.75. The cracking in the floor led to the paper reels being stored over uncracked locations and plant having to negotiate the cracks. This proved a hindrance to the operation of the warehouse*

*Fig. 5.76. A hard spot has been left in place beneath the floor leading to this pattern of cracking. The concrete near the keys is higher than the surrounding concrete*

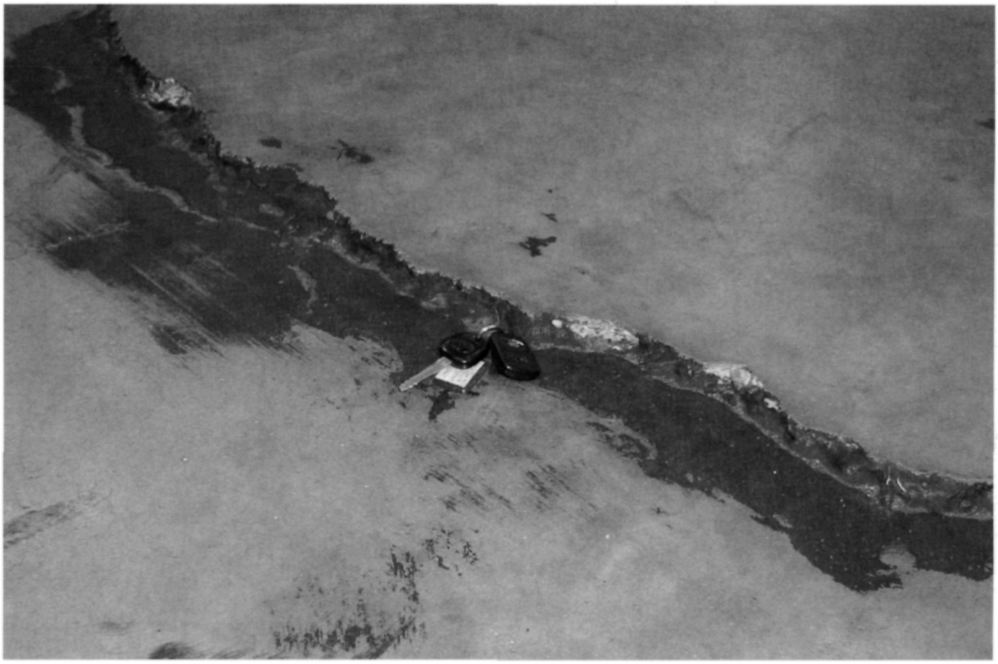


*Fig. 5.77. An attempt has been made to seal a crack but continued relative vertical movement of the slabs each side of the crack is leading to the need for continued maintenance*

*Fig. 5.78. The movement of the floor is causing a tearing action to occur. The concrete is being held together by steel fibres spanning the tear. Trafficking will remove the torn pieces leading to a severely spalled joint*

## 5.8 Industrial hardstanding failure case study

Figures 5.79–5.85 illustrate an external hardstanding that cracked as a result of its unreinforced concrete bays being of dimensions $9 \times 9$ m. Experience indicates that the maximum bay size for unreinforced concrete slabs should be $6 \times 6$ m. Effectively, the slabs have divided themselves into smaller bays through their cracking.

Approximately 25 000 m$^2$ concrete was removed and replaced with mesh reinforced concrete. The mesh is located towards the upper surface of the concrete to manage any further cracking that might develop.

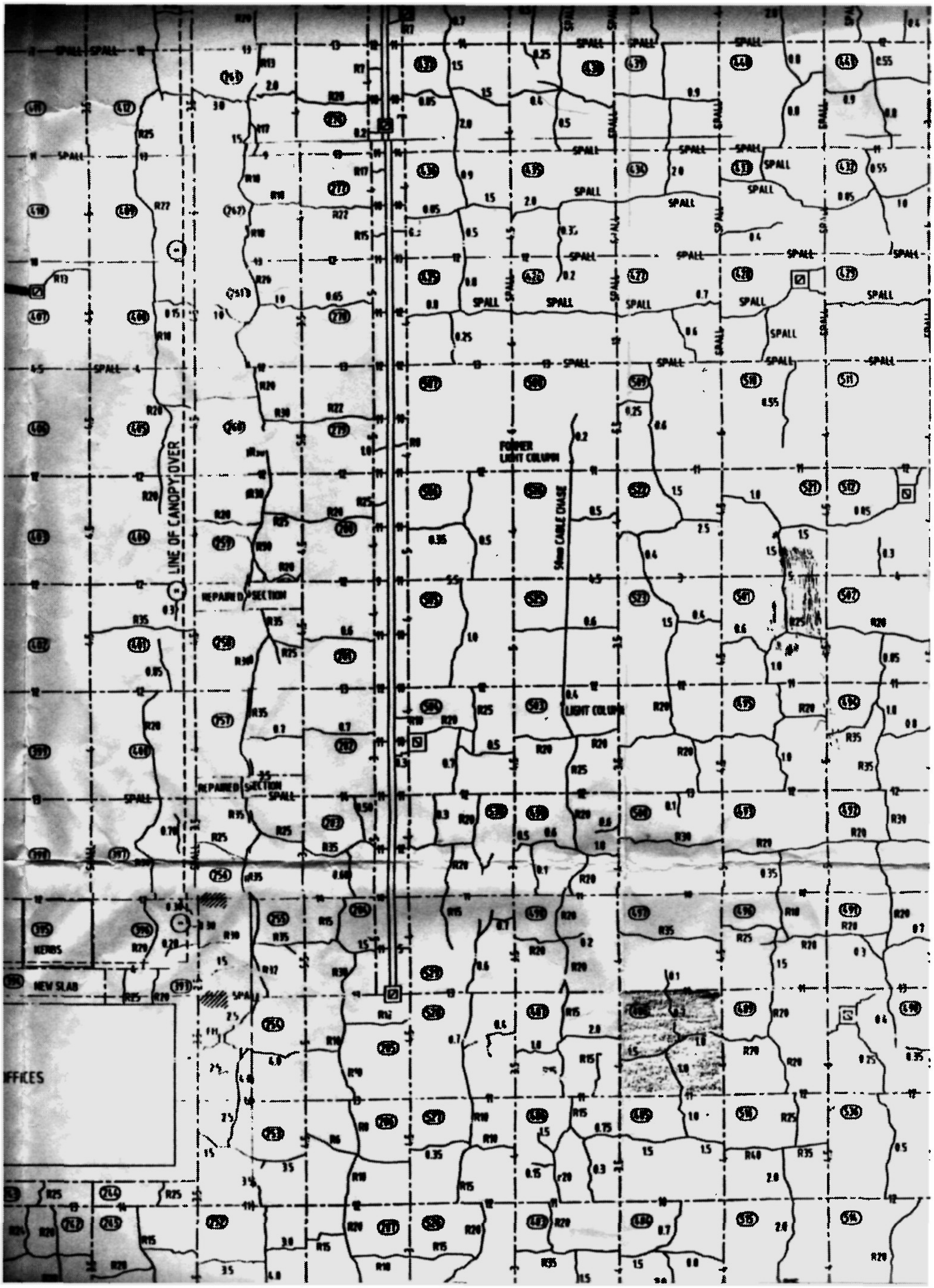



*Fig. 5.79.* *Pattern of cracking in an unreinforced external hardstanding. The joints are on the square grid shown. Each square is 9 × 9 m, which is too large, hence the cracks have sub-divided the squares into ones in which the stresses are less*

*Fig. 5.80. Cracks have been sealed and the external hardstanding remains serviceable*

*Fig. 5.81. The cracked slabs were broken out and reconstructed with mesh reinforcement placed near the upper surface of the slabs. Timber formwork was used with holes drilled for dowel bars*

*Fig. 5.82. A polyethylene slip membrane was introduced and the mesh was supported on fabricated supports*

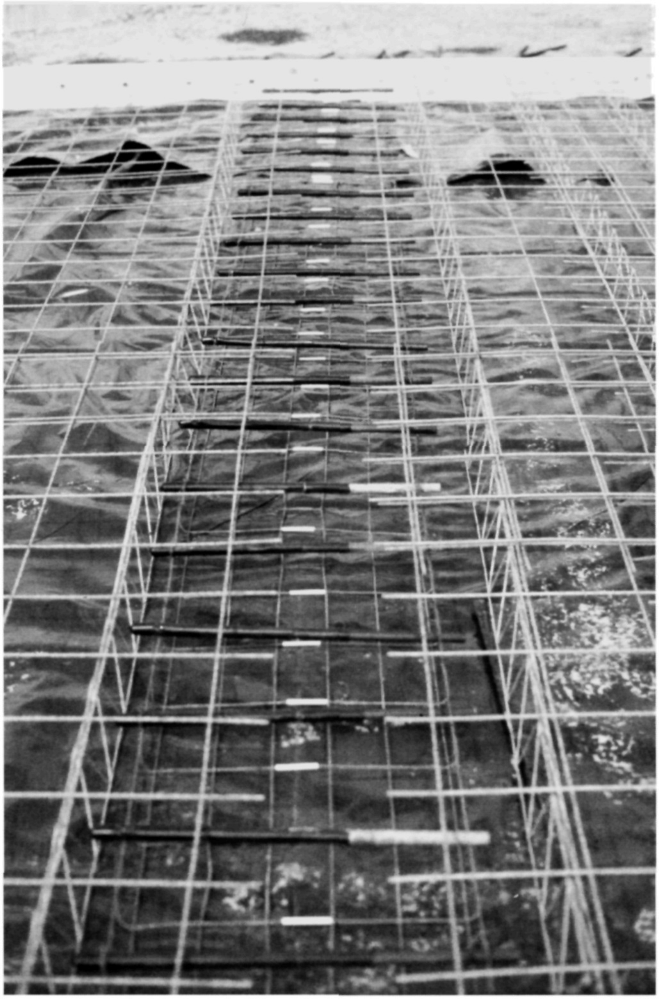


*Fig. 5.83. At the joints, the mesh reinforcement was stopped and a lighter mesh was placed across the joint at slab mid-depth to reduce the moment capacity of the joint. Dowel bars, half of which were coated with debonding compound, were installed*

*Fig. 5.84.* *Note how the operative deforms the mesh as he gains access to the concrete. It is common for mesh to vary in location as a result of its having been displaced in this manner*

*Fig. 5.85.* *Dowel bars are inserted into the holes drilled through the timber formwork. It can be difficult to ensure that all of the dowel bars remain parallel*

## 5.9 Perimeter beam to a floor slab

Before a ground bearing floor is installed, it is usual to construct a perimeter beam. This defines the level of the floor, acts as permanent formwork and accommodates level differences between the floor and the building's external works. Figures 5.86–5.93 illustrate the installation of a perimeter beam. This example was installed at Port of Tyne's Tyne Dock complex. The building comprises a high bay storage warehouse whose arrangement of portal frames was dictated by the layout of the racking system.

Figure 5.87 shows a typical 3 × 3 m pad foundation. The site comprises a layer of competent material overlying weak fill material which was used to fill in the old Tyne Dock. This large foundation reduces the stress introduced by the column into the underlying fill. The floor is to be supported by a system of stone columns termed 'Vibrofloatation'.

Figure 5.88 shows the perimeter beam (sometimes referred to as a ring beam) formwork in place. Note the steel ties used to prevent the hydrostatic pressure exerted by the fresh concrete pushing the formwork apart. Even for relatively shallow pours such as this, the horizontal forces applied by the concrete to the formwork are sufficiently large for a full structural analysis of their effects to be advisable.

Figure 5.89 shows how special care has to be taken in constructing a perimeter beam at a column. Here, the beam passes the external face of the column. More complex formwork is required to allow the beam to negotiate the inner face of the column.

