Fourth Edition

Technical Report 34

CONCRETE INDUSTRIAL GROUND FLOORS

A guide to design and construction

The Concrete Society
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Acknowledgements

This revised guidance document was prepared by a Concrete Society Project Steering Committee and Design sub-group, consisting of:

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Contents

Preface

This is the fourth edition of Concrete Society Technical Report 34 *Concrete industrial ground floors*. TR34 is recognised globally as a leading publication giving guidance on many of the key aspects of concrete industrial ground floors.

Guidance on the design and construction of ground-supported concrete floors was originally developed and published by the Cement and Concrete Association in the 1970s and 1980s. The first edition of Technical Report 34 was published in 1988 and took account of the rapid development of new construction techniques and gave guidance on thickness design. The second (1994) edition[1] and third edition (2003)[2] continued to update this guidance to reflect current knowledge and practice.

As with previous editions, this fourth edition is the result of a thorough review of all aspects of floor design and construction. Experience since 2003 suggests that ground-supported floors constructed in accordance with TR34 have provided good performance. This experience has been based largely on steel fabric floors with sawn joints and on 'jointless' steel-fibre-reinforced ground-supported floors.

Significantly, the design guidance in this edition has been expanded to include comprehensive guidance on the design of pile-supported floors.

The Society acknowledges the support and assistance of its members and of the concrete flooring industry who have contributed to the preparation of this report, and also the help and comments provided by many individuals and companies, both in the UK and overseas.

Glossary of terms and abbreviations

Key terms and abbreviations are defined below. A list of the symbols and units used in the report follow.

Abrasion – Wearing of the concrete surface by rubbing, rolling, sliding, cutting or impact forces.

Abrasion resistance – The ability of the floor surface to withstand the abrasion produced by long-term use of the floor.

Aggregate interlock – Mechanism that transfers load across a crack in concrete by means of interlocking between irregular aggregate and cement paste surfaces on each side of the crack.

Armoured joint – Steel protection to joint arrises.

Bay – Area of concrete defined by formwork.

Block stacking – Unit loads, typically pallet loads, paper reels or similar goods, stacked directly on a floor, usually one on top of another.

Client – The party who commissions the building and employs a principal contractor to build it.

Contraction/expansion – Change of length caused by shrinkage, temperature variation etc.

Crazing – Pattern of fine, shallow random cracks on the surface of concrete.

Curing – Procedure to significantly reduce the early loss of moisture from the slab surface.

Curling – The tendency of slab edges to lift, caused by differential drying shrinkage with depth.

Datum – A reference point taken for surveying.

Defect – A feature causing obvious serviceability or structural issues that directly prevents safe and efficient use of the floor.

Defined-movement area – Narrow aisles in warehouses where materials handling equipment is move only in defined paths.

Deflection – Elastic or creep deformation of the slab or its support under loading.

Delamination – Debonding of a thin layer of surface concrete.

Dominant joint – A joint that opens wider than adjacent (typically dormant) joints in a sawn-jointed floor.

Dormant joint – Sawn joint that does not open, usually because of failure of crack to form below the saw cut; generally associated with a dominant joint.

Dowel – Round or square steel bar or plate device used to transfer shear loads across a joint between a slab, bay or panel and to prevent differential vertical movement.

Dry-shake topping – A mixture of cement and fine hard aggregate, sometimes with admixtures and pigment, applied as a dry thin layer that is trowelled into the fresh concrete.

End-user – The party who uses the building and floor in service. The user may not be the client or the owner.

Expansion – See contraction.

Flatness – Surface regularity over short distances.

Floor – The complete structure, consisting of several slabs.

Floor contractor – The contractor or subcontractor responsible for the construction of the floor.

Floor designer – The party responsible for the structural design and detailing of the floor.

Formed joint – Joint formed by formwork.

Free-movement area – Floor area where materials handling equipment can move freely in any direction.

Free-movement joint – Joint designed to provide a minimum of restraint to horizontal movements caused by drying shrinkage and temperature changes in a slab, while restricting relative vertical movement.

Ground-supported floor – Floor supported on original or improved ground, where universal uniform support from the ground is assumed.

Isolation joint – Joint detail designed to avoid any restraint to a slab by fixed elements such as columns, walls, bases or pits, at the edge of or within the slab.

Joint – Vertical discontinuity provided in a floor slab to allow for construction and/or relief of strains. The terminology relating to the various types of joint is complex, and reference may be made to the definitions of individual joint types.

Jointless floor – Floor constructed in large panels without intermediate joints.

Large-area construction – Area of floor of several thousand square metres laid in a continuous operation.

Levelness – Surface regularity over a longer distance, typically 3m, and to datum.

Line loads – Loads acting uniformly over extended length.

Load-transfer capacity – The load-carrying capacity of joints in shear.

Mezzanine – Raised area, e.g. for offices; typically a steel frame on baseplates supported off the floor.

MHE – Materials handling equipment.

Modulus of subgrade reaction – Measure of the stiffness of the subgrade; load per unit area causing unit deflection, expressed as '*k*'

Overlay – Concrete layer constructed on, and commonly debonded from, a hardened concrete base slab to provide a wearing surface.

Owner – The party who owns the building in service. The owner may not be the client.

Panel – Smallest unit of a floor slab bounded by joints.

Pile head – Structure provided at the top of a single pile, cast separately or integrally, immediately below the slab to act as the bearing surface between the pile and slab.

Pile-supported slab – Floor constructed on, and supported by, piles; used where ground-bearing conditions are inadequate for a ground-supported floor.

Point load – Concentrated load from a baseplate or wheel.

Pour – An area of slab constructed in one continuous operation.

Power finishing – Use of machinery for floating and trowelling floors.

Principal contractor – The contractor employed by the client to construct the building.

Property – Term used for defining floor regularity; elevational differences or measurements derived from elevational differences that are limited for each class of floor.

Racking – Systems of frames and beams for storage, usually of pallets.

Racking upright loads – Loads imposed upon the floor surface from the uprights of loaded racking.

Remedial grinding – The process of removing areas of a floor surface by abrasive grinding of the hardened concrete, usually in order to achieve the required surface regularity.

Restrained-movement joint – Joint designed to allow limited movement to relieve shrinkage-induced stresses in a slab at predetermined positions.

Sawn joint – Joint in the bay where a crack is induced beneath a saw cut.

Scheme designer – The designer employed by client or principal contractor who is responsible for the overall design and specification of the building and floor.

Settlement – Non-reversible deformation of the slab, due to long-term deformation of supporting ground.

Shrinkage – Shortening of length caused by drying.

Slab – Structural concrete element finished to provide the wearing surface of a floor; can also be overlaid by screeds or other layers.

Slip membrane – Plastic sheet laid on the sub-base before concrete is placed, to reduce the friction between slab and sub-base. Note: other forms of membrane are used for other requirements, e.g. gas membranes.

Slip resistance – The ability of a floor surface to resist slippage.

Sub-base – Layer (or layers) of materials on top of the subgrade to form a working platform on which the slab is constructed.

Subgrade – The upper strata of the soil under a ground floor.

Surface regularity – Generic term to describe the departure of a floor profile from a theoretical perfect plane.

Tied joint – Joint in a slab provided to facilitate a break in construction at a point other than a free-movement joint.

Tolerance – Allowable variation from intended value or plane.

Uniformly distributed load – Load acting uniformly over relatively large area.

User – See end-user.

VNA – Very Narrow Aisle; aisle between racking where the MHE always runs in a defined path.

Wearing surface – The top surface of a concrete slab or applied coating on which the traffic runs.

Wide aisle – Aisle between racking or areas of block stacking where the MHE does not move in a defined path, but can move in any direction.

Units and symbols

A	effective contact plan area for fan yield line mechanism
A_p	cross-sectional area of plate
A_s	cross-sectional area of reinforcement
A_v	shear area
a	radius of contact area
b	width or effective diameter of pile head
d	effective depth of cross-section
E	distance of application of load from face of concrete
E_{cm}	secant modulus of elasticity of concrete
E_s	modulus of elasticity for reinforcing steel
F	reduction factor
F_R	applied load at stage R of beam test (EN 14651)
f_{cd}	design value for concrete cylinder compressive strength
f_{ck}	characteristic cylinder compressive strength of concrete at 28 days
f_{cm}	mean value of concrete cylinder compressive strength; also $f_{ck,cyl}$
f_{ctd}	design value of axial tensile strength of concrete
$f_{ctk,fl}$	characteristic flexural strength of concrete
f_{ctm}	mean value of axial tensile strength of concrete
f_{cu}	characteristic compressive concrete cube strength at 28 days; also $f_{ck,cube}$
f_R	residual flexural strength of beam test (EN 14651)
f_{R1}	residual flexural strength at point 1 in beam test (EN 14651)
$f_{r(n)}$	mean axial tensile strength at point n
f_{sy}	yield strength of fibre reinforcement
f_{yk}	yield strength of reinforcement
h	design slab thickness
h_c	crack height
h_{sp}	depth of section to tip of crack
h_{ux}	depth of section to neutral axis
k	modulus of subgrade reaction
k_s	coefficient or factor
k_1	coefficient or factor
k_2	coefficient or factor
k_3	coefficient or factor
L	span centre-to-centre of pile support
L_{eff}	effective pile span
l	radius of relative stiffness
$M_{fl,r}$	residual moment capacity of fibre-reinforced section
M_n	ultimate negative (hogging) resistance moment of the slab
M_p	ultimate positive (sagging) resistance moment of the slab
M_{pfab}	moment capacity of fabric-reinforced section
M_{pfib}	moment capacity of fibre-reinforced section
M_u	ultimate moment capacity
$P_{lin,n}$	ultimate line load capacity controlled by negative bending moment
$P_{lin,p}$	ultimate line load capacity controlled by positive bending moment
P_p	slab load capacity
P_{sh}	shear capacity of dowel
P_u	ultimate capacity under concentrated load
p_b	dowel plate width
Q_l	imposed line load
q	load per unit area
q_ℓ	line load
q_{sw}	uniformly distributed deal load
q_u	uniformly distributed loading including self-weight
R	distance from centre of point load to centre of nearest pile
R_{cp}	sum of ground pressures within critical perimeter
R_{fan}	radius of fan mechanism
R_g	resistance of ground to punching
t_p	dowel plate depth
u_0, u_1	length of critical punching perimeter
v_{max}	strength factor for concrete cracked in shear
$v_{Rd,c,min}$	minimum shear resistance of concrete
α	expression related to dowel punching shear