*Fig. 5.86. This building at Port of Tyne's Tyne Dock is designed to accommodate a high bay racking storage system. Each portal has two props at third points and each prop has a pad foundation beneath the floor*

*Fig. 5.87. Note the 3 × 3 m pad foundation. This spreads the column load so as to avoid overstressing the underlying weak fill material which had been used to fill the old Tyne Dock*

*Fig. 5.88. Perimeter beam formwork in place awaiting concrete to be placed*

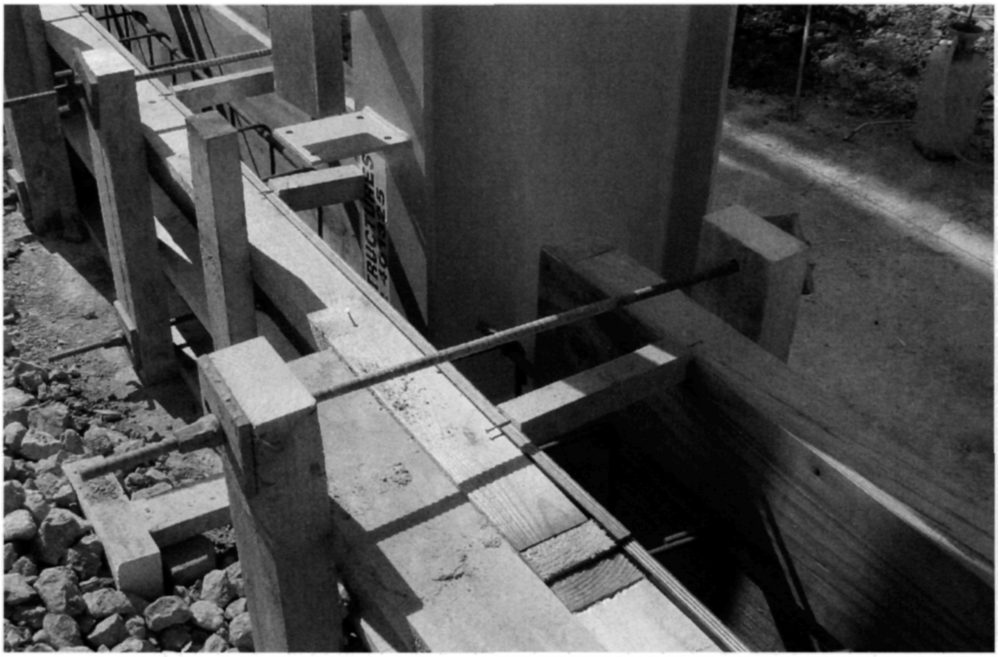


*Fig. 5.89. The perimeter beam formwork passes the external face of a column*

*Fig. 5.90. The reinforcement has been fixed firmly to avoid disturbance when the concrete is poured*

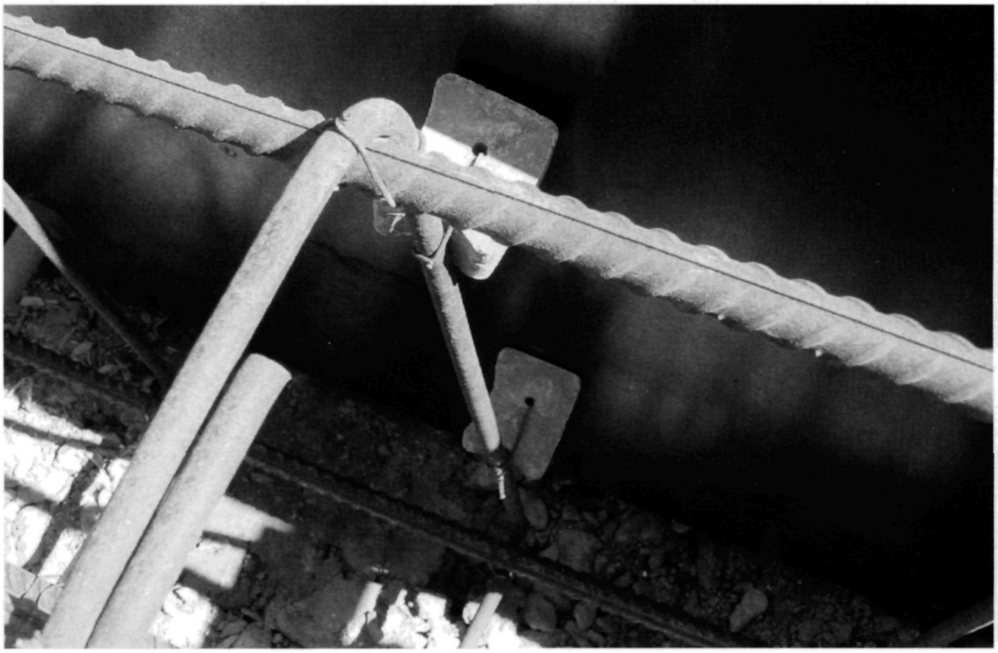


*Fig. 5.91. The reinforcement is kept away from the formwork by the spacers wired to the vertical links*

*Fig. 5.92. Perimeter beam formwork positioned at the inner face of a column*

*Fig. 5.93. The perimeter beam has been cast successfully around this column. Note the absence of air pockets in the concrete indicating a good level of compaction*

Figure 5.90 shows the reinforcement installed and ready for concreting. Note how the 16 mm diameter horizontal bars are positioned at each face at the top and the bottom of the beam, following the recommendations of BS 8110 *The structural use of concrete*.[1.8] The bars are connected together by links so as to form a reinforcement fabrication that will withstand the pressures developed when the concrete is placed.

Figure 5.91 shows the use of spacers to ensure that the reinforcement will have sufficient cover after the concrete has been poured. The spacers are wired to the reinforcement links in the same way that the links are wired to the main horizontal reinforcement bars. The whole fabrication has sufficient strength to withstand the effects of concreting. Not only will the concrete press onto the reinforcement but the poker vibrators, which will be used to compact the concrete, could displace the bars if they were not fixed firmly. Note that the inner face of the formwork has been treated with a release agent to prevent the concrete from being damaged when the formwork is struck. It also reduces damage to the plywood formwork so allowing it to be reused, say, five times.

Figure 5.92 shows how the perimeter beam formwork is positioned at the inner face of a column and Fig. 5.93 shows the completed perimeter beam at a column. An accurately positioned well designed and constructed perimeter beam greatly enhances the quality of the final floor.

## 5.10 Traditional mesh reinforced floor

In this case study (Figs 5.94–5.101), a 200 mm thick smaller mesh reinforced concrete floor is being installed as an extension to an existing reinforced concrete floor. The project was constructed in January 2002 at the Silverlink retail development in North Shields, UK. High winds on the day of concreting caused some difficulties, as shown in Figs 5.100 and 5.101. This is typical of a project of area 300 $m^2$ to 1000 $m^2$, where it would be inappropriate to use laser guided screeding equipment.

*Fig. 5.94. A concrete floor is being installed in an extension to a building on a retail development in North Shields, UK. A piece of A142 mesh is being carried towards the concrete laying face. The floor is being cast against a concrete block wall to the left and an existing concrete floor to the right*


*Fig. 5.95. The concrete delivery truck has to run over the mesh and this can lead to its being displaced. In this case, the mesh is positioned towards the underside of the concrete so the displacement is minimal. Designers should be aware of this*

*Fig. 5.96. The concrete is placed so as to avoid further movement of the material by hand. Inevitably, the reinforcement is deformed by the operatives and by the weight of the concrete*

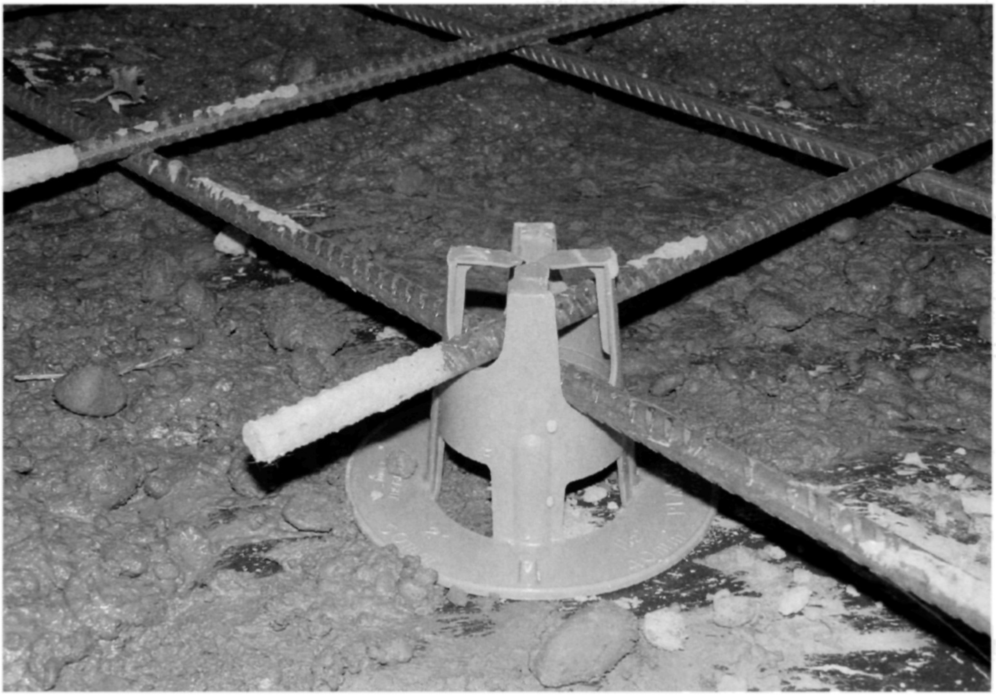


*Fig. 5.97. The mesh reinforcement is supported on plastic stools which are located at four bar intervals in orthogonal directions*

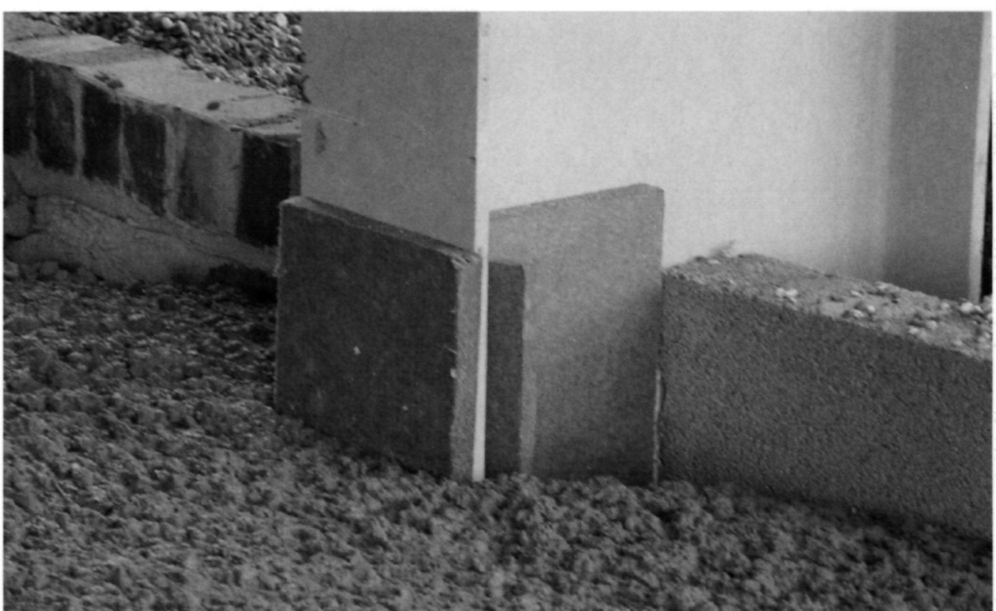


*Fig. 5.98. Flexible fibre board is placed around the steel columns to provide a full movement joint between the floor and the structure. The concrete is being cast directly against brickwork to the left and concrete blocks to the right*

*Fig. 5.99. The concrete has been compacted and screeded, and awaits power floating when it has gained sufficient strength to support the trowelling equipment. The fibre boards have ensured structural separation between the floor and the column, and will be trimmed later*

*Fig. 5.100. As finishing proceeds, difficulty is being experienced in placing the polythene sheeting on the sub-base on a very windy day. To the left, a piece of mesh is being cut to size*

*Fig. 5.101. Although high wind has little impact upon floor installation, high winds during January 2002 caused sheeting stored on the roof of the adjoining portal framed building to blow over and damage the purlins. The safety nets would not arrest any heavy falling sheets. Gusts in excess of 35 m/s were measured nearby and this should be compared with the basic design wind speed of 46 m/s at this location. The basic design wind speed is the average speed of a three second gust likely to recur every 50 years. Because concrete floor construction always takes place in the context of the construction or modification of a building, factors such as high wind may impact floor installation*

## 5.11 Industrial road defects

A 5 m wide single lane road constructed in 2001 developed cracking soon after its construction. The road comprised the perimeter road to an electricity sub-station in Scotland. Although the in-service traffic was expected to be light, there was considerable heavy traffic during the construction phase. A particular issue was the movement of construction traffic over the edge of the road. Remedial work comprised the removal of concrete which displayed full depth wide cracks and treating the entire road with surface dressing to seal the minor cracks and joints. Details of the defects are shown in Figs 5.102–5.107.

*Fig. 5.102. This 5 m wide single lane industrial road was heavily trafficked during construction but should receive only light traffic during its service life. Construction traffic caused a number of defects*

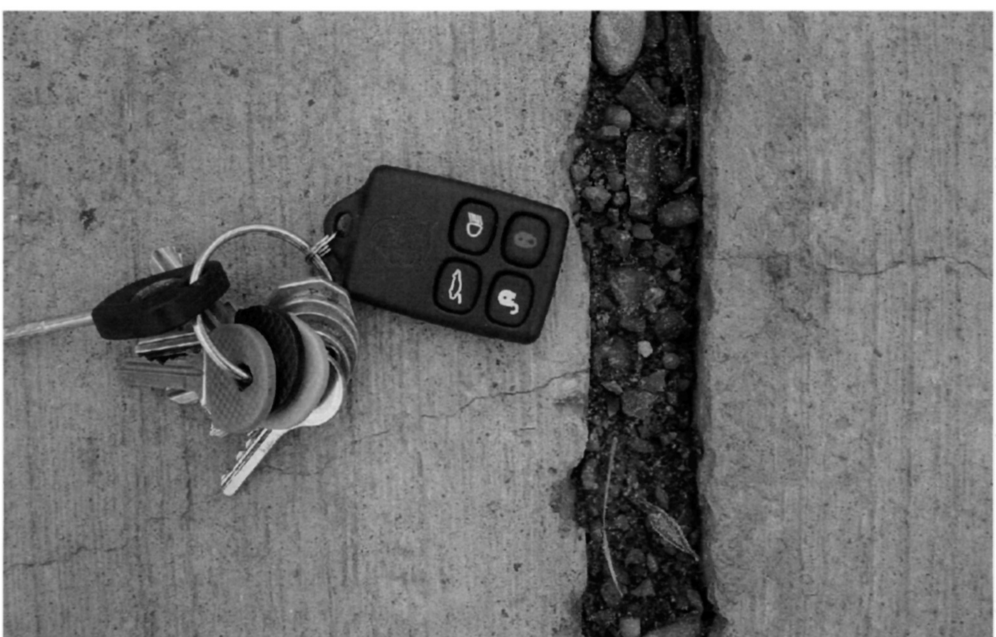


*Fig. 5.103. Minor longitudinal cracks caused by hogging moments when heavy construction vehicles trafficked the site here meet a full movement joint*

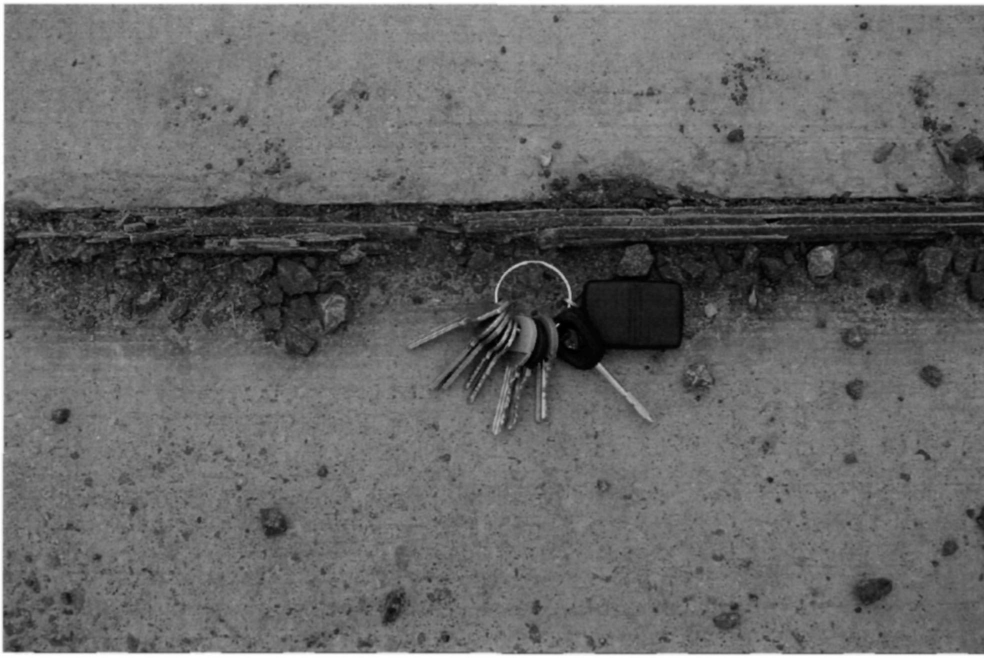


*Fig. 5.104. A length of timber fillet has been left in this joint. it should have been removed and replaced with a sealant. Detritus is entering the joint and causing deterioration of the adjoining concrete*

*Fig. 5.105. This joint has not been sealed so allowing a stone to become lodged between the concrete slabs and spalling the concrete*

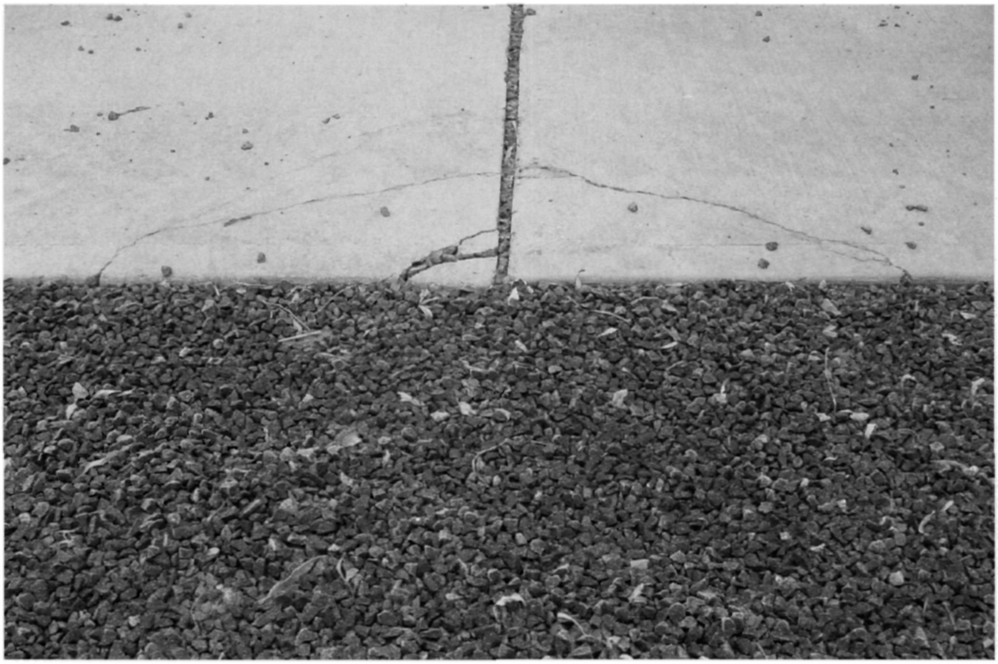


*Fig. 5.106. Two types of failure have occurred here. The narrower cracks are classical corner cracks and indicate inadequate structural capability. The wider cracks in the near corner of the left slab are explained in Fig. 5.107*

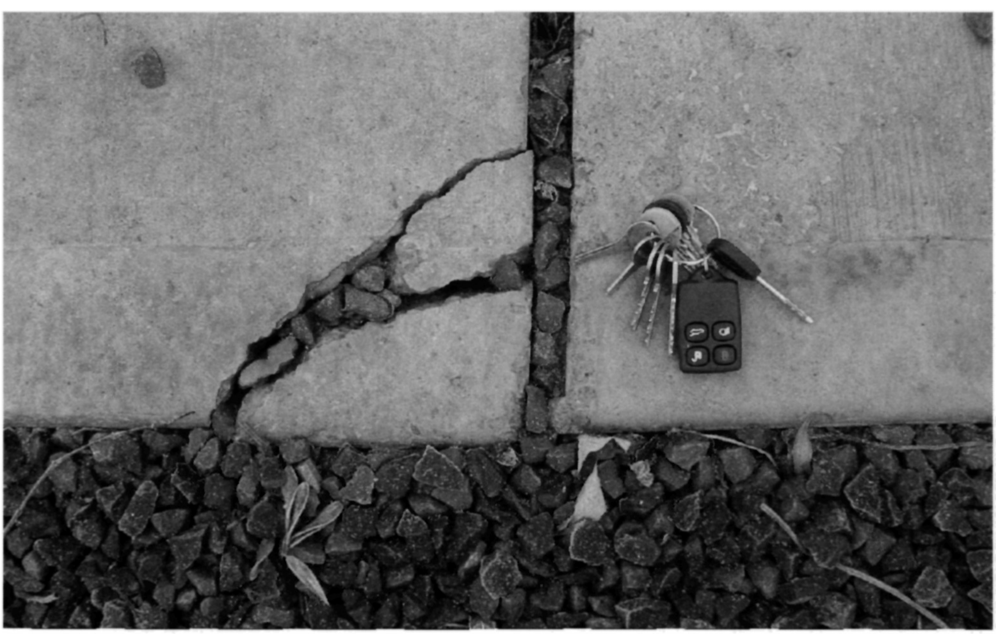


*Fig. 5.107. This corner detail from Fig. 5.106 has deteriorated as a result of stones initially entering the unsealed joint, so prising the concrete apart. The same stones then entered the cracks and caused local failure of the corner of the slab*

## 5.12 Overlaying a cracked concrete industrial hardstanding

A reinforced concrete hardstanding at Port of Tyne's Tyne Dock container handling facility has sustained cracking as a result of it having been trafficked by front lift trucks with front axle loads of 95 t. It was decided to install a concrete block paving overlay to strengthen the pavement and thereby to extend its life. Trials were undertaken to check that the cracked concrete had sufficient residual strength to accommodate further loading with a concrete block overlay. The existing concrete comprised 250 mm thick C35 mesh reinforced concrete with a 150 mm thick lean concrete sub-base on fill material.

Before undertaking the overlay, the existing concrete was surface dressed to seal the existing fine cracks. The wider cracks were treated separately. Ramps were installed around the overlay where it interfaced with other hardstandings and, elsewhere, edge restraint was introduced. The concrete block paving joints were treated with a stabiliser/sealer to reduce the ingress of moisture. Details of the case study are shown in Figs 5.108–5.112.

*Fig. 5.108. This concrete hardstanding has proven inadequate to sustain regular trafficking by front lift trucks with 95 t front axles handling 40 ft containers. In order to continue using the concrete in the same way, it was decided to undertake a concrete block paving overlay*

*Fig. 5.109. Precast concrete units had been installed previously as permanent formwork at the slab joints. These had performed particularly poorly and were one of the reasons for the need for an overlay*


*Fig. 5.110. Testing comprised loading the whole of the existing concrete hardstanding with the maximum load likely to occur during the life of the hardstanding and checking that none of the existing slabs displayed significant vertical movement*