Δc_{dev}	allowance for deviation from minimum cover to reinforcement
γ_F	partial safety factor for loads
γ_m	partial safety factor for materials
ε_{fc}	compressive strain in concrete
ε_{ft}	tensile strain in concrete
ε_s	strain in steel
λ	factor determined from Equation 33
ν	Poisson's ratio
ρ	reinforcement ratio
$\sigma_{r(n)}$	mean axial tensile strength derived from beam test (EN 14651)
ϕ	dynamic modification factor

Greek letters

alpha	α	Α	nu	ν	Ν
beta	β	Β	xi	ξ	Ξ
gamma	γ	Γ	omicron	ο	Ο
delta	δ	Δ	pi	π	Π
epsilon	ε	Ε	rho	ρ	Ρ
zeta	ζ	Ζ	sigma	σ	Σ
etaη	Η		tau	τ	Τ
theta	θ	Θ	upsilon	υ	Υ
iota	ι	Ι	phi	φ	Φ
kappa	κ	Κ	chi	χ	Χ
lambda	λ	Λ	psi	ψ	Ψ
mu	μ	Μ	omega	ω	Ω

The following units are used for calculations:

forces and loads	kN, kN/m, kN/m^2
moments (bending)	kNm/m
modulus of subgrade reaction	N/mm^2/mm
stresses and strengths	N/mm^2
unit mass	kg/m^3
unit weight	kN/m^3
unit length	mm, m
unit area	mm^2, m^2

1 Introduction

A warehouse or industrial facility should be considered as a single interconnected system. Optimal performance can only be expected if the racking, materials handling equipment (MHE) and the floor are designed and operated to common tolerances and requirements. This report provides guidance on the design and construction of industrial floors to meet these demands.

1.1 Scope

The guidance relates to internal concrete floors that are fully supported by the ground or supported on piles that are primarily found in industrial warehousing (both ambient and temperature controlled) and retail applications.

Figures 1.1 to 1.4 show some typical floors.

The report is not intended for use in the design or construction of external paving, docks and harbour container parks or for conventional elevated suspended floors in buildings.

Figure 1.1: Low-level operation with mezzanine to the right.

1.2 Changes in fourth edition

1.2.1 Floor surface regularity

Since the third edition of TR34[2] the European Standard EN 15620[3] has been published providing recommendations for the surface regularity of floors on which racking is situated. The section on surface regularity in this edition has been revised to reflect the key aspects of this European Standard.

1.2.2 Design

This edition includes comprehensive guidance on the material properties and methods of analysis and design for both ground-supported and pile-supported floors.

The post-cracking properties of fibre-reinforced concrete are now determined from the European Notched Beam Test described in EN 14651[4].

This edition includes guidance on the design of floors subjected to repeated trafficking associated with heavy counterbalance trucks in applications such as in paper handling facilities or in heavy engineering.

1.2.3 Maintenance

This edition now includes more comprehensive guidance on regular inspection and maintenance.

Figure 1.2: A reach truck between wide aisle racking.

1.3 Design and specification

The performance of a floor depends on the design, specification and the techniques used in its construction. Scheduled regular inspection and maintenance is necessary to retain in-service performance.

Successfully constructed floors are a result of an integrated and detailed planning process that focuses on the current and potential future use of the floor. The use of a design brief from the start of the planning process is strongly recommended, resulting in a comprehensive specification.

The requirements for concrete industrial ground floors include the following:

- The floor should remain serviceable, assuming planned maintenance and no gross misuse or overloading.
- The floor must be able to carry the required static point loads, uniformly distributed loads and dynamic loads, without unacceptable deflection, cracking, settlement or damage to joints.
- Joint layouts should take into account the location of racking uprights or mezzanine floor columns.
- Joints should be robust in both design and construction.
- Joints and reinforcement should be detailed to minimise the risk of cracking.
- The floor surface should have suitable surface regularity.
- The floor surface should have suitable abrasion, chemical and slip resistance.
- The floor should have the required type of finish.

A model design brief is given in Appendix A, which can be adapted to suit the requirements of each project.

There may be additional factors to be taken into consideration. For example, slabs for waste transfer facilities will be subjected to high wear from the mechanical damage associated with front-loader buckets. Joints and drain-lines are particularly vulnerable. Similar problems are experienced with floors for bio-stores and composting facilities. In addition there is a risk of gradual surface deterioration from the liquor produced from the composting. Temperature generated from the composting process can also be high, causing differential movements.

Figure 1.3: A transfer aisle in a large distribution warehouse.

Figure 1.4: Concrete floor before occupation.

2 Floor surfaces

This section is intended to help provide an understanding of what can be expected of floor surfaces and to evaluate the significance of particular features that may be observed on a completed floor.

Wherever practical, specifications should give specific criteria to be achieved, but it is recognised that some floor characteristics are not easily defined and their descriptions can be open to interpretation.

Requirements relating to surface regularity are discussed separately in Section 3.

2.1 Abrasion resistance

Abrasion resistance is the ability of a concrete surface to resist wear caused by rubbing, rolling, sliding, cutting and impact forces. Wear, which is the removal of surface material, is a process of displacement and detachment of particles or fragments from the surface. Abrasion mechanisms are complex and combinations of different actions can occur in many environments – for example, from truck tyres, foot traffic, scraping and impact. Excessive and early wear can be caused by the use of under-specified or non-compliant concrete or water damage at the construction stage.

In normal warehouse working conditions, poor abrasion resistance is rarely a problem for a typical power-trowelled and well-cured floor using good quality concrete. Lower concrete strength classes may require a dry-shake topping to achieve adequate abrasion resistance.

A test to measure the abrasion resistance of a floor surface is described in EN 13892-4[5]. The minimum age of test is not noted but the concrete must have developed its required strength, i.e. a minimum of 28 days is considered sensible. It is suggested that a sampling rate of 1 test per 4000m^2 is adequate. The maximum limit of abrasion should be 0.20mm.

If a floor is to be tested, it should be noted that resin-based curing compounds create a layer or 'skin' on the surface that can be impenetrable to the abrasion test machine[6].

Inadequate abrasion resistance in service can be improved by surface-penetrating resin sealers and/or grinding.

2.2 Chemical resistance

Chemical attack on concrete floors usually arises from the spillage of aggressive chemicals. The intensity of attack depends on a number of factors, principally the composition and concentration of the aggressive agent, its pH, the permeability of the concrete and the contact time.

Examples of common substances that may come into contact with concrete floors are acids, wines, beers, milk, sugars, and mineral and vegetable oils. Commonly encountered materials that are harmful to concrete are listed in Appendix B and a more comprehensive listing is given in a Portland Cement Association guide[7].

Any agent that attacks concrete will eventually cause surface damage if it remains in contact with the floor for long enough. Although frequent cleaning to remove aggressive agents will reduce deterioration, repeated cycles of spillage and cleaning will cause long-term surface damage – see Section 9.6

Where chemical attack is likely, consideration should be given to protecting the floor with a chemically resistant treatment.

2.3 Slip resistance

The commonly used process of power trowelling, which produces good abrasion resistance, also tends to produce smooth floors. The slip potential of a power-trowelled floor surface depends on several factors: the footwear worn by people, the tyres on the MHE and the presence of surface contaminants such as dusts, coatings and liquids. In many industrial situations, contaminants may be the most important factor. The scheme designer should therefore establish at an early stage what contaminants are likely to be present during the normal operation of the premises, as this may dictate the floor finish required and the cleaning regime.

Where slip resistance is of importance, consideration may be given to further surface treatment such as shot blasting, acid etching, surface grinding or the application of resin-bound aggregate finishes. This latter method is particularly useful in areas adjacent to entrances where floors can become wetted by rain or water from incoming vehicles but it should be noted the abrasion resistance will be reduced with these treatments and periodic reapplication may be required.

For further information see CIRIA C652 *Safer Surfaces To Walk On* [8].

2.4 Colour and appearance

Concrete floors are constructed primarily from naturally occurring materials and finished by techniques that cannot be controlled as precisely as would be expected in a factory production process. Good materials and workmanship may reduce variations in colour and appearance, but they will not eliminate them and the final appearance of a floor will never be as uniform as an applied coating. Some features of concrete floors that are visible in the first few weeks after it has been cast relate to the early drying of the floor and become less visible with time.

Trowel marks and discoloration caused by the finishing processes are related to normal variations in concrete setting, the visual impact of which will usually reduce significantly with time.

Excess curing compound or overlapping layers of curing compound cause darker areas. These wear and disappear with time without adverse effect on the surface.

Some floors are constructed with a 'dry-shake topping' as a monolithic thin layer – see Section 10.4. These sometimes include pigments to give colour to the finished surface and, if a light-coloured dry-shake topping is used, improved light reflectivity, see Figure 2.1. These do not give the uniformity or intensity of colour of a painted finish or applied coating and the same appearance considerations apply to these finishes as to ordinary concrete. Floor users are recommended to inspect in-use existing floors to evaluate the benefits of such finishes and the effects that can be achieved.