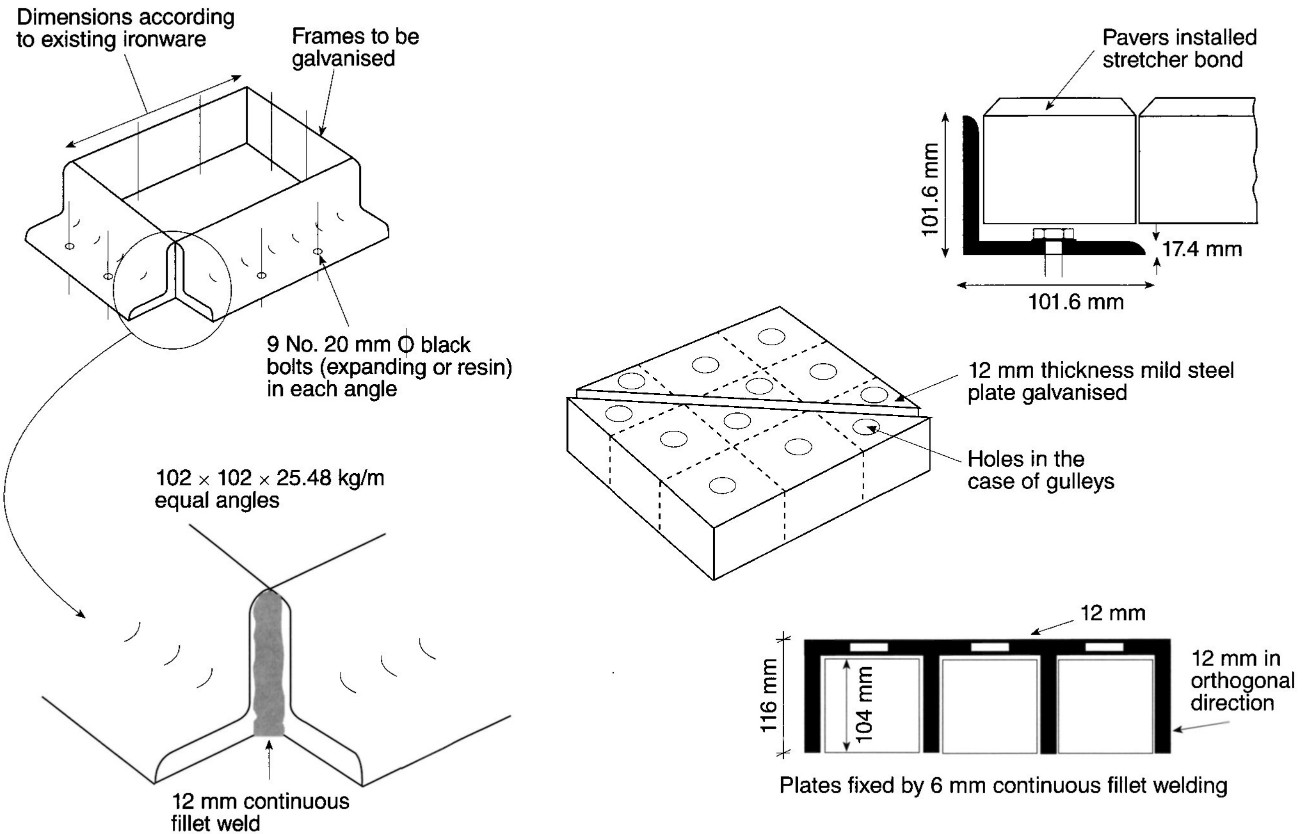



*Fig. 5.111. Existing gulleys and inspection covers were retained at their original level and the steel angle section was fabricated to form a raised surround. Triangular boxes were fabricated to fill the void above the existing ironware. The boxes were designed to facilitate their lifting to provide access to the gulleys*


*Fig. 5.112. Concrete block paving is being installed as an overlay on this container handling yard. Only 30% of the area needed to be taken out of service at any one time. By careful planning of both construction and container handling, disruption to the handling of containers was minimised*

The specification for the overlay was as follows.

## 1. Concrete block pavers

The pavers shall be supplied as new material according to BS 6717: 2001 *Precast, unreinforced concrete paving blocks — Requirements and test methods*, BSI, London (ISBN 0 580 33308 6). They shall be rectangular pavers of work dimensions 100 × 200 mm in plan and 80 mm in thickness. They should include spacer nibs and chamfers whose dimensions are to be stated by the manufacturer. Weathering resistance shall be Class W2. Abrasion resistance shall be Class A2. Slip/skid resistance shall be Class S4. The manufacturer shall provide a statement setting out their quality control system and shall include details of the accreditation body.

## 2. Installation of pavers

The pavers shall be installed according to BS 7533: Part 3: 1997 *Pavements constructed with clay, natural stone or concrete pavers. Part 3. Code of practice for laying precast concrete paving blocks and clay pavers for flexible pavements*, BSI, London (ISBN 0 580 27071 8). Unless otherwise stated or indicated, falls shall be 1·5%. Nominal sand thickness shall be 30 mm. This may vary between 20 mm and 60 mm, these figures being the absolute limits and not subject to tolerances. Details at ironwork and edge restraint shall be as on Port of Tyne Drawings. Laying course material shall be Category II as described in Annex D of BS 7533: Part 3: 1997. Edging shall be with a single stretcher course as shown on Fig. E2a of BS 7533: Part 3: 1997. All cutting of pavers shall be by saw, not by guillotine. Tents shall be on hand at all times and shall be used to ensure that precipitation does not saturate the laying course material during construction. Any saturated sand shall be removed and discarded.

### 3. Surface dressing

Prior to application of the laying course material, the whole of the area to be paved shall be surface dressed according to Road Note 39 *Recommendations for road surface dressing*, second edition (1989), Department of the Environment, Transport and Road Research Laboratory (HMSO). The contractor shall select an appropriate surface dressing specification based upon the following. Hardness Category of substrate — Very Hard (Table 1, Road Note 39). Approximate number of comercial vehicles per day in the lane under construction — 500 (Table 3, Road Note 39). Target binder content shall be according to Table 4 of Road Note 39. Rate of spread of chippings shall be according to Table 5 of Road Note 39. The binder type shall be selected from one of those stated in Clause 62 of Road Note 39. The nature of the binder proposed shall be reported to Port of Tyne Authority and shall be subject to approval prior to commencement of surface dressing. The binder viscosity shall be according to Table 6 of Road Note 39, depending upon the time of year of the work being installed. Spraying temperature shall be according to Table 8 of Road Note 39. If modified binders are being proposed, the recommendations of Appendix 2 of Road Note 39 shall be followed and the proposal shall be reported to Port of Tyne for approval prior to commencement.

### 4. Ramps

Where shown on the drawings, ramps shall be provided between the overlain areas and the existing concrete or asphalt surface. The ramps shall comprise C35 pavement quality concrete with a minimum cement content of 320 kg/m$^3$. They shall be installed to a slope of 2%, which indicates a nominal length of 6 m. The maximum aggregate size shall be 37 mm, although 25 mm aggregate will be acceptable. The concrete shall include steel fibres at a dosage of 30 kg/m$^3$ and polypropylene fibres at a dosage of 0·91 kg/m$^3$. The polypropylene fibres shall have been tested and the test results shall demonstrate that they are equal to or better than air-entrainment at resisting frost action. The concrete shall not be air-entrained. The concrete shall be fully compacted. The ramps shall have a brushed finish whose direction shall be at right angles to the slope. Water/cement ratio shall not exceed 0·5. A curing compound shall be applied to the surface of the concrete immediately following the brushing operation. Joints shall be provided by saw-cutting to a depth of 75 mm at 10 m centres, with the joints running in the direction of the slope. Joint width shall be 4 mm. Formed joints to be constructed according to contractor's requirements. Where formed joints are installed, 25 mm diameter 600 mm long half-debonded dowel bars shall be provided at approximately mid-depth at 300 mm centres.

### 5. *Making good defects in existing concrete*

Where concrete has deteriorated such that multiple cracking has developed in a localised area, the material shall be removed to full depth and new concrete to the specification in (4) shall be installed. The face of the removed material shall be vertical but keyed. The excavation shall be clean and dry prior to the installation of the new material. Cracks of width 1 mm to 5 mm and all joints shall be cleaned out and filled with a two-part epoxy mortar installed according to the manufacturer's instructions. This shall be undertaken prior to surface dressing. All loose concrete shall be removed and any stubborn oil or similar material shall be removed by steam cleaning prior to surface dressing.

### 6. *Edge restraint*

In-situ concrete shall be as specified in (4) except that 20 mm maximum size aggregate shall be permitted. 20 mm diameter vertical steel dowels of length 240 mm shall be inserted into holes of depth 120 mm at 1 m centres along the length of the edge restraint. The dowels shall be grouted into the existing concrete and shall protrude vertically by no more than 120 mm. The edge restraint shall be of width 240 mm and height 120 mm or to the level of the surface of the overlay whichever is greater.

### 7. *Paver joint sealant*

Immediately following the installation of the pavers and before precipitation wets the surface, joint stabiliser shall be applied to the surface of the pavers. The material shall be applied according to the manufacturer's instructions. The material shall be RESIBLOCK 22 or similar approved. The material shall be a moisture curable liquid pre-polymer in an aromatic and/or aliphatic solvent having the following properties.

(*i*) Surface curing time not to exceed 24 hours (trouch dry — approximately 3 hours).
(*ii*) Solids content shall be not less than 20%.
(*iii*) Be highly resistant to oils, fuels, petrol, and de-icing fluids.
(*iv*) Appearance shall be clear to straw coloured and be free from foreign matter.
(*v*) Elongation at break of film — not less than 350% (ASTM D2370).
(*vi*) Withstand temperatures in excess of 200°C.
(*vii*) Withstand direct jet of water at a pressure no less than 1500 psi. (Lance 150 mm from surface.)

The coverage rate for the sealer will vary depending upon such factors as width of joints, porosity of pavers and ambient temperature but shall be of the order of 2·2 $m^2$ per litre. The sealer must be capable of penetrating the joints to a minimum depth of 20 mm and shall form a flexible elastomeric water and fuel resistant bond.

### 8. *Level make-up material*

Where new surface levels would otherwise lead to a thickness of laying course material exceeding 60 mm, a make-up layer of dense bitumen macadam roadbase shall be installed according to Series 900 of DTp *Specification for Highway Works* (1991). Clause 907 'Regulating Course' shall apply. The contractor shall inform Port of Tyne of his proposals in terms of materials and construction details. Hand placing and raking shall be permitted in those locations set out in Clauses 901/10 and 901/11.

### *9. Gulleys and inspection covers*

Existing gulleys and inspection covers shall be left in-situ and a boxed out void shall be formed using welded steel angle section as specified on the drawings. Double triangular stiffened steel boxes shall be fabricated and placed over the existing gulleys/inspection covers. In the case of gulleys, the boxes shall include holes in the upper plate surface to allow the continued use of the gulley. The steel angle surrounds and the boxes shall be galvanised.

## 5.13 Flatness

### *5.13.1 The need for a flat floor*

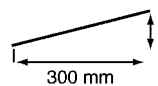
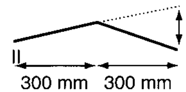
Flatness is an essential requirement of many ground floor slabs especially in the aisles of Very Narrow Aisle (VNA) high density warehouses where fixed path fork lift trucks operate. Variations in level across the aisle between the wheel tracks of the truck are magnified in proportion to the ratio of the height of the racking to the trackwidth of the truck. It is not uncommon for such vehicles to collide with high racking if one wheel is at a different level from the equivalent wheel at the other side. Warehouses are designed to achieve a throughput of a given number of pallets per hour and a poor floor may reduce the number of pallets handled. Also, those storage systems that have non-adjustable feet require particularly flat floors. Other situations where floor flatness is an important property include those trafficked by automatically controlled transportation equipment and the movement of heavy plant within a factory by hover transportation.

Floors are divided into those where movement is defined and those allowing free movement. Defined movement areas include the aisles in VNA warehouses, whereas free movement areas include warehouses with wide aisles or free running areas. Defined movement areas need to have enhanced flatness characteristics as compared with free movement areas. The criteria defined in Concrete Society publication TR34[5.2] are shown in Tables 5.9 and 5.10. The floor specifier should define the category of flatness required.

### *5.13.2 Flatness of laser screeded floors*

A floor with a very high degree of flatness and levelness can be produced using a laser screed. In most cases, a good Category 2 tolerance will be achieved. With careful supervision an experienced laser screed contractor will be able to achieve a Category 1 floor but, in some cases, it may be necessary to undertake some remedial grinding in the wheelpaths of lift trucks within racking aisles.

*Table 5.9. Flatness requirements for defined movement areas*

| Category | Location | Allowable limits: mm | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Property I | | Property II | | Property III | | | |
| | |   | |   | |   | | | |
| | | | | | | Wheeltrack up to 1·5 m | | Wheeltrack over 1·5 m | |
| | | A | B | A | B | A | B | A | B |
| Superflat (SF) | VNA warehouses with minimum clearance between fixed and moving pallets. Maximum throughputs, truck speed and permitted rack height | 0·75 | 1·00 | 1·00 | 1·50 | 1·50 | 2·50 | 2·00 | 3·00 |
| Category 1 | VNA warehouses with racking height between 8 and 13 m. Top guided trucks between 13 and 20 m | 1·50 | 2·50 | 2·50 | 3·50 | 2·50 | 3·50 | 3·00 | 4·50 |
| Category 2 | VNA warehouses with racking height less than 8 m | 2·50 | 4·00 | 3·25 | 5·00 | 3·50 | 5·00 | 4·00 | 6·00 |

Notes:

| | | |
|---|---|---|
| Tolerance of level to datum plane (in mm): | Category SF | ±10 |
| | Category 1 | ±10 |
| | Category 2 | ±15 |

The floor can be considered satisfactory when:

(*a*) not more than 5% of the total number of measurements exceed the particular property limit in column A, and
(*b*) none of the measurements exceed the particular property limit in column B.

*Table 5.10. Flatness requirements for free movement areas*

| Floor classification | Location | Maximum permissible limits: mm | | |
|---|---|---|---|---|
| | | Property II (as for Table 5.9) | Property IV: difference in elevation of points on a 3 m grid | |
| | | | A | B |
| FM1 | Hover transport and areas of special consideration | 2·5 | 3·0 | 4·5 |
| FM2 | Wide aisle warehouses, automatically guided transfer vehicles, transfer aisles | 3·5 | 5·5 | 8·0 |
| FM3 | Wide aisle warehousing using counter balanced trucks. Manufacturing facility, general warehousing for block stacking | 5·0 | 5·5 | 8·0 |

Notes:
The floor can be considered satisfactory when:
(*a*) not more than 5% of the total number of measurements exceed the particular property limit in column A, and
(*b*) none of the measurements exceed the particular property limit in column B or the property II column.

## 5.14 Engineering data for fibre reinforced concrete

The following test data have been obtained as the use of steel and polypropylene fibres has grown.

### 5.14.1 Data relating to steel fibre reinforced concrete

Several research programmes have been undertaken on slabs incorporating steel fibre reinforced concrete. Three are summarised below.

(*a*) Imperial College, London (Table 5.11)
- 12 slabs tested (1 m × 1 m × 50 mm)
- Simply supported on four edges and loaded in the centre
- Same basic concrete mixture
- Slabs contain varying quantity of hooked 60 mm long, 1·0 mm diameter fibres.

*Table 5.11. Summary of results of work undertaken at Imperial College, London*

| | | | | |
|---|---|---|---|---|
| Fibre dosage: kg/m$^3$ | 0 | 20 | 25 | 30 |
| First-peak stress: N/mm$^2$ | 4·56 | 4·66 | 5·39 | 5·40 |

*Table 5.12. Summary of results of work undertaken at Thames Polytechnic*

| Steel fibre type | None | Length 60 mm, diameter 1 mm | Length 60 mm, diameter 1 mm | Length 60 mm, diameter 0·8 mm | Length 60 mm, diameter 0·8 mm |
|---|---|---|---|---|---|
| Quantity: kg/m$^3$ | 0 | 20 | 30 | 20 | 30 |
| Load at first visible crack: t | 18·4 | 21·5 | 24·5 | 26·5 | 29·6 |
| Maximum load: t | 20·4 | 33·2 | 34·7 | 39·9 | 35·3[a] |

[a] In this first test, the capacity of the testing apparatus was not sufficient; for subsequent tests, the apparatus was modified.

(*b*) Thames Polytechnic, London (Table 5.12)
- 9 slabs tested (3 m × 3 m × 150 mm)
- $K = 0{\cdot}35\ \mathrm{N/mm^3}$
- Loaded until failure with point load in centre of slab
- Load at first visible crack also recorded.

(*c*) Newcastle University (Table 5.13)
A series of four point bending tests has been conducted on C30 concrete specimens of dimensions 150 × 150 × 450 mm with different steel fibre dosages as shown in Fig. 5.113. For each specimen, two load cycles were applied to produce stress/strain relationships similar to those illustrated in Fig. 5.114. During the first load cycle, each specimen attained a peak load when the concrete first cracked. Each specimen was then strained to an additional level by a lesser load. The load was removed and a second cycle was applied onto the cracked specimen. A second stress level was determined which represents the

*Table 5.13. Summary of flexural strength results from Newcastle University tests on steel fibre reinforced concrete*

| Fibre dosage: kg/m$^3$ | Peak load stress: N/mm$^2$ | Characteristic strength: N/mm$^2$ | Second load cycle stress: N/mm$^2$ |
|---|---|---|---|
| 0 | 2·2 | 1·5 | 0 |
| 5 | 2·5 | 1·8 | 0·3 |
| 10 | 2·7 | 2·0 | 0·7 |
| 15 | 2·9 | 2·2 | 1·0 |
| 20 | 3·2 | 2·5 | 1·3 |
| 25 | 3·4 | 2·7 | 1·7 |
| 30 | 3·7 | 3·0 | 1·9 |
| 35 | 3·9 | 3·2 | 2·2 |
| 40 | 4·1 | 3·4 | 2·6 |
| 45 | 4·4 | 3·7 | 2·9 |
| 50 | 4·6 | 3·9 | 3·2 |

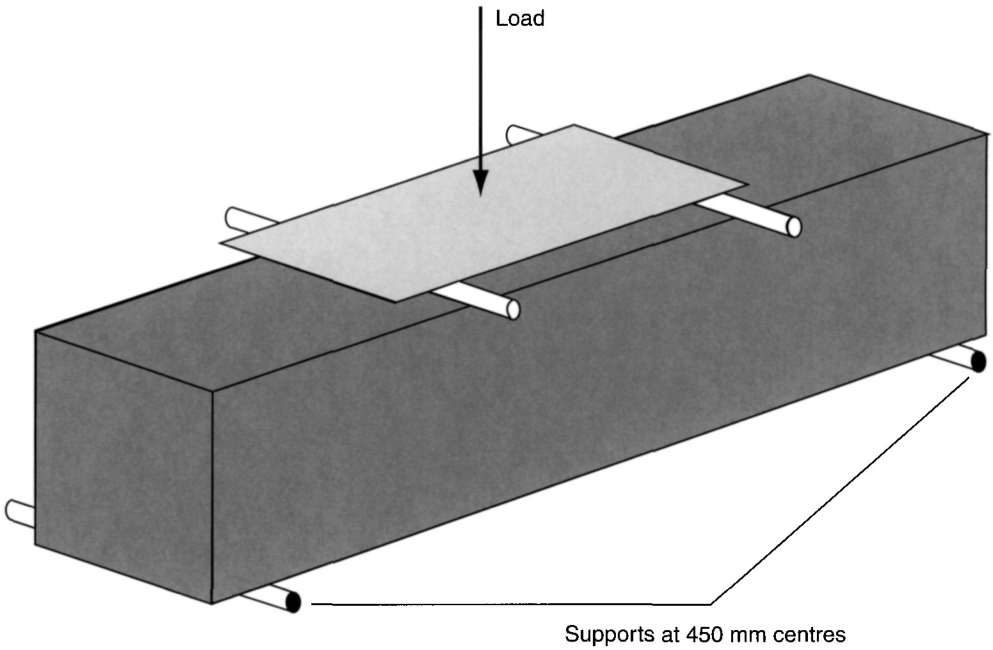



*Fig. 5.113. Four point bending test arrangement. The central third of the specimen is subjected to a constant bending moment with zero shear force*

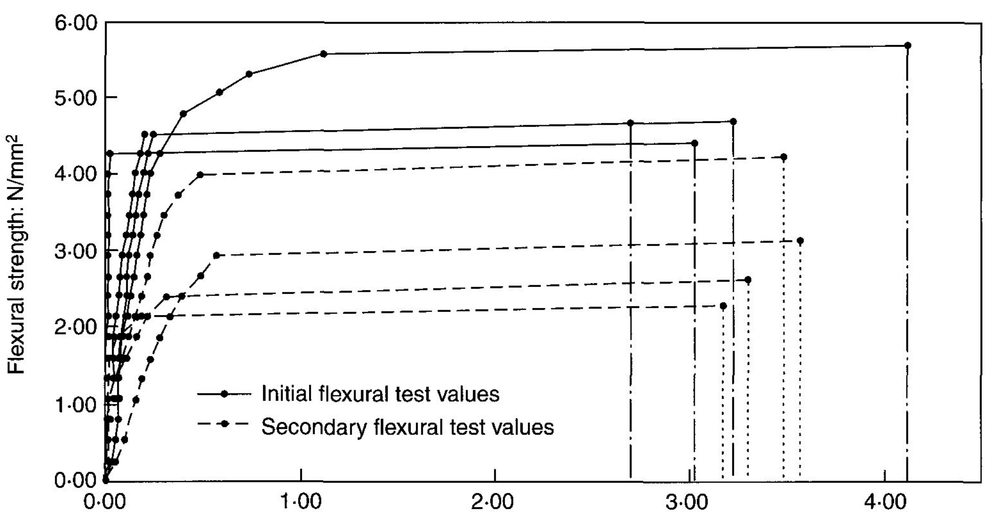



*Fig. 5.114. Typical stress/strain results from the Newcastle University tests on steel fibre reinforced concrete. These results were obtained with a dosage of 50 kg/m³*

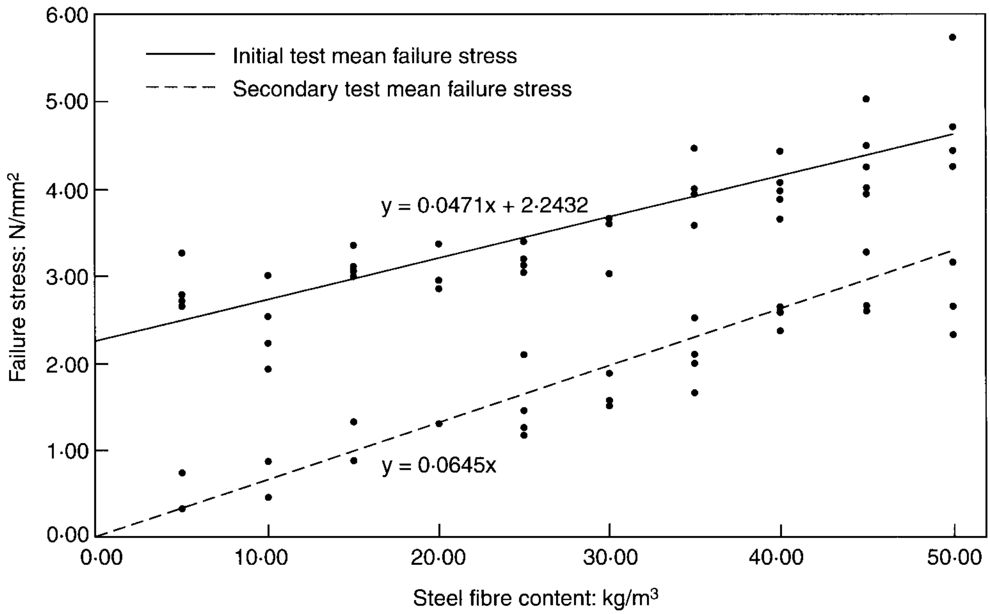



*Fig. 5.115. Summary of Newcastle University flexural strength test results. The upper line is for the first load application which causes the specimens to crack. The lower line applies to the lower load cycle when the concrete is in its post-crack condition*

effective post-crack flexural strength of the steel fibre reinforced concrete. In the second cycle, the forces were being transmitted by the fibres rather than through the concrete. Figure 5.115 shows the effective failure stresses for the first and second load cycles for 11 different fibre dosages (including zero fibres). Some authorities use the second cycle values in their design: in this book, the first cycle values are used to determine the corresponding characteristic flexural strengths. These values are shown in Table 5.13.

### 5.14.2 Data relating to polypropylene fibre reinforced concrete

The data that follow were obtained at Signet Laboratory and at San Jose State University using plain concrete and concrete with polypropylene fibres added. The concrete had a cement content of 240 kg/m$^3$, water/cement ratio of 0·64 and maximum aggregate size of 20 mm. The following results (Tables 5.14–5.16) were obtained.

*5.14.2.1 Rate of gain of compressive strength* Table 5.14 shows the rate of gain of compressive strength for both plain and polypropylene reinforced concrete. Fibre was added at the rate of 890 g/m$^3$.

*5.14.2.2 Effect of fibre dosage on compressive strength* Table 5.15 shows how three levels of fibre dosage influence compressive strength at ages of 7 and 28 days.

*Table 5.14.*

| Age of concrete | Compressive strength: N/mm$^2$ | |
|---|---|---|
| | Plain concrete | Polypropylene fibre reinforced concrete |
| 18 hours | 2·5 | 2·6 |
| 24 hours | 3·3 | 3·4 |
| 7 days | 9·8 | 10·4 |
| 28 days | 16·0 | 17·4 |

*Table 5.15.*

| Polypropylene fibre dosage: g/m$^3$ | Compressive strength: N/mm$^2$ | |
|---|---|---|
| | 7 day strength | 28 day strength |
| Zero | 14·2 | 27·3 |
| 600 | 15·9 | 29·7 |
| 1200 | 16·2 | 30·4 |

*Table 5.16.*

| Polypropylene fibre dosage: g/m$^3$ | Flexural strength: N/mm$^2$ | | Tensile strength: N/mm$^2$ | |
|---|---|---|---|---|
| | 7 days | 28 days | 7 days | 28 days |
| Zero | 1·8 | 2·5 | 1·2 | 1·9 |
| 890 | 2·0 | 2·7 | 1·4 | 2·0 |

*5.14.2.3 Effect of fibre dosage on flexural strength and tensile strength* Table 5.16 shows flexural strength values determined by bending beams of dimensions 150 × 150 × 450 mm and tensile strength values obtained by undertaking cyclinder splitting tests to ASTM C-490.