Concrete incorporating a through-colour pigment may be used, but variations in colour can be expected.

For bold and consistent colour it is necessary to use a surface coating. Coatings will degrade depending on the type and usage and therefore may need replacement during the life of the building.

Grinding can be used to improve surface regularity or to remedy light surface damage. This will not usually affect the use of the floor but will affect its appearance. It will wholly or partially remove any surface treatment such as a dry-shake topping.

Figure 2.1: A retail store floor with dry-shake topping.

2.5 Cracking

There is a risk of cracking in all concrete floors. This risk increases with the size of the bays and distance between stress relief joints. There is a greater risk of cracks in jointless ground-supported floors than in jointed ground-supported floors. Pile-supported floors, constructed in large areas with fewer stress relief joints also have a higher risk of cracking, see Section 8.1.

When considering jointed or jointless construction for ground-supported floors, the potential reduction in joint maintenance costs of jointless floors should be weighed against the greater risk of cracking and associated repair costs. See Section 13.5.

Cracking is often associated with the restraint to shrinkage, fine cracks generally having no structural significance. Less commonly, cracks can occur because of overloading or structural inadequacy, and some restraint-induced cracks could have structural implications because of their position in relation to applied loads.

The earlier the loads are applied, the greater the risk of cracking due to restraint to shrinkage and/or load-induced stresses. Loading at an early age will cause pinning of the slab to the sub-base. This can be mitigated by consideration of the following:

- Racking should remain unloaded for as long a period as possible.
- Loads should be partial and evenly spread.
- Loading should follow the floor bay construction sequence, oldest first.
- Phased loading from the centre of the bays outwards is advisable.
- Avoidance of bolting any base-plates that straddle joints.

For treatment of cracks refer to Section 13.5

2.6 Crazing

Many power-trowelled concrete floors exhibit an irregular pattern of fine cracks. This is known as surface crazing. It is an inherent feature of power-trowelled concrete surfaces and is considered to be a matter of appearance only, and not a structural or serviceability issue. It should not be confused with in-panel cracking due to restrained contraction. It tends to be more visible on floors that are wetted and cleaned, as the extremely fine cracks trap moisture and dust. Crazing can equally occur in floors with a dry-shake topping or through-colour pigment.

The mechanisms of crazing in floors are not fully understood so it is not possible to recommend measures that can reduce its occurrence. It is known that the surface zone consists predominantly of mortar paste which in power-finished floors is intensively compacted by the trowelling process and can have a very low water/cement (w/c) ratio compared to the underlying concrete. Crazing is a result of differential contraction, probably caused by drying shrinkage and carbonation shrinkage, between the surface layer and the underlying concrete. Keeping the w/c ratio of the specified concrete as low as practicably possible should reduce this differential and therefore the intensity of crazing.

There is no appropriate treatment for crazing and so if this feature is unacceptable to the end-user, provision should be made at planning stage for a surface coating, but this will incur ongoing maintenance costs. For additional information, see Concrete Advice Sheet 08 *Crazing* [9].

2.7 Curling

Curling is caused by the differential shrinkage of the concrete. The exposed top surface dries and shrinks more than the bottom, causing the floor to curl upwards.

Curling can occur at any time up to about 2 years after construction, cannot be totally eliminated and tends to be unpredictable. Curling may occur at joints and edges of slabs and may result in cracking. Floor bays sometimes curl to such an extent that truck performance is affected. Where necessary, departures from the required surface regularity can be corrected by grinding. Section 3 provides detailed guidance on surface regularity.

Curling can cause the loss of sub-base support, causing the floor to move under the passage of trucks. This movement can be a major contributor to joint arris breakdown, particularly where there is weak or non-existent load transfer across the joint. Movement should be monitored as part of the maintenance regime and dealt with as required. Under-slab grouting can restore support but load transfer across the joint should also be restored.

Particular care should be taken at personnel doors as a curled slab can introduce a trip hazard and this should be taken into account during the design process by the use of dowels and sleeves to maintain load transfer and minimise vertical movement. For additional information see, Concrete Advice Sheet 44 *Curling Of Ground Floor Slabs* [10].

2.8 Delamination

Delamination is the process whereby a thin (typically 2–4mm) layer becomes detached from the surface. It is primarily caused by the entrapment of air and/or bleed water beneath the surface of the concrete during finishing operations.

It is believed that there is a strong link between bleed water and air within the concrete, as the air uses the fine bleed channels to escape. If closing of the surface prevents bleed water from escaping, the air can accumulate causing a weak plane and, potentially, delamination.

Several factors affect the occurrence of delamination including:

Differential setting of the surface
Accelerated drying of the surface by cross winds or high ambient temperatures can significantly increase the chances of delamination as the surface is prematurely sealed.

Air content
Entrained air generated from admixtures should be minimised by careful selection of the admixture. Entrapped air from the concrete mixing or agitation must be minimised by efficient compaction and consolidation. Compacted concrete will generally retain <1% of entrapped air but some admixtures can entrain additional unwanted air.

Bleed characteristics of the concrete
Bleed is very important in relation to the escape of the excess air. Adjustments to the fine aggregate grading will permit the air to escape early before the surface has any chance of sealing. An early bleed is required especially when a dry-shake topping is used. The bleed must be sufficient to thoroughly wet the topping and to hydrate the cement component.

Application of dry-shake toppings
The risk of delamination is increased when using a dry-shake topping. Sufficient bleed water is required to thoroughly wet the dry-shake within about 1 minute of application. This indicates that the water is present to hydrate the cement and that bleed channels exist through the topping to allow air to escape.

Delaminated surfaces can be repaired by removing the affected surface in areas bounded by shallow saw cuts and then filling with a cementitious- or resin-based proprietary mortar system, for example:

- *Small patches*. Cut a rectangle around the area and prepare (scabble or similar) the parent concrete to about 2–10mm. Infill with a suitable repair material.
- *Larger areas*. Cut a rectangle around the area and prepare to a depth of 20–30mm. Infill with cementitious mortar incorporating the dry-shake topping where applicable to match the existing finish.

For additional information, see Concrete Advice Sheet 18 *Delamination Of Concrete Surfaces* [11].

2.9 Surface aggregate

Occasionally, aggregate particles lie exposed at or are very close to the surface. If they are well ‘locked into’ the surface they are unlikely to affect durability, although their appearance may be considered an issue. However, particles can be dislodged by MHE or other actions, leaving small surface voids. These voids can be drilled out and filled with resin mortar – see Concrete Advice Sheet 36 *Surface Blemishes* [12].

Where soft particles, such as naturally occurring mudstone, chalk or lignite, are exposed in the surface, they can be removed by drilling and replaced with mortar as described above. The durability of the floor is unlikely to be affected.

2.10 Surface fibres

There is a risk of some fibres protruding through the surface but their incidence can be significantly reduced by the use of a dry-shake topping.

Surface fibres can be cut flush with the surface. Any significant resultant blemishes can be treated with a suitable resin-based material. Macro-synthetic fibres tend to be spun by the power finishing process, leaving fan-shaped imprints and unravelled fibres.

The acceptability of fibres at a surface is subjective and depends on the use of the floor.

3 Surface regularity

Surface profiles of floors must be controlled so that departures from a theoretically perfectly flat plane are limited to an extent appropriate to the planned use of the floor. For example, high-lift materials handling equipment (MHE) requires tighter control on surface regularity than for a low-level factory or warehouse.

Inadequate surface regularity increases the risk of collision between the trucks and racking, causes driver fatigue and forces materials handling equipment to be operated at lower speeds.

The floor should have an appropriate **flatness** in order to provide a suitable surface for the operation of materials handling equipment, and an appropriate **levelness** to ensure that the building as a whole, with all its static equipment and MHE, can function satisfactorily. The difference between flatness and levelness is illustrated in Figure 3.1.

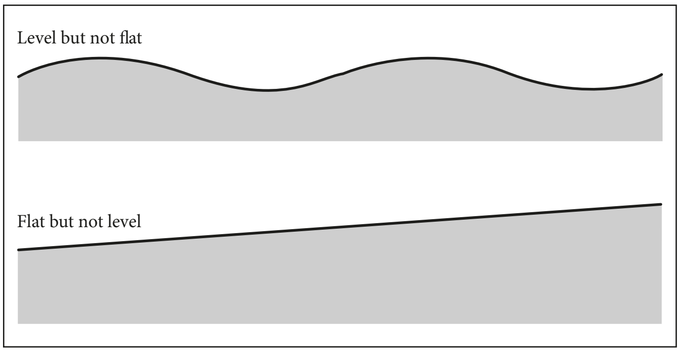


Figure 3.1: Flatness and levelness.

It can be seen that flatness is generally related to variations over shorter distances whereas levelness is generally related to longer distances. These distances are not easily definable, but traditionally flatness has been controlled over a distance of 600mm and levelness over a distance of 3m. Where MHE is operated in defined-movement areas (see Section 3.4), floor surfaces are measured over distances relative to the dimensions of the MHE.

The methods of assessing surface regularity described below assume the floor is to be horizontal and not laid to falls.

3.1 Departure from datum

The deviation in height of the surface of all new floor construction should be within ±15mm of a fixed datum plane. Where an original datum plane is not available, no point should be outside ±15mm of the mean floor level.

3.2 Free and defined-movement

In warehouses, MHE is used in two distinct areas of traffic movement:

- In **free-movement (FM) areas**, MHE can travel randomly in any direction – see Figure 3.2. Free-movement areas typically occur in warehouses with wide aisle racking installations, factories, retail outlets, low-level storage, marshalling zones and food distribution.
- In **defined-movement (DM) areas**, vehicles use fixed paths. Defined-movement areas are usually associated with high-level storage racking with very narrow aisles (VNAs) in warehouses (see Figure 3.3).