# 6 Specification of industrial hardstandings

## 6.1 Introduction

This chapter presents a set of model contract documents which can be used as a guide for a typical in-situ industrial hardstanding. It is assumed that a hardstanding needs to be upgraded so the existing concrete needs to be removed and a new reinforced concrete slab is to be installed. The documents comprise the following.

- Instructions for tendering.
- Brief description of works.
- Conditions of Contract.
- Specification.
- Preamble and Bill of Quantities.
- Form of Quotation.

Anyone wishing to receive the documentation as a Microsoft Office Word file should contact the author by e-mail (John.knapton@blueyonder.co.uk).

## 6.2 Instructions for tendering

TENDERS MUST BE SUBMITTED IN ACCORDANCE WITH THE FOLLOWING INSTRUCTIONS. TENDERS NOT COMPLYING WITH THESE INSTRUCTIONS MAY BE REJECTED BY THE EMPLOYER WHOSE DECISION IN THE MATTER SHALL BE FINAL.

1. The tender shall be made on the Form of Quotation incorporated in the tender documents. It shall be signed and submitted with the Bill of Quantities which shall be fully priced and totalled in ink. The quote shall be based on the Specification, Drawings and the Bill of Quantities.
2. The tender documentation shall be treated as private and confidential.
3. Tenders shall not be qualified and shall be submitted strictly in accordance with the tender documents. Only unqualified tenders shall be considered for acceptance. However, any alterations or qualifications which the tenderer wishes to be considered should be made in an accompanying letter with any financial consequences.

4. Tenders shall be submitted exclusive of VAT.
5. Tenderers are expected to visit the site and appraise the content of the work. Arrangements to visit site should be made through Mr B. Field who can be contacted by telephone on: 0161 911 5503.
6. Tenders shall be forwarded to the offices of John Knapton Consulting Engineers, City Road, Newcastle upon Tyne, no later than noon on 2 September 1999.
7. The Employer does not bind himself to accept the lowest or any tender.
8. The tenderer should note that the works will not commence on site until January 2000 and will be completed by the end of March 2000. The contract is a fixed price quotation and does not include any price fluctuation clause.

## 6.3 Brief description of works

THIS INFORMATION IS FOR TENDERING PURPOSES ONLY AND DOES NOT CONSTITUTE PART OF ANY CONTRACT.

The works comprise the following.

1. The excavation and removal of the existing concrete slabs and its removal from site.
2. The replacement of the in-situ concrete slabs including all associated transverse, induced, longitudinal and expansion joints and reinforcing materials.
3. Any damaged sub-base to the excavated slabs to be removed and replaced with DTp *Specification for Highway Works* Clause 803 Type 1 sub-base material and compacted by vibratory roller as specified in Clause 802 of the same document.

## 6.4 Conditions of Contract

This project shall be carried out according to the Institution of Civil Engineers' *Conditions of Contract for Minor Works*, 2nd Edition (March 1998) as approved by the Institution of Civil Engineers and the Association of Consulting Engineers.

### *6.4.1 Institution of Civil Engineers' Conditions of Contract for Minor Works*

### *6.4.2 Appendix to the Conditions of Contract*

This is to be written by each prospective tenderer and shall comprise the following.

1. Short description of the work to be carried out under the Contract under the heading:

   Replacement and Repair to Concrete Slabs.

2. Payment to be made under Article 2 of the Agreement in accordance with Clause 7 will be calculated on the following basis.
3. Where a Bill of Quantities or a Schedule of Rates is provided the method of measurement used is: (insert as required).

**PREAMBLE**

4. Name of the Engineer (Clause 2.1)
   JOHN KNAPTON
5. Starting date (if known) (Clause 4.2)
   2 January 2003
6. Period for completion (Clause 4.2)
   12 weeks
7. Liquidated damages (Clause 4.6)
   £600/day
8. Limit of liquidated damages (Clause 4.6)
   £15 000
9. Defects Correction Period (Clause 5.1)
   12 months
10. Rate of retention (Clause 7.3)
    5%
11. Limit of retention (Clause 7.3)
    £15 000
12. Minimum amount of interim certificate (Clause 7.3)
    £20 000
13. Bank whose base lending rate is to be used (Clause 7.8)
    Byker Peoples Credit Union
14. Insurance of the Works (Clause 10.1)
    Required — enter name of insurer
15. Minimum amount of third party insurance (persons and property)
    (Clause 10.6)
    £5 million
    Any one accident/number of accidents unlimited
16. Name of the Planning Supervisor (Clause 13(1)(b))
    Bernard Field
    19 City Road
    Newcastle upon Tyne, NE1
17. Name of the Principal Contractor (Clause 13(1)(b))
    Company's name to be inserted here

### *6.4.3 ICE Conditions of Contract for Minor Works*

### CONTRACT SCHEDULE

(List of documents forming part of the Contract)

- The Agreement
- The Contractor's Tender Price (excluding any general or printed terms contained or referred to therein unless expressly agreed in writing to be incorporated in the Contract)
- The Conditions of Contract

- The Appendix to the Conditions of Contract
- The Drawings. Reference numbers
- As listed on page XX of the Tender Document
- The Specification
- The priced Bill of Quantities
- The Schedule of Rates
- The Daywork Schedules

## 6.5 Specification

The specifications should include details of the following.

- General
- Traffic safety
- Cold weather workings
- Saw cutting
- Sub-base
- Concrete
- Reinforcement
- Curing
- Tolerances of concrete slabs
- Joint sealants and filler

### 6.5.1 *Specification — general*

**Noise**

1. The Contractor shall comply with the general requirements set out in BS 5228: Parts 1 and 2: 1984 *Noise control on construction and open sites.*

**Setting Out**

2. The Contractor's method of establishing setting out points must ensure that they are maintained and that replacement concrete bays comply with the agreed dimensions on the drawings.

**Dust and mud on highways**

3. The Contractor shall take all reasonable steps to minimise dust and/or mud nuisance during the construction of the works.
4. All public highways used by vehicles of the Contractor or any of his Sub-contractors or suppliers of materials or plant shall be kept clean and clear of all dust and mud dropped from those vehicles or their tyres. Similarly, all dust and mud from the works spreading on public highways shall be immediately cleared at the expense of the Contractor.
5. Clearance shall be effected immediately by manual sweeping and removal of debris, or, if so directed by the Engineer, by mechanical sweeping and cleaning equipment, and all dust, mud and other debris shall be removed entirely from the

road surface. Additionally, if so directed by the Engineer, the road surface shall be hosed or watered using suitable equipment.

6. Compliance with the foregoing will not relieve the Contractor of any responsibility for complying with the requirements of the Highway Authority in respect of keeping roads clean.

**Programme of works**

In accordance with the Conditions of Contract, the Contractor shall submit a firm programme of works as soon as practicable after the acceptance of his tender. In addition, the Contractor shall provide all subsequent revisions which may be required by the Engineer.

Consultations with the Supervising Officer and the Employer will be necessary to provide seven days notice of the Contractor's intention to enter any of the Employer's premises to enable the operations of the employer to be modified to accommodate the works programme.

**Permits**

7. The Supervising Officer will issue a permit to the Contractor's Agent who, after establishing his own identity, shall vouch for all employees of the Contractor under his charge of control. On commencement of the works, the Contractor's Agent shall ensure that he is in possession of the necessary permit at all times.
8. Security checks may be made from time to time and the Contractor's employees will be required to show proof of identity and of their need to be on the Employer's property to any responsible member of the Employer's staff.
9. The Contractor's Agent shall ensure the return of all permits when the need to be on the site ceases.

**Health and safety**

10. The Contractor, when carrying out the works, shall comply with all requirements of and the Health and Safety at Work Act 1994, so far as they affect personnel who were required to undertake the work on site.

The Contractor's attention is also drawn to the Construction (Design and Management) Regulations 1994, and shall provide all details required under these Regulations such as Health and Safety Policy, Risk Assessment and Training Certificates of personnel, etc. All requirements shall be provided prior to the works commencing and the Contractor shall comply with all requests of the Planning Supervisor.

**List of drawings**

11. The drawings which form part of the Contract are as follows:

| | |
|---|---|
| 99/100/1 | Existing |
| 99/100/2 | Proposed |
| 99/100/3 | Location Plan |

### 6.5.2 Traffic safety

1. The Contractor shall be responsible for the erection, maintenance and repositioning of appropriate traffic signs to ensure the safety of all vehicles, pedestrians and maintenance associated with the employer's operations. Where appropriate, all signing shall conform to the requirements of Chapter 8 of the *Traffic Signs Manual* published by HMSO. The Contractor shall ensure that all employees are familiar with Chapter 8 and shall observe the rules and regulations enforced within the Employer's Safety Book.
2. All traffic control proposals shall be approved by the Employer's Supervising Officer prior to commencement of the works.
3. All excavations shall be effectively protected from all road users, pedestrians and operations implemented by the client. The Contractor shall be responsible for maintenance of all guards and barriers during the contract period.
4. All personnel of the Contractor or his Sub-contractor must at all times wear approved reflective or fluorescent clothing.
5. All precautions shall be taken to ensure that all roads and hardstandings, both within and outside of the site, are kept clean of any spillage or debris occurring from the works. Any such spillage or debris shall be cleaned immediately.

### 6.5.3 Cold weather working

1. No hardstanding construction materials shall be incorporated in the works in a frozen condition.
2. Materials for use in the hardstanding shall not be laid on any surface which is frozen or covered with ice.
3. The temperature of concrete or cement-bound material in any pavement layer shall be not less than 5°C at the point of delivery. These materials shall not be laid when the air temperature in the shade falls below 3°C and laying shall not be resumed until the rising air temperature in the shade reaches 5°C.
4. If frost occurs during the first 20 days after placing the concrete slabs, one day shall be added to the period which would otherwise be required before opening to traffic for each night on which the temperature of the surface of the layer in question falls to 0°C or below.
5. For the use of hot applied sealants, the temperature of the pavement receiving the sealant shall be not less than 5°C.

### 6.5.4 Saw cutting

1. The excavation shall commence by saw cutting vertically around the perimeter of the excavation to the full depth of the concrete to be removed.
2. All excavated materials shall be removed, the bottom of the excavation swept of loose material and all dust arising from the saw cutting operation shall be blown out using compressed air.

*Table 6.1. Grading limits for DTp Type 1 sub-base material. Particle size shall be determined by the washing and sieving method of BS 812 Part 103*

| BS sieve size | Percentage by mass passing |
|---|---|
| 75 mm | 100 |
| 37·5 mm | 85–100 |
| 10 mm | 40–70 |
| 5 mm | 25–45 |
| 600 $\mu$m | 8–22 |
| 75 $\mu$m | 0–10 |

### *6.5.5 Sub-base*

**Granular sub-base material DTp Type 1**

1. DTp Type 1 granular sub-base material shall be crushed rock, crushed slag, crushed concrete or well burnt non-plastic shale. The material shall be well-graded, and lie within the grading envelope of Table 6.1.
2. The material passing the 425 $\mu$m BS sieve shall be non-plastic as defined by BS 1377: 1990, and tested in compliance therewith.
3. The material shall be transported, placed and compacted without drying out or segregating.
4. The material shall have a 10% fines value of 50 KN or more when tested in accordance with BS 812 except that samples shall be tested in a saturated and surface dried condition. Prior to testing, the selected test items shall be soaked in water at room temperature of 24 hours without previously having been oven dried.

**Compaction**

1. Compaction shall be completed as soon as possible after the material has been spread and in accordance with the requirements for individual materials.
2. Special care shall be taken to obtain full compaction in the vicinity of both longitudinal and transverse joints.
3. Compaction of unbound materials shall be carried out by a method specified in Table 6.2, unless the Contractor demonstrates at site trials that a state of compaction achieved by an alternative method is equivalent to, or better than, that using the specified method. The procedure for these trials shall be subject to approval by the Supervising Officer.
4. The surface of any layer of material shall, on completion of compaction and immediately before overlaying, be well-closed, free from movement under compaction plant and from ridges, cracks, loose material, pot holes, ruts or other defects. All loose, segregated or otherwise defective areas shall be removed to the full thickness of the layer and new material laid and compacted.
5. For the purpose of Table 6.2, vibrating-plate compactors are machines having a base-plate to which is attached a source of vibration consisting of one or two eccentrically-weighted shafts.