Distribution and warehouse facilities often combine areas of free-movement for low-level activities such as unloading and packing alongside areas of defined-movement for high-level storage.

Different surface regularity specifications are required for each floor use so that appropriate performance of the floor can be achieved. The different specifications are reflected in the survey techniques used and the limits on measurements (properties) that are prescribed.

Figure 3.2: A free-movement area.

Figure 3.3: A defined-movement area in a very narrow aisle.

Where racking layouts have not been determined at the time of construction, the developer is advised to build to as high a standard as possible: a free- movement surface regularity category FM2 (see Table 3.1) is suggested. This will limit the amount of grinding required in aisles to meet defined-movement tolerances if VNA is subsequently installed.

3.3 Surface regularity in free-movement areas

In assessing the surface regularity in FM areas it is not possible to survey an infinite number of points and so a sample representing the floor is surveyed. Free-movement floors and associated construction tolerances are not intended for VNAs, where a defined-movement specification should be used.

3.3.1 Choosing the free-movement floor classification

New floors should be constructed to the highest practical standard of levelness and flatness. FM2 is suggested in order to give the greatest flexibility for future use of the building.

When deciding on the classification, it should be recognised that, apart from a higher potential cost of the floor, the requirement for higher flatness tolerances may lead to construction methods with more formed joints.

3.3.2 Properties measured

Two properties are measured in FM areas, as defined below.

Property E
To control levelness, the elevational difference in millimetres directly between fixed points 3m apart (not across the diagonals), see Figure 3.4.

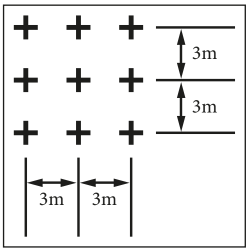


Figure 3.4: Property E, 3m grid.

Property F
To control flatness, the change in elevational difference between two consecutive measurements of elevational difference each measured over 300mm, see Figure 3.5.

In addition, the deviation in height of the surface of all new floor construction should be within ±15mm of a fixed datum plane. The level data from the Property E survey should be used for this purpose.

3.3.3 Surveying

A 3m grid of points is accurately set out on the whole of the floor area and elevations are taken on these points. The grid location should be recorded accurately so that the points can be revisited if subsequent level checks are needed. Areas within 1.5m of a wall, column or other existing structure are not surveyed.

Property E is measured between all adjacent survey points on the grid.

Property F is measured across a sample of the grid lines used to measure Property E. The sample should consist of a minimum total length of survey runs in metres calculated as the floor area in square metres divided by 10. The runs should be distributed uniformly across the floor or sections of large or irregular floors with the total length of runs in each direction being equal. The location of these Property F runs should be recorded.

Surveying techniques
Property E is measured using a precise level and staff, or other method with appropriate accuracy. Property F is usually measured using specialist digital equipment. These techniques are illustrated in Figure 3.6a and b.

Figure 3.6: Floor surveying equipment.
(a) optical level for measuring property E;
(b) digital instrument measuring property F.

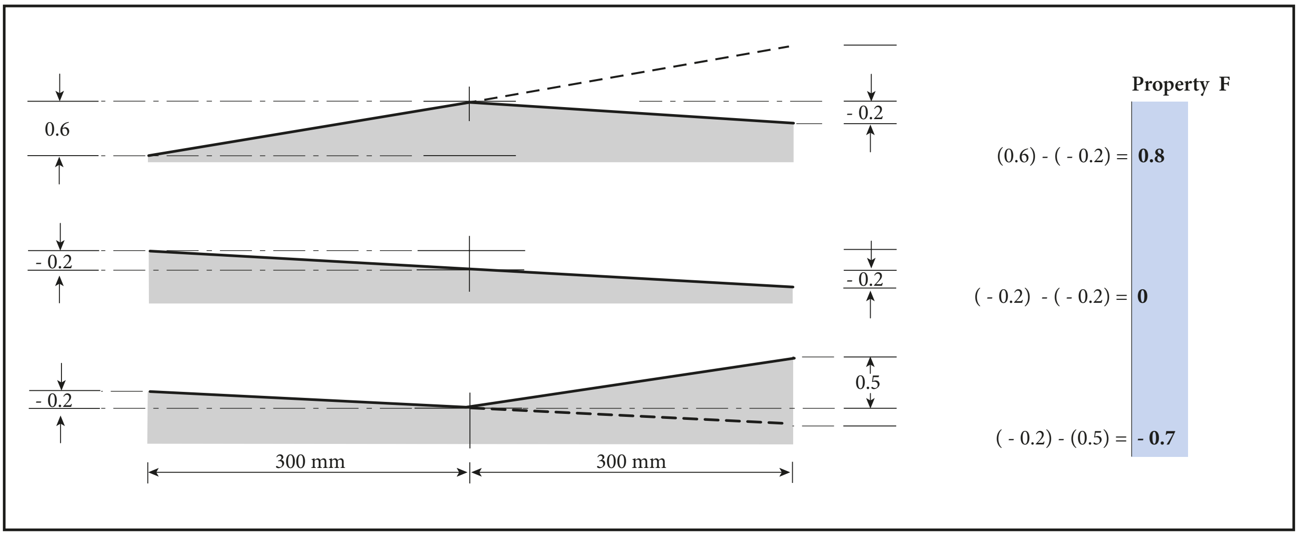


Figure 3.5: Three examples of Property F elevational differences.

Data analysis and permissible limits

The Property E data are analysed and the 95 percentile value is calculated. The Property F data for the total sample of Property F runs are analysed and the 95 percentile value of the total sample is calculated.

The 95 percentile value is the Property value below which 95% of the values will fall. Five per cent of the values will be greater.

Upper limits on the 95 percentile values for Properties E and F are given in Table 3.1. The floor is non-compliant if:

- the maximum permitted 95 percentile values are exceeded
- any point on the Property E survey grid is outside the ±15mm of datum.

Table 3.1: Permissible 95 percentile values on Properties E and F.

Floor class	Typical floor use	Property E	Property F
FM1	Where very high standards of flatness and levelness are required. Reach trucks operating at above 13m without side-shift.	4.5	1.8
FM2	Reach trucks operating at 8 –13m without side-shift.	6.5	2.0
FM3	Retail floors to take directly applied flooring. Reach trucks operating at up to 8m without side-shift. Reach trucks operating at up to 13m with side-shift.	8.0	2.2
FM4	Retail floors to take applied screeds. Workshops and manufacturing facilities where MHE lift heights are restricted to 4m.	10.0	2.4

Note: Side-shift is the ability of a truck to adjust the pallet transversely to the fork direction.

Reporting

The 3m grid of level readings and the level differences (Property E) should be presented in relation to the building layout. Property E values exceeding the 95 percentile limits should be highlighted.

The location of the Property F runs should be shown and linked to the Property E grid. Property F values should be presented for each run with values exceeding the 95 percentile limits highlighted.

Non-compliance

Where the required property limits are exceeded, it is recommended that individual measurements are examined in detail to determine the significance of any possible effect on the performance of a floor. Remedial actions will affect the appearance of the floor.

3.4 Surface regularity in defined-movement areas

Defined-movement is most commonly associated with very narrow aisles (VNAs). In these aisles, the surface regularity of the floor is a critical factor in the performance of the MHE.

If the precise positions of the aisles are not known at the time of floor construction, it is not appropriate to specify the surface regularity of the aisles as defined-movement areas.

These survey methods are used for MHE pathways and have no relevance to the areas of floor under the racking. Areas away from racking such as goods in and out and transfer areas should be regarded as free-movement areas.

Figure 3.7 shows the static lean and how the variation in floor level across an aisle between the wheel tracks of a truck is magnified at the top of the mast in direct proportion to its height. Variations in level also induce dynamic movements in the mast that can significantly magnify the static lean.

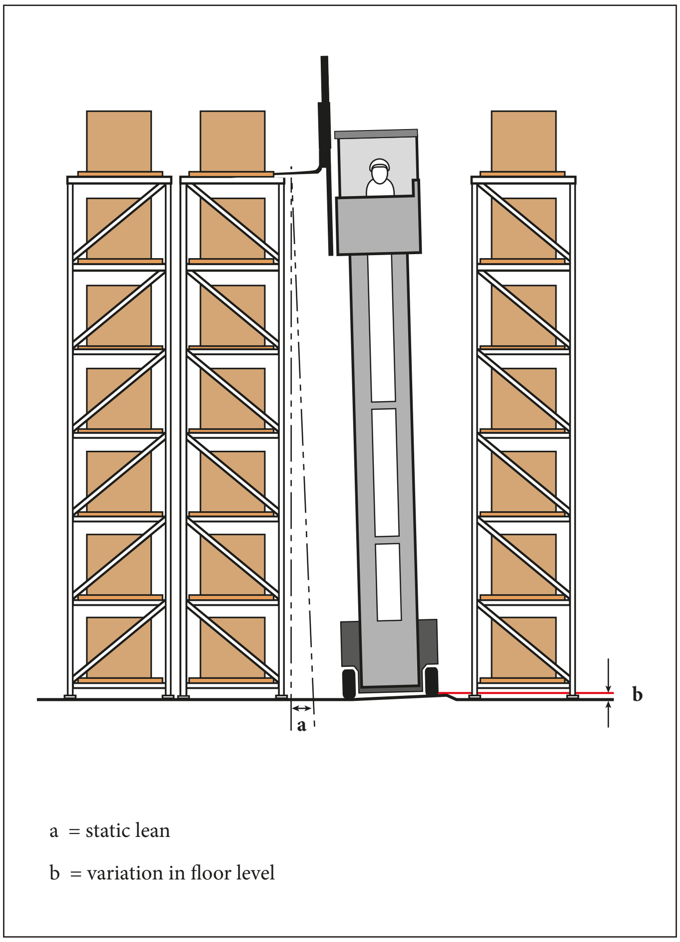


Figure 3.7: Static lean.