*Table 6.2. Compaction requirements for DTp granular material Type 1*

| Type of compaction plant | Category | Number of passes for layers not exceeding the following compacted thicknesses: mm | | |
|---|---|---|---|---|
| | | 110 | 150 | 225 |
| Smooth-wheeled roller (or vibratory roller operating without vibration) | Mass per metre width or roll: | 16 | Unsuitable | Unsuitable |
| | over 2700 kg up to 5400 kg | 8 | 16 | Unsuitable |
| | over 5400 kg | | | |
| Pneumatic-tyred roller | Mass per wheel: | | | |
| | over 4000 kg up to 6000 kg | 12 | Unsuitable | Unsuitable |
| | over 6000 kg up to 8000 kg | 12 | Unsuitable | Unsuitable |
| | over 800 kg up to 12 000 kg | 10 | 16 | Unsuitable |
| | over 12 000 kg | 8 | 12 | Unsuitable |
| Vibratory roller | Mass per metre width of vibrating roll: | | | |
| | over 700 kg up to 1300 kg | 16 | Unsuitable | Unsuitable |
| | over 1300 kg up to 1800 kg | 6 | 16 | Unsuitable |
| | over 1800 kg up to 2300 kg | 4 | 6 | 10 |
| | over 2300 kg up to 2900 kg | 3 | 5 | 9 |
| | over 2900 kg up to 3600 kg | 3 | 5 | 8 |
| | over 3600 kg up to 4300 kg | 2 | 4 | 7 |
| | over 4300 kg up to 5000 kg | 2 | 4 | 6 |
| | over 5000 kg | 2 | 3 | 5 |
| Vibrating-plate compactor | Mass per square metre of base plate: | | | |
| | over 1400 kg/m$^2$–1800 kg/m$^2$ | 8 | Unsuitable | Unsuitable |
| | over 1800 kg/m$^2$–2100 kg/m$^2$ | 5 | 8 | Unsuitable |
| | over 2100 kg/m$^2$ | 3 | 6 | 10 |
| Vibro-tamper | Mass: | | | |
| | over 50 kg up to 65 kg | 4 | 8 | Unsuitable |
| | over 65 kg up to 75 kg | 3 | 6 | 10 |
| | over 75 kg | 2 | 4 | 8 |
| Power rammer | Mass: | | | |
| | 100 kg up to 500 kg | 5 | 8 | Unsuitable |
| | over 500 kg | 5 | 8 | 12 |

(*a*) The mass per square metre of base-plate of a vibrating-plate compactor is calculated by dividing the total mass of the machine in its working condition by its area in contact with the material to be compacted.

(*b*) Vibrating-plate compactors shall be operated at the frequency of vibration recommended by the manufacturer. They shall normally be operated at travelling speeds of less than 1 km/h but if higher speeds are necessary, the number of passes shall be increased in proportion to the increase in speed of travel.

### 6.5.6 *Concrete*

**Pavement quality concrete**

1. All concrete in the slabs shall comprise C30 grade concrete to BS 5328, with a minimum of 320 kg/m$^3$ OPC cement and including polypropylene fibres to achieve frost resistance.
2. The water content shall be the minimum required to provide the required workability for full compaction of the concrete as determined by trial mixing or approved means, and the maximum free water/cement ratio shall be 0·5.
3. Aggregates for all pavement concrete shall be naturally occurring materials complying with BS 882.

   The nominal coarse aggregate size shall not exceed 40 mm.

   The chloride ion content of the aggregate to be used in concrete with embedded metal, determined in accordance with BS 812 shall satisfy the requirements given in Appendix C of BS 882.

   Fine aggregate containing more than 25% by mass of acid soluble material, as determined in BS 812, in either the fraction retained on or the fraction passing the 600 $\mu$m BS sieve, shall not be used in the top 50 mm of slabs.
4. The density of concrete grade C30 shall be such that the total air voids shall not be more than 8% for 20 mm aggregate or 7% for 40 mm aggregate.
5. Sampling and testing and compliance for the specified characteristic strength of concrete mixes during the works shall be in accordance with BS 5328, except that it shall be at the following rate of sampling and testing and with the following requirements.

   Concrete shall be supplied from a QSRMC Registered Plant. The Contractor shall supply details of the Ready Mix Concrete Supplier and Mix Design for approval and shall allow for trial mixes to be undertaken and testing to be carried to determine the suitability of the concrete.

   Three number 150 mm cubes shall be made and cured and tested in accordance with BS1881 from concrete delivered onto the site. At least one batch of cubes shall be made for 150 m$^3$ of concrete slab or for each day concrete is to be poured.

   One cube shall be tested at seven days and one cube at 28 days by a reputable company and the results shall be presented to the Supervising Officer for assessment.

   The remaining cube shall be retained until all results are available and be crushed if instructed to provide a comparison of strength.
6. The workability of the concrete at the point of placing shall enable the concrete to be fully compacted and finished without undue flow. The workability shall be determined by the slump test in accordance with BS 1881 for each concrete load delivered to site and shall be maintained at the optimum level within a tolerance in accordance with BS 5328.

**Placing and compaction**

1. The concrete shall be spread uniformly without segregation or varying degrees of precompaction, by conveyor, chute or by other means approved by the Supervising Officer. The concrete shall be struck off by a screed so that the average and differential surcharge is sufficient to ensure that after compaction the surface is to the required levels. The concrete shall be compacted by vibrating finishing beams. In addition, internal poker vibration shall be used for slabs thicker than 200 mm and may be used for lesser thicknesses. When used, the pokers shall be at points not more than 500 mm apart over the whole area of the slab, and adjacent to the side forms or the edge of a previously constructed slab.

   The surface shall be regulated and finished to the top of the forms or adjacent slab or pavement layer by using twin vibrating finishing beams. The beams shall be metal with a contact face at least 50 mm wide and a vibrating unit having a minimum centrifugal force of 4 kN, with the frequency recommended by the manufacturer or an equivalent compactive effort. The vibrating beams shall be moved forward at a steady speed of 0·5 m to 1 m per minute while vibrating over the compacted surface to produce a smooth finish.

   Joint grooves shall be constructed in compliance with Specification Clause 6.5.3. Any irregularities at wet formed joint grooves shall be rectified by means of a vibrating float at least 1·0 m wide drawn along the line of the joint. The whole area of the slab shall be regulated by two passes of a scraping straight edge not less than 1·8 m wide or by a further application of a twin vibrating finishing beam. Any excess concrete on top of the groove former shall be removed before the surface is textured.

   The concrete shall be placed and compacted within the time to completion given in Table 6.3.

   After the final regulation of the surface of the slab and before the application of the curing membrane, the surface of concrete slabs shall be brush textured in a direction at right angles to the longitudinal axis of the bay. The brushed surface

*Table 6.3. Maximum working times (in hours)*

| | Reinforced concrete slabs constructed in two layers without retarding | | All other concrete slabs | |
|---|---|---|---|---|
| Temperature of concrete at discharge | Mixing to finishing concrete | Between layers | Mixing to finishing concrete | Between layers in 2 layer work |
| Not more than 25°C | 2 | $\frac{1}{2}$ | 3 | $1\frac{1}{2}$ |
| Exceeding 25°C but not exceeding 30°C | 2 | $\frac{1}{2}$ | 2 | 1 |
| Exceeding 30°C | Unacceptable for paving | — | Unacceptable for paving | — |

texture shall be applied evenly across the slab in one direction by the use of a wire brush not less than 450 mm wide. The brush shall be made of 32 gauge tape wires grouped together in tufts spaced at 10 mm centres. The tufts shall contain an average of 14 wires and initially be 100 mm long. The brush shall have two rows of tufts. The rows shall be 20 mm apart and the tufts in one row shall be opposite the centre of the gap between the tufts in the other row. The brush shall be replaced when the shortest tufts wears down to 90 mm long. The minimum texture depth shall be an average of 0·75 mm with no measurement less than 0·65 mm.

## Joint grooves

*General*

1. Joint grooves shall be wet formed or sawn in the surface slabs to promote cracks at the required positions. They may be of any convenient width but their depth shall be as given in this Specification.
2. Transverse joint grooves which are initially constructed to less than the full width of the slab shall be completed by sawing through to the edge of the slab and across longitudinal joints as soon as any forms have been removed and before an induced crack develops at the joint.

## Sawn transverse joint groove

3. Sawing shall be undertaken as soon as possible after the concrete has hardened sufficiently to enable a sharp edge groove to be produced without disrupting the concrete and before random cracks develop in the slab. The grooves shall be of depth 50 mm and of width 6 mm.

## Wet-formed transverse joint grooves

4. Grooves shall be formed in the plastic concrete prior to the final regulation and finishing of the surface, either by vibrating a metal blade into the concrete to the required depth and inserting a proprietary groove former into the groove or the groove former may be vibrated vertically into the plastic concrete.
5. The disturbed concrete shall be fully recompacted around the former on each side and the surface regulated. If grooves to be formed are wider than 15 mm some of the disturbed concrete shall be removed unless it can be shown that the surface regularity can be achieved across each joint. The groove former shall then remain in the correct position, alignment and depth below the surface, until temporary or permanent sealing is carried out.

## Longitudinal construction joint grooves in surface slabs

6. The grooves shall be formed by fixing a groove forming strip along the upper edge of the slab already constructed before concreting the adjacent slab. Where the edge of the concrete is damaged it shall be ground or made good to the satisfaction of the Supervising Officer before fixing the groove forming strip. Alternatively, the subsequent slab may be placed adjacent to the first and a sealing groove sawn later in the hardened concrete to a depth of 25 mm.

**Slip membrane**

1. Slip membranes shall be impermeable plastic sheeting 125 $\mu$m thick laid flat without creases. Where an overlap of plastic sheets is necessary this shall be at least 300 mm. There shall be no standing water on or under the membrane when the concrete is placed upon it.
2. When necessary, the surface of the sub-base should be lightly blinded with fine material prior to placing the slip membrane.

### 6.5.7 Reinforcement

**Fabricated reinforcement**

*General*

1. Reinforcement shall comply with any of the following standards and be prefabricated sheets, or bars assembled on site, and shall be free from oil, dirt, loose rust or scale:

| | | |
|---|---|---|
| (*a*) | Steel fabric in flat sheets | BS 4483 |
| (*b*) | Hot rolled steel bars grade 250 | BS 4449 |
| (*c*) | Hot rolled steel bars grade 460 | BS 4449 |
| (*d*) | Cold worked steel bars | BS 4461 |

2. When deformed bars are used they shall conform to Type 2 bond classification of BS 4449 or BS 4461.
3. Laps in the longitudinal bars shall be not less than 35 bar diameters or 450 mm, whichever is the greater. At laps between prefabricated sheets the first transverse bar of one sheet shall lie within the last complete mesh of the previous sheet. There shall be a minimum of 1·2 m longitudinally between groups of transverse laps or laps in prefabricated reinforcement sheets.
4. Laps in any transverse reinforcement shall be a minimum of 300 mm. Where prefabricated reinforcement sheets are used and longitudinal and transverse laps would coincide, no lap is required in the transverse bars within the lap of longitudinal reinforcement. These transverse bars may be cropped or fabricated shorter so that the requirements for cover are met.
5. The reinforcement is to be positioned prior to concreting and shall be fixed on approved metal supports and retained in position at the required depth below the finished surface and distance from the edge of the slab so as to ensure that the required cover is achieved. Reinforcement assembled on site shall be tied, or firmly fixed, by a procedure agreed with the Supervising Officer, at sufficient intersections to provide the above rigidity.
6. The reinforcement shall be so placed that after compaction of the concrete the cover below the finished surface is 60$\pm$10 mm.

**Dowel bars**

1. Dowel bars shall be Grade 250 steel complying with BS 4449 and shall be free from oil, dirt, loose rust or scale. They shall be straight, free of burrs or other irregularities, and the sliding ends sawn or cropped cleanly with no protrusions outside the normal diameter of the bar. Dowel bars shall be 20 mm diameter at

300 mm spacing, 400 mm long for slabs up to 225 mm thick, and 25 mm diameter for slabs 225 mm thick or more.