3.4.1 Choosing the defined-movement floor classification

The defined-movement specification appropriate for the racking top beam height should be specified. Classifications of floors based on racking top beam heights are given in Table 3.2.

Table 3.2: Permissible limits on Properties dZ, dX, d^2Z and d^2X in defined-movement areas.

Floor classification	Racking top beam height	Property Z_{SLOPE}	Property dZ	Property d^2Z	Property dX	Property d^2X
Calculation	–	mm per m	$Z \times Z_{SLOPE}$	$dZ \times 0.75$	Fixed values $2 \times Z_{SLOPE} \times 1.1$	Fixed values
DM1	Over 13m	1.3	$Z \times 1.3$	$Z \times 1.0$	2.9	1.5
DM2	8–13m	2.0	$Z \times 2.0$	$Z \times 1.5$	4.4	2.0
DM3	Up to 8m	2.5	$Z \times 2.5$	$Z \times 1.9$	5.5	2.5

Properties measured

The following properties are defined in Figures 3.8–3.10 as follows:

- **Property Z:** The transverse dimension between the centres of the truck front wheels, in m.
- **Property X:** The longitudinal dimension between the centre of the front and rear truck axles. This is taken to be a fixed 2m.
- **Property Z_{SLOPE}:** The cross-aisle slope between the centres of the truck front wheels in mm/m.
- **Property dZ:** The elevational difference in mm between the centres of the truck front wheels.
- **Property dX:** The elevational difference in mm between the centre of the front axle and the centre of the rear axle.

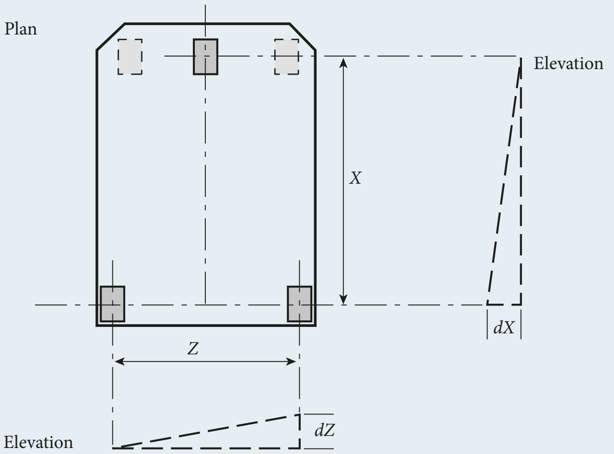


Figure 3.8: Symbols for dimensions.

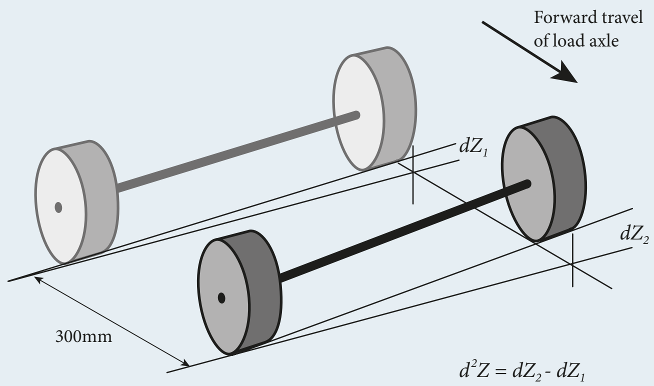


Figure 3.9: Determination of d^2Z.

Property d^2Z: The change in dZ in mm over a forward movement of 300mm along the wheel tracks
Property d^2X: The change in dX in mm over a forward movement of 300mm along the wheel tracks.

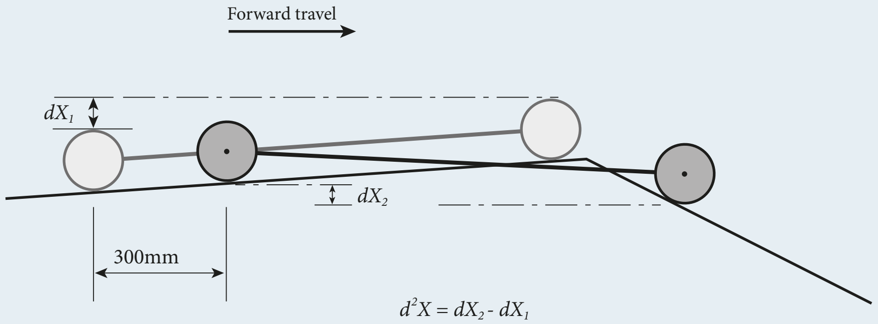


Figure 3.10: Determination of d^2X.

3.4.2 Surveying

Aisles are surveyed over their full length along the wheel tracks of the MHE starting with the load axle at the first rack upright.

Surveying techniques
Properties dZ and dX are measured using specialist digital equipment, commonly known as profileographs – see Figure 3.11. These produce continuous or semi-continuous data readings. Data readings should be taken at not greater than 50mm intervals. Properties d^2Z and d^2X are derived by computation of the data for Properties dZ and dX.

Figure 3.11: Profileograph in use in the position of an aisle.

Data analysis and permissible limits
The survey data are analysed and compared with the permissible limits for Properties dZ, dX, d^2Z and d^2X as given in Table 3.2. The floor is non-compliant if any measurement in any aisle exceeds any Property limit.

Reporting
Data are typically presented graphically and should be in relation to the building layout. Summary data should be provided for each aisle with non-compliances highlighted.

Non-compliance
Where limits are exceeded, it may be possible to grind the high areas of the surface or, in unusual circumstances, to fill the low areas of the surface. If wheel tracks have been ground or filled, the wheels should be in full contact with the floor surface so that no transverse thrust or other stresses on wheels are created – see Figure 3.12. Grinding (see Figure 3.13 and 3.14) will affect the appearance of the floor.

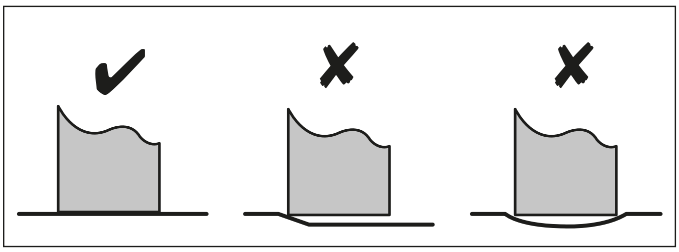


Figure 3.12: Remediation in wheel tracks.

Figure 3.13: Typical automatic grinding operations.

Figure 3.14: Typical manual grinding operations.

3.5 Survey practice for all floor types

3.5.1 Accuracy of surveys

All surveying instruments should be calibrated to an accuracy of 0.1mm and all survey data should be reported to 0.1mm.

There are no Standards covering the use of specialist floor survey equipment such as profileographs. Third party accreditation, such as UKAS, is available for these survey methods.

Timing of surveys
Surveys should be carried out within one month of completing the whole floor, or major sections of it, to check that 'as-built' it complies with the specification.

For purposes of quality control, assessments can be made at any stage during the construction to check the completed floor will meet the specification.

3.6 Change of floor flatness with time

Surface regularity can change over time. Deflection of suspended slabs and settlement of the ground, piles or other supporting structures can have significant effect.

Levelness and flatness can also change at the edges or corners of floor panels and as a result of curling – see Section 2.7.

4 Warehouse equipment and floor loadings

The common loads on floors in warehouses are the point loads from pallet racking, the associated materials handling equipment (MHE) and from mezzanines. Other loads arise from uniformly distributed loads (UDL) such as stacking of palletised products or bulk loose materials and from line loads such as internal walls and floor rail systems.

4.1 Load type

It is generally the case that point loads are critical for design and reliance should not be placed on the commonly specified UDL alone.

In all cases, the design should be based on anticipated loads from all forms of equipment and other loads and the specifier should take into account future possible uses of the floor. Higher buildings should be expected to take greater loads from, for example, pallet racking.

Point loads from pallet racking and mezzanines are treated as static loads while MHE is treated as a dynamic load that attracts greater safety factors in design.

Where very heavy MHE is in use, enhanced fatigue effects need to be considered. Typically, this will arise where heavy counterbalance trucks are used for applications such as double pallet handling, paper reel handling with clamps and loads in heavy engineering works. See Figures 4.1 and 4.2.

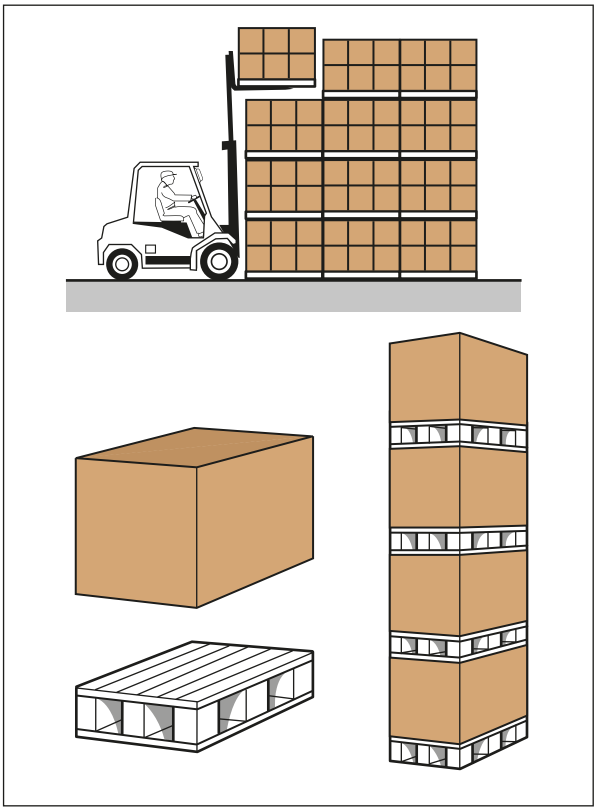


Figure 4.1: Block stacking.

Figure 4.2: Block stacking of pallets.

The floor design should also take into account temporary loads from cranes or other MHE used during installation, maintenance and removal of manufacturing or storage equipment. Such temporary loads may be greater than permanent loads.

4.2 Warehouse equipment – static loads

4.2.1 Adjustable pallet racking (APR)

Pallet racking is used for storage of products on pallets at up to considerable heights, while providing access to the individual pallets. APR consists of frames of pairs of uprights connected by bracing. Beams supporting the pallets span between these frames. See Figure 4.3.

Rows of racking are usually placed back to back, with a clearance of 250–350mm between the inner uprights. Aisles between the racks allow loading by fork-lift trucks or stacker cranes. Loads from back-to-back racking, as shown in Figure 4.4, are commonly the governing case for slab design. The self-weight of the racking should be taken into account.

Pallets are commonly stored directly on the floor slab beneath the racking. Where rail-guided MHE is used, the first level of pallets is carried on beams close to floor level.

Figure 4.3: Back-to-back racking.

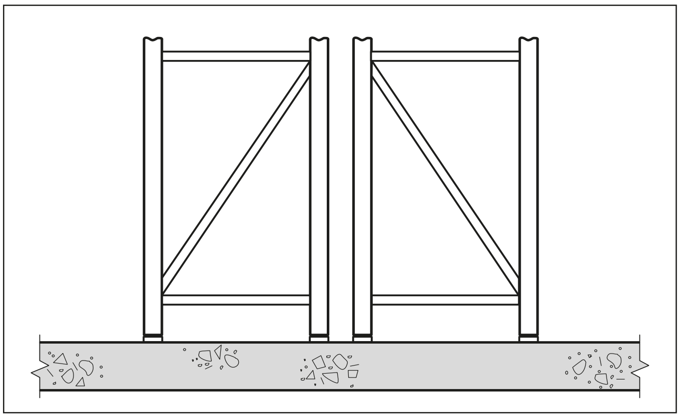


Figure 4.4: Typical 'back-to-back' configuration of storage racking.

4.2.2 Mobile pallet racking

This consists of sets of racks on mobile chassis running on floor-mounted rails (see Figure 4.5). The racks are individually driven by electric motors so that each aisle can be opened up as required for access to individual pallets.

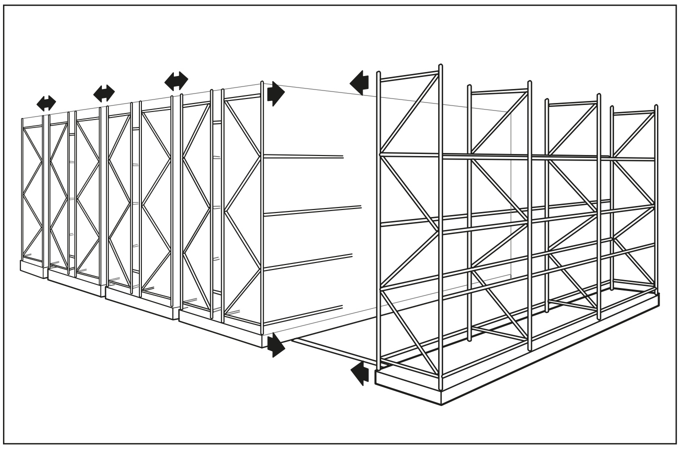


Figure 4.5: Mobile pallet racking.

Laden rack stability usually limits the lift height to about 13m. The racking applies point loads to the rails. Depending on the stiffness and fixing arrangements of the rails, the load on the floor may be considered as a point load or a line load.

4.2.3 Live storage systems

Live storage systems provide a high-density block of loads without load selectivity (see Figure 4.6). Incoming pallets are placed on the 'high' end of a downward sloping set of roller conveyors. As loads are removed from the 'low' end, the pallets move by gravity towards the outlet end of the racking. This type of storage enables stock to be rotated on the first-in, first-out principle.

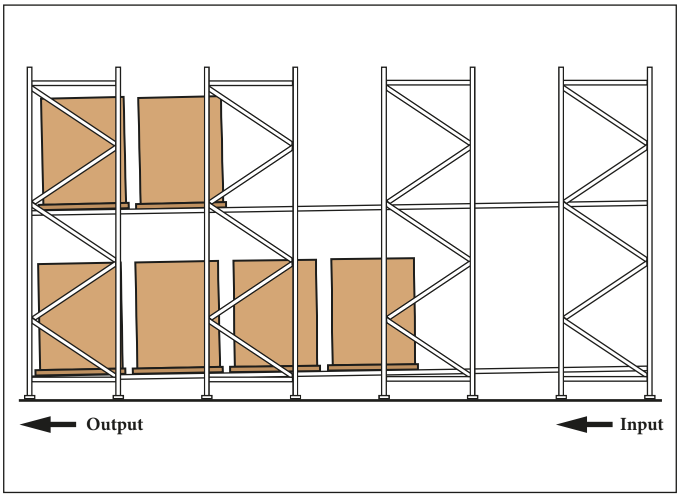


Figure 4.6: Live storage systems.

4.2.4 Drive-in racking

In drive-in racking systems (Figure 4.7) there is no division by aisles and therefore high storage density is achieved. Cantilever brackets attached to the racking frames support the pallets. This type of racking can be up to about 12m high.

Figure 4.7: Drive-in racking.

4.2.5 Push-back racking systems

These provide a high-density block of loads. Incoming pallets are placed on the push-back carrier; subsequent loads are positioned on the next available carrier and used to push the previous load back up a slope. See Figure 4.8. Typically, installations are less than five pallets in depth and are not usually higher than 8m.

Figure 4.8: Push-back racking.

4.2.6 Cantilever racks

Cantilever racks (Figure 4.9) are used to store long loads and are sometimes referred to as 'bar racks'. The racks consist of a row of uprights with arms cantilevering out on either or both sides and are often used in conjunction with side-loading fork-lift trucks. They are not usually higher than 8m and can be heavily loaded.

Figure 4.9: Cantilever racking.

4.2.7 Mezzanines

Mezzanines (see Figures 4.10) are commonly used for production, handling machinery and storage. Column baseplates should be designed to provide the required load-spreading capability. Additional slab reinforcement or discrete foundations may be required.

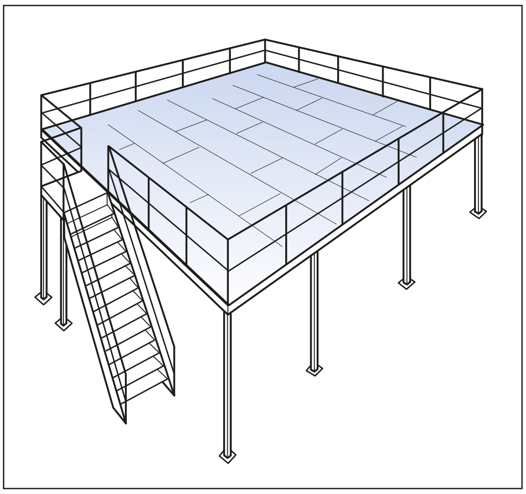


Figure 4.10: Mezzanine (raised platform).

4.2.8 Clad rack structures

In clad rack structures (Figure 4.11) the racking itself provides the structural framework for the building and supports the walls and roof. Clad rack warehouses can be up to 45m high and the point loads can be extremely high and at close centres. It is not possible to give typical point loads from these structures onto the floor slab as each application will depend upon the size of building, the goods to be stored, as well as wind and snow loads. Clad rack design and construction is a specialist field and the advice of the rack supplier should be sought.

Figure 4.11: Clad rack system during construction.

4.3 Warehouse equipment – dynamic loads

In order to design floors to support MHE loads, the all-up weight of the trucks must be known, along with maximum axle and individual wheel loads and contact areas of the wheels. The carrying capacity of the MHE is not an adequate indicator of the loads applied to a floor. The load distribution and therefore the axle weights can vary significantly between the loaded and unloaded condition and the truck manufacturer's data should be used.

Some common MHE types are described below, with some guidance on floor surface requirements.

4.3.1 Pallet trucks

Pallet trucks are used at floor level for moving single or multiple pallets and for order picking. They can be controlled by pedestrians alongside or operators riding on them (Figure 4.12).

Figure 4.12: The small-wheeled pallet truck, ride-on type.

Floor surfaces on which this equipment operates should be flat and have a good standard of levelness, particularly across joints. Joints in floors are prone to damage by the small hard wheels on this type of equipment.

4.3.2 Counterbalance trucks

Counterbalance trucks are fitted with telescopic masts with the load carried ahead of the front (load) wheels (Figure 4.13). They are used within buildings and externally for block stacking, in storage racking up to about 7m high and for general materials movement. Because they approach stacking and racking face on, aisle widths are generally in excess of 3m.

Truck tyres are usually solid rubber or pneumatic. Rubber tyres can be aggressive on dusty or wet floor surfaces and it is important to keep floors clean to avoid such conditions. Counterbalance trucks can tolerate relatively uneven surfaces and joints.

Figure 4.13: Counterbalance truck.

4.3.3 Reach trucks

Reach trucks (Figure 4.14) pick up and deposit pallets by means of a moving mast that reaches forward of the load wheels. The mast retracts within the truck wheelbase when moving. They normally operate in aisles of 3m or more. Lift heights do not normally exceed 10–12m.