2. Dowel bars shall be supported on cradles in prefabricated joint assemblies positioned prior to construction of the slab.
3. Dowel bars shall be positioned at mid-depth from the surface level of the slab, ±20 mm. They shall be aligned parallel to the finished surface of the centre line of the slab and to each other within the following tolerances.
   (*a*) For bars supported by cradles prior to construction of the slab:
      (*i*) all bars in a joint shall be within ±3 mm.
      (*ii*) two thirds of the bars shall be within ±2 mm
      (*iii*) no bar shall differ in the alignment from an adjoining bar by more than 3 mm in either the horizontal or vertical plane.
   (*b*) For all bars after construction of the slab:
      (*i*) twice tolerances for alignment as above
      (*ii*) equally positioned about the intended line of the joint with tolerances of ±25 mm.
4. Cradles supporting dowel bars shall not extend across the line of the joint.
5. The assembly of dowel bars and supporting cradles, including the joint filler board in the case of expansion joints shall have the following degree of rigidity when fixed in position.
   (*a*) For expansion joints deflection of the top edge of the filler board shall be no greater than 30 mm, when a load of 1·3 kN is applied perpendicularly to the vertical face of the joint filler board and distributed over a length of 600 mm by means of a bar or timber packing, at mid-depth and mid-way between individual fixings, or 300 mm from either end of any length of filler board, if a continuous fixing is used. The residual deflection after removal of the load shall not be more than 3 mm.
   (*b*) The joint assembly fixings to the sub-base shall not fail under a 1·3 kN load applied for testing the ridgity of the assembly but shall fail before the load reaches 2·6 kN.
   (*c*) The fixings for the contraction joints shall not fail under 1·3 kN load and shall fail before the load reaches 2·6kN when applied over a length of 600 mm by means of bar or timber packing placed as near to level of the line of fixings as practicable.
   (*d*) Failure of the fixings shall be deemed to be when there is displacement of the assemblies by more than 3 mm with any form of fixing, under the test load. The displacement shall be measured at the nearest part of the assembly to the centre of the bar or timber packing.
6. Dowel bars shall be covered by a thin plastic sheath for at least two-thirds of the length from one end for dowel bars in contraction joints or half the length plus 50 mm for expansion joints. The sheath shall be tough, durable and of an average thickness not greater than 1·25 mm. The sheathed bar shall comply with the following pull out test.

(*a*) Four bars shall be taken at random from stock and without any special preparation shall be covered by sheaths as required in this clause. The halves of the dowel bars which have been sheathed shall be cast centrally into concrete specimens $150 \times 150 \times 450$ mm, made of the same mix proportions to be used in the pavement, but with the maximum nominal aggregate size of 20 mm and cured in accordance with BS 1881. At seven days a tensile load shall be applied to achieve a movement of the bar of at least 0·25 mm.

7. For expansion joints, a closely fitted cap 100 mm long consisting of waterproofed cardboard or an approved synthetic material shall be placed over the sheathed end of each dowel bar. An expansion space equal in length to the thickness of the joint filler board shall be formed between the end of the cap and the end of the dowel bar.

### *6.5.8 Curing*

1. Immediately after the surface treatment is complete, the surface and exposed edges of the slabs shall be cured by the application of an approved resin-based aluminised reflecting curing compound.
2. Resin-based aluminised curing compound shall contain sufficient flake aluminium in finely divided dispersion to produce a complete coverage of the sprayed surface with a metallic finish. The compound shall become stable and impervious to evaporation of water from under the concrete surface within 60 minutes of application and shall have an efficiency index of 90%.
3. The curing compound shall not react chemically with the concrete to be cured and shall not crack, peel or disintegrate within three weeks after application.
4. Prior to application, the contents of any containers shall be thoroughly agitated. The curing compound shall be mechanically applied using a fine spray onto the surface at a rate of 10% above the recommended coverage rate.
5. The mechanical sprayer shall incorporate an efficient mechanical device for continuous agitation and mixing of the compound in its container during spraying.

### *6.5.9 Tolerances of concrete slabs*

1. The longitudinal regularity of the surface of the slab shall be laid to the stated levels to a tolerance of $\pm 6$ mm.
2. The surface shall also be placed to ensure that the tolerance of $\pm 3$ mm to the stated levels is achieved under a 3 m long straight edge.
3. At the edge of the repair and existing concrete pavement, the maximum allowable tolerance shall be 3 mm between the pavement surface and the underside of a 3 m long straight edge.
4. If any tolerances are exceeded the Contractor shall provide a method statement for the approval of the supervising officer to remedy the situation.

### 6.5.10 Joint sealants and filler

**Hot-applied sealants**

1. Hot-applied sealants shall comply with ASTM D3406, or D3569 (for fuel resistance).
2. Hot-applied sealants complying with BS 2499 shall be used only in a joint between concrete surface slabs and bituminous surfacing.

**Cold-applied sealants**

3. Cold-applied sealants shall be polysulphide-based sealants complying with BS 5212. The sealant shall consist of a polymer resin and a curing agent. In addition, a separate retarder may be used in accordance with the sealant manufacturer's instructions.
4. The seal shall be durable, elastometric material of low modulus without plasticity after curing for the manufacturer's recommended period. In addition to the manufacturer's certificate of compliance with BS 5212, a certificate shall be provided confirming that the 5 second reading on the Short Hardness Scale A, as measured by a meter in accordance with BS 2719, is less than 20° for a cured sample seven days after mixing. The difference between the Short Hardness measurement at seven days and the measurement carried out, shall be not more than 5°C.
5. For joints in kerbs and joints other than pavements, gunning grades of two-part polysulphide-based sealants complying with BS 4254 may be used. Alternatively, polyurethane-based sealing compounds may be used provided their performance is not inferior to BS 4254 material.

**Preformed compression seals**

6. Preformed compression seals made of polychloroprene elastomers complying with BS 2752 shall conform with the requirements of ASTM D2628. Seals of butadiene-acrylonitrile or other synthetic rubbers may be used if certificates are produced to show that they conform to the performance requirements of ASTM D2628 for oven ageing, oil and ozone resistance, low temperature stiffening and recovery.
7. Seals made of ethylene vinyl acretate in microcellular form and other synthetic materials may be used in longitudinal joints and in other structures if test certificates are produced to show the adequate resistance against ageing when testing in accordance with BS 4443: Part 4, Method 10 and Method 12 respectively. The compression set of any seal shall not be greater than 15% when the specimen is subjected to a 25% compression in accordance with BS 4443: Part 1, Method 6. When immersed in standard oils for 48 hours at 25°C in accordance with BS 903: Part A16, the volume change shall not be greater than 5%.
8. Compression seals shall be shaped so that they will remain compressed at all times and shall have a minimum of 20 mm contact face with the sides of the sealing groove. If lubricant-adhesive is used, it shall be compatible with the

*Table 6.4. Dimensions of applied joint seals*

| Spacing of contraction joints: m | Minimum width: mm | Minimum depth of seal: mm | | Depth of top of seal below the concrete surface: mm |
|---|---|---|---|---|
| | | Cold applied | Hot applied | |
| 15 and under | 13 | 13 | 15 | 5±2 |
| Over 15 to 20 | 20 | 15 | 20 | 5±2 |
| Over 20 to 25 | 25 | 20 | 25 | 5±2 |
| Over 25 | 30 | 20 | 25 | 7±2 |
| Expansion all | 30 | 20 | 25 | 7±2 |
| Gully/manhole slabs | 20 | 15 | 20 | 0±3 |

Note. The depth of seal is that part in contact with the vertical face of the joint groove. The depth of seal below the surface shall be taken at the centre of an applied seal relative to a short straight edge, 150 mm long, placed centrally across the joint.

seal and the concrete, and shall be resistant to abrasion, oxidation, fuels and salt.

**Dimensions of applied joint seals**

9. Joint seals shall be constructed in accordance with Table 6.4 for hot and cold applied sealants, a compressible caulking material, debonding strip or tape of a suitable size to fill the width of the sealing groove shall be firmly packed or stuck in the bottom of the sealing groove to provide the correct depth of seal as described in Table 6.4, with the top of the seal at the correct depth below the surface of the concrete.

**Joint filler board**

10. Joint filler board for expansion joints and manhole and gully slab joints shall have a thickness of 25 mm within a tolerance of ±1·5 mm and be of firm compressible material or a bonded combination of compressible and rigid materials of sufficient rigidity to resist deformation during the passage of the concrete paving plant. The depth of the joint filler board for manhole and gully slabs shall be the full depth of the slab less the depth of the sealing groove. In expansion joints, the filler board shall have a ridged top. Holes for dowel bars shall be accurately bored or punched out to form a sliding fit for the sheathed dowel bars.

## 6.6 Preamble and Bill of Quantities

PREAMBLE TO THE QUOTATION

THE TENDER SHALL BE MADE ON THE FORM OF QUOTATION ENCLOSED. IT SHALL BE SIGNED AND SUBMITTED WITH THE SCHEDULE OF WORKS, WHICH SHALL BE FULLY PRICED AND TOTALLED IN INK AND SHALL INCLUDE FOR THE FOLLOWING UNLESS STATED OTHERWISE

1. Labour and cost in connection therewith.
2. Supply of materials, goods, storage and costs in connection therewith, including delivery to site. Taking delivery of materials and goods supplied by others, unloading, storage and costs in connection therewith.
3. Disposal of unsuitable materials off site. Stockpiling and double handling of materials as necessary.
4. Plant and costs in connection therewith.
5. Fixing, erecting and installing or placing of materials in position, including forming of all joints.
6. Temporary works, including protective fencing and coning of operations.
7. The effect of phasing the works for alterations or additions to accommodate the operations of the client to the extent that such work is set forth or recently implied in the documents on which the tender is based.
8. General obligations, liabilities and risks involving the execution of works set forth or reasonably implied in the documents on which the tender is based.
9. Waste.
10. Supply and provide results for tests on materials and workmanship as specified in the tender documentation.
11. Complying with Quality Assurance standards.
12. Preparation and supply of detailed drawings showing position of all slabs and levels on completion of the work.
13. Compliance with all Health and Safety Requirements of the Construction (Design and Management) Regulations 1994.
14. Provision of automatic level, tripod and staff, and 3 m straight edge and wedge together with appropriate tapes for temporary use by the Supervising Engineer during the course of the works.

Tables 6.5–6.7 set out items to be included in the Bill of Quantities.

*Table 6.5.*

| No. | Description | Unit | Quantity | Rate | £ |
|---|---|---|---|---|---|
| | Preliminaries | | | | |
| 1. | Insurance of works | Item | | | |
| 2. | Implementation and maintenance of all phasing and protection of the works | Item | | | |
| 3. | Traffic control | Item | | | |
| 4. | Provision of all welfare facilities | Item | | | |
| | TOTAL TO QUOTATION | | | | |

*Table 6.6.*

| No. | Description | Unit | Quantity | Rate | £ |
|---|---|---|---|---|---|
| | Reference is made to Drawing Nos 156/03/04 and 156/03/05 | | | | |
| 1. | Excavate and dispose off site of existing concrete slabs and sand bed | $m^3$ | 1400 | | |
| 2. | Full depth saw cut through existing concrete slab, approximate depth 225 mm, and clean vertical face | m | 62 | | |
| 3. | Full depth saw cut existing salvacim pavement, approximate depth 225 mm, and clean vertical face | m | 62 | | |
| 4. | Reinstate existing sub-base, and place and compact regulating Type 1 sub-base to level | $m^2$ | 4242 | | |
| 5. | Provide and place 225 mm thick pavement concrete on separation membrane, including all joints as shown on the drawings and in accordance with the Specification | $m^2$ | 4242 | | |
| | TOTAL TO QUOTATION | | | | |

*Table 6.7.*

| No. | Description | Unit | Quantity | Rate | £ |
|---|---|---|---|---|---|
| | Dayworks — provisional | | | | |
| 1. | General labourer | Hour | 20 | | |
| 2. | Driver/machine operator | Hour | 20 | | |
| 3. | Concrete finisher | Hour | 20 | | |
| 4. | Steel fixer | Hour | 20 | | |
| 5. | Materials at cost +12·5% | Sum | — | — | 1000 |
| 6. | Percentage adjustments to materials | £ | 1000 | % | |
| 7. | Plant in accordance with the Daywork Schedule issued by the FCEC | Sum | — | — | 1000 |
| 8. | Percentage adjustment to plant | £ | 1000 | % | |
| | TOTAL TO QUOTATION | | | | |

## 6.7 Form of Quotation

John Knapton Consulting Engineers
City Road
Newcastle upon Tyne

Dear Sir,

Quotation for: Replacement and Repair to concrete slabs

Having examined the drawings, Conditions of Contract and Appendix, Schedule of Works and Specification for the above-mentioned Work. We offer to carry out and maintain the whole of the said Works in conformity with the said documents.

For the sum of
Preliminaries
Main works
Dayworks
Sub-total
Add 5% contingencies

TENDER TOTAL £ ________________

For such sum as shall be ascertained in accordance with the Conditions of Contract and Bill of Quantities returned herewith.

Yours faithfully,

Signature ......................................

Firm ..........................................

Address: ......................................

Tel. No: ......................................

Date: .........................................

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