Truck tyres are generally of polyurethane, which are not unusually aggressive to surfaces but can damage joints. Floor surfaces should be flat and level with no wide, stepped or uneven joints.

Figure 4.14: Reach truck with extended mast.

4.3.4 Front and lateral stackers (VNA trucks)

These lift trucks handle pallets at right angles to the direction of travel and are also known as very narrow aisle (VNA) trucks. Operators travel at floor level or in a compartment that lifts with the forks; these are known as ‘man-down’ and ‘man-up’ trucks respectively (see Figure 4.15).

Truck tyres are generally of polyurethane, which are not unusually aggressive to surfaces but can damage joints.

Figure 4.15: 'Man-up' stacker truck in a VNA warehouse.

Most trucks have three wheels – two on the front load axle and one drive wheel at the rear. Some have two close-coupled wheels at the rear acting as one wheel. A few trucks have four wheels with one at each 'corner'. When operating in the aisles, the trucks are guided by rails at the sides of the aisle or by inductive guide wires in the floor and are not directly steered by the operator.

The inclusion of inductive guide wires in the slab may affect its design thickness. Guide wires need to be kept clear of steel reinforcement. Steel fibres in concrete do not normally affect guidance systems if adequate measures are undertaken to ensure even fibre distribution.

Some floor-running stackers have fixed non-retractable masts and run between top guidance rails that can also provide power to the truck through a bus-bar system. These systems are designed to provide some limited restraint to sideways movement of the mast to effectively stiffen it but they are not designed to compensate for inadequate floor flatness.

In VNAs, trucks run in defined paths and so it is appropriate to measure and control the flatness in each of the tracks. Floor surfaces should be flat and level with no wide, stepped or uneven joints. Floors are specified with a defined-movement classification that depends on the maximum height of lift, as defined in Section 3.4 and Table 3.2.

4.3.5 Articulated counterbalance trucks

Articulated trucks are three- or four-wheel counterbalance trucks with the ability to rotate the front section of the truck which carries the mast, allowing the pallet to be inserted into the racking. Articulated counterbalance trucks, see Figure 4.16, can operate in aisles as small as 1.6m and to a racking height of 12m. Floor surfaces should be flat and level with no wide, stepped or uneven joints.

Figure 4.16: Articulated counterbalance lift truck.

4.3.6 Stacker cranes

Stacker cranes run on floor-mounted rails (Figure 4.17). They have fixed masts with a top guidance rail. There are no onerous floor flatness requirements as the rails are set level by shimming or grout. However, the floor should have a good overall level to datum as the racking and rails are fixed level to a datum. Limiting long-term settlement of slabs is important for stacker crane installations as changes in levels can lead to operational problems. Horizontal and uplift loads should be considered. Uplift at buffer or emergency stop locations can be significant and may need separate foundations.

Figure 4.17: Stacker crane in a automated storage and retrieval system.

5 Soils and support structures

The structural integrity of the layers below a ground-supported slab or the construction and capacity of the piles beneath a pile-supported slab is of vital importance to the bearing capacity and serviceability of the slab and this aspect is covered in this section. This section also covers the build-up of cold store slabs where the slab is supported on a layer of insulation material.

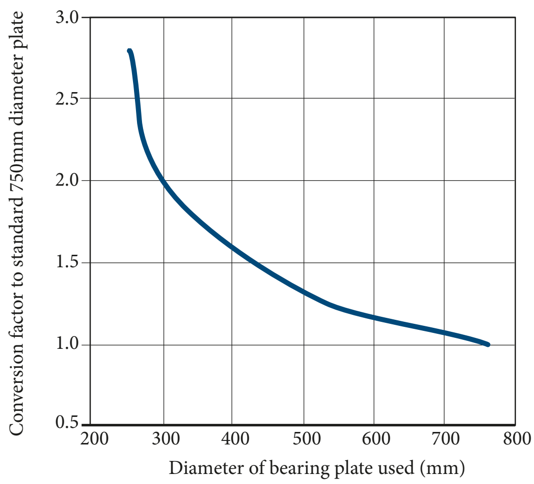
5.1 Soil investigation

A soil investigation in accordance with the recommendations of Eurocode 7 (EN 1997-1[13]) must be undertaken to examine the ground conditions on the site. An appropriately qualified geotechnical engineer should plan the investigation and interpret the results. The responsibility for the scope, commissioning and execution of the investigation should be clearly established. The investigation should include an estimate of all the parameters needed for the design of the slab support system including long-term settlement under load.

Cohesive soils (clays and silts) tend to consolidate under load, leading to long-term settlement of the slab. This effect could result in differential settlement between heavily and lightly loaded areas, with a consequential effect on floor surface regularity.

For ground-supported floors, the design process requires the measurement of the modulus of subgrade reaction '*k*'. Derivation of the value of *k* from the California bearing ratio (CBR) tests is not ordinarily acceptable.

For floor construction, the values of *k* of the subgrade should be verified from plate bearing tests in accordance with EN 1997-2 [14]. Larger plates give greater accuracy and it is preferable to use a plate of diameter 750mm. If other loading plate diameters are used it is necessary to employ a conversion factor, as shown in Figure 5.1. The minimum size of plate used should be 300mm. Values of *k* should be read at a fixed settlement of 1.25mm. Generally, there should be a minimum of one plate-loading test per 2000m² of floor.



Note: The *k* value obtained with the plate used should be divided by the appropriate conversion factor on the y-axis.

Figure 5.1: Conversion factors for different loading plate sizes.

Materials closer to the ground surface have more effect on the measured subgrade properties than those at larger depths. Measured values of '*k*' do not reflect long-term settlements due to soil consolidation under loading. However, low values of *k* are indicative of plastic behaviour of the near-to-surface soils. Checks should be made on the likely deformation of the subgrade, particularly for soils with low *k* values.

Whilst it is recommended that *k* values should be determined directly from plate loading tests on the subgrade, it is recognised that in some cases this is not possible and indirect methods of assessing the *k* value from other soil parameters may be necessary. In these cases, advice from a qualified geotechnical engineer should be sought and this assessment should take into account the inherent inaccuracies of the method used.

In the case of pile-supported slabs, the ground investigation should also establish soil parameters that will enable the load-bearing and deflection (both short-term and long-term) characteristics of the piles to be determined.

5.2 Subgrade

Subgrades may take a number of different forms, for example natural ground, imported fill or stabilised or dynamically compacted in-situ soil. They should provide uniform support and so hard and soft spots should be removed and replaced with material placed and compacted so as to achieve properties as nearly as possible conforming to the surrounding soil.

The design and construction of suitable subgrades for ground-bearing slab construction is beyond the scope of this document and recognised guidance on the relevant form of construction should be followed, such as:

- *Specification for Highway Works* Series 600[15] and 800[16]
- *Hydraulically Bound Mixtures for Pavements* [17]
- EN 14227 *Unbound and hydraulically bound mixtures* (various parts)[18]
- Highways Agency *Design Manual for Roads and Bridges* Vol 4 section 1 part 6 (HA 74/07)[19].

5.3 Sub-base

A sub-base has three main purposes, as follows:

- to transmit the load from the floor slab to the subgrade, thus improving the quality of support from the underlying soil – ground supported only
- to provide a level formation for the construction of the floor slab
- to provide a firm working platform for construction activity.

Sub-bases should be constructed from stable, well-graded granular material such as Type 1 or Type 2 complying with and laid in accordance with the Highways Agency *Specification for Highway Works*, Series 800, *Road pavements – unbound materials*[16]. Alternatively, bound materials such as hydraulically bound mixtures

(HBM) can be used for sub-base construction, provided they are used in accordance with the appropriate specification, such as those referred to in Section 5.2.

All fill materials should be checked for content of potentially expansive materials or reactions with lime and/or cement.

If granular sub-base material is used, it should generally have a minimum compacted thickness of 150mm. However, thicknesses of less than 150mm may be used provided the material used has a maximum particle size and distribution such that it can be fully compacted to achieve a hard, flat, durable surface with adequate strength. There is generally no requirement for materials in sub-bases for internal slabs to be frost resistant. This also applies to cold stores where the sub-base is protected from frost by an insulation layer and heater mats.

Where very heavy counterbalanced trucks or other materials handling equipment with similar wheel loads are to be used on the floor, such as in paper stores, reference should be made to Sections 7.3 and 8.3.

It is particularly important that the surface of the sub-base should be well closed and free from movement under compaction plant and from ridges, cracks, loose material, potholes, ruts or other defects.

Any trimming of the surface should leave the sub-base homogeneous and well compacted. Trimming layers cannot make up for deficiencies in the sub-base construction. Sand or stone fines may be used solely for closing the surface.

It is essential to minimise the risk that the slab top level and sub-base top surface are both out of tolerance at the same point and in the adverse direction, as this may reduce the thickness of the concrete slab so much that its load-carrying capacity is reduced to an unacceptable extent. The finished surface of the sub-base should be within +0, –25mm of the datum for the bottom of the slab. Good practice should consistently achieve +0, -15mm. A low sub-base will on average thicken the slab and therefore use more concrete. In the case of pile-supported slabs the tolerance is also subject to the constraints identified in Section 5.7. Positive tolerances above zero datum should not be permitted as these will directly affect the thickness of the slab.

Sub-base finished levels should be surveyed on completion. Survey points should be not more than 4m apart.

5.4 Membranes

The main purpose of a membrane is to reduce the friction between the slab and the sub-base. Membranes are normally 1200 gauge (300 micron) plastic. Slip membranes do not compensate for abrupt variations in level of the sub-base, which should be flat and smooth. Any foundation or pile head that abuts to the slab soffit should be provided with an additional layer of membrane.

It is important to lay the membrane without creases and that it is overlapped at the edges by at least 300mm. Care must be taken to ensure that it is not damaged during the construction process.

The plastic sheet will inhibit the loss of water and fines from the concrete to the sub-base. However, in some circumstances, a polythene slip membrane may not provide sufficient resistance to water vapour – see BS 8103 *Structural design of low-rise buildings* [20] and BS 8102 *Code of practice for protection of below ground structures against water from the ground* [21].

Gas membranes and venting systems have become commonplace as more construction is carried out on contaminated land. Guidance can be found in CIRIA Report 149 *Protecting development from methane* [22].

5.5 Slabs on insulation

Floors in temperature-controlled buildings incorporate an insulation layer above a heater mat to protect the sub-base from frost heave. The insulation may be laid on a concrete blinding layer or, more commonly, on a concrete base slab, which typically also supports the cold store insulated wall panels. For pile-supported floors, the base slab will usually be required to span between the piles and support the insulation, wearing slab and imposed loads. This avoids the 'cold bridging' that will occur if the piles pass through the insulation to support the wearing slab.

A typical layer structure is shown in Figure 5.3. Machine laying may be logistically difficult as the plant can not track over the insulation. Therefore, in general, manual laying followed by power floating and trowelling is necessary.

Figure 5.2: Sub-base preparation.

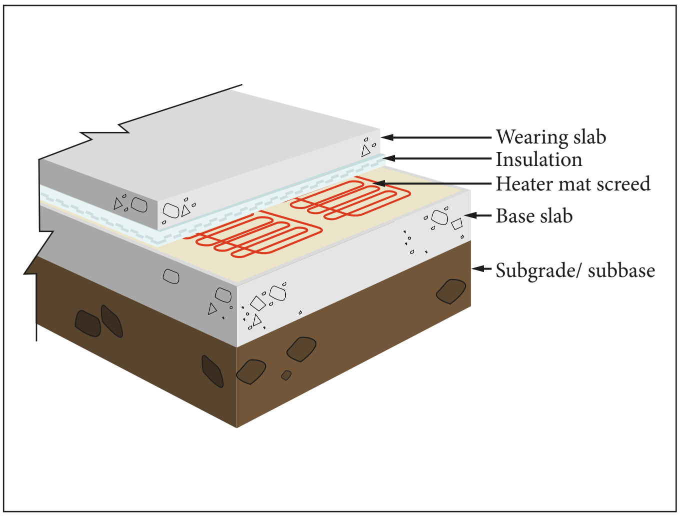


Figure 5.3: Typical construction layers in cold stores.

Information on cold store heater mats and other aspects of cold store construction can be found in *Guidelines for the specification, design and construction of cold store floors* [23].

The design of the wearing slab is no different in principle to the design of a ground-supported slab. It is necessary, however, to determine the correct modulus of subgrade reaction '*k*' to use in the calculations. This will vary depending on the type of insulation, thickness of insulation, presence or absence of a base slab and, in the case of ground-supported slabs, the modulus of the ground. For ground-supported slabs, a key factor to consider is whether the modulus of the insulation is higher or lower than the modulus of the ground. The *k* value for the type and thickness of insulation proposed should be obtained from the manufacturer. If the information is not available, a plate bearing test should be undertaken on the insulation, using an appropriate plate size and test load, as described in Section 5.1. Long term creep effects should also be considered in the assessment of the '*k*' value.

The design procedure is as follows:

Ground-supported slabs, no base slab provided
Determine the modulus of subgrade reaction for the ground (refer to Section 5.1). Use the lower of the 'ground' modulus and 'insulation' modulus and design the wearing slab as a ground-bearing slab using the methods described in Sections 6 and 7.

Ground-supported slabs, base slab provided
Using the modulus of subgrade reaction of the insulation, design the wearing slab as a ground-bearing slab using the methods described in Sections 6 and 7. If the modulus of the ground is lower than the modulus of the insulation, use the modulus of the ground and design the base slab as a ground-supported slab. The distribution of load on the base slab should be determined, conservatively, by assuming a 30° load spread through the wearing slab or, more precisely, by finite-element analysis. If the modulus of the ground is greater than the modulus of the insulation, no structural design check of the base slab is required.

Pile-supported slabs
Using the modulus of subgrade reaction of the insulation, design the wearing slab as a ground-bearing slab using the methods described in Sections 6 and 7. Design the base slab to support the loads from the wearing slab and span between the piles. The distribution of load on the base slab should be determined, conservatively, by assuming a 30° load spread through the wearing slab or, more precisely, by elastic finite-element analysis.

It should be noted that although wearing slabs in freezer stores are subject to temperatures well below 0°C, freeze–thaw damage does not generally occur. This is because, unlike external service yard slabs, the wearing slab is not subject to regular cycling between freezing and ambient temperatures, and is not usually saturated while frozen. Joints in wearing slabs will, however, typically open more than slabs at ambient temperature, as the thermal shortening is greater. The design of the joint and joint sealant should take this factor into account.

5.6 Design model for a ground-supported slab

Ground-supported slabs are not rafts and do not have the ability to span over soft zones or poor-quality subsoil. They will tend to conform to the shape of the subsoil as it deflects under loading or as the subsoil settles from the effects of consolidation or ground movements at depth.

In his design concept, Westergaard [24, 25] assumed that a slab acts as a homogeneous, isotropic elastic solid in equilibrium with the reactions from the subgrade which are vertical only and are proportional to the deflections of the slab. The subgrade is assumed to be an elastic medium whose elasticity can be characterised by the force that, distributed over unit area, will give unit deflection. Westergaard termed this soil characteristic the 'modulus of subgrade reaction' *k*, with units N/mm^2/mm.

A detailed discussion of *k* values is given in the comprehensive 1995 NCHRP Report 372, *Support under Portland cement concrete pavements* [26]. The report makes the important recommendation that the elastic *k* value measured on the subgrade is the appropriate input for design. It has been suggested that the addition of a granular sub-base can enhance the value of *k*. However, the enhancement that can be achieved in this way is, in fact, dependent on the type and magnitude of the imposed load and the nature of the sub-base. In any event, in normal circumstances such enhancements have little effect on the thickness design for flexural stresses. It is recommended therefore that the design of the slab should be based on the *k* value of the subgrade without enhancement.

It is recognised that in some cases the existing subgrade materials are improved by stabilisation or the addition of a designed capping layer. Where this has occurred it is considered appropriate to base the design of the slab on the *k* value of that stabilised or capping layer. Any potential enhancement of the *k* value arising from a regulating layer immediately beneath the slab should however be ignored. It is recommended that the expertise of a suitably qualified geotechnical engineer be sought to advise on the appropriate value of *k* in such circumstances.

5.7 Design model for a pile-supported floor

If geotechnical investigations indicate that ground conditions are inadequate for a ground-supported floor, the floor may be constructed on piles. For warehouses with racking, the design of the joint layout arrangement should take into account both the piling grid and the racking grid, see Appendix H for optimised pile layouts.

Most forms of piling can be used for pile-supported floors, including all forms of cast-in-situ concrete piles and driven piles of cylindrical or square precast concrete or steel sections. All piling should be designed and constructed in accordance with EN 1997-1: 2004 [13].

Where pile-supported slabs are used, long-term support to the slab is assumed to be provided solely by the piles and not by the sub-base. However, the sub-base provides support for the slab during construction and until it has achieved adequate strength. It is therefore important that the sub-base is sufficiently stable to resist deformation under construction traffic and loads and to provide a flat surface to enable the slab to undergo shrinkage without undue restraint.

It is strongly recommended that enlarged pile heads are provided.

5.7.1 Pile head construction

The sub-base preparation and pile head construction are integral to a successful foundation to carry the floor slab.

The purpose of a pile head is to:

- reduce the effective span of the slab
- increase the shear perimeter resisting punching shear
- provide a smooth bearing surface to minimise restraint.

Concrete pile heads should have vertical faces. They should be designed and constructed in accordance with the recommendations of Eurocode 2[27]. The pile head should include a reinforcement cage and starter bars passing down into the pile shaft.

The distance between vertical faces on plan should not exceed 3 times the diameter of the pile. The depth should be at least the diameter of the pile – see Figure 5.4.

Pile head design and detailing must consider both the permanent slab loading condition and the effect of construction plant trafficking over the pile heads. It is usually impractical to avoid trafficking of pile heads so a robust detail should be adopted. Damaged pile heads should be repaired.

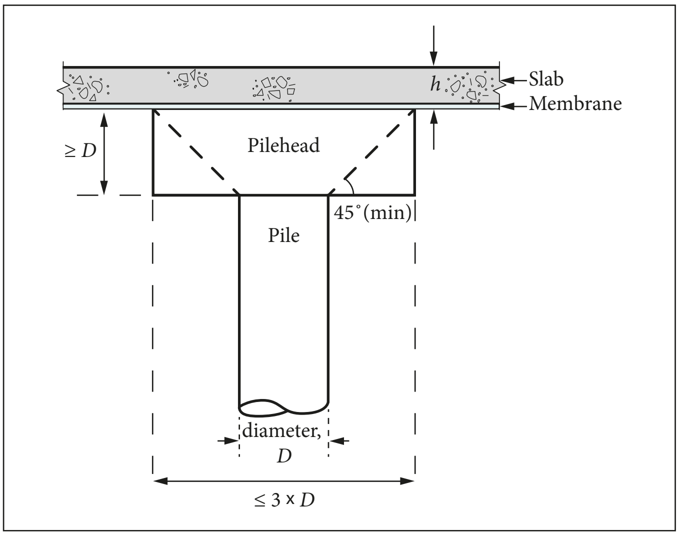


Figure 5.4: Pile-head detail.

The pile head should be level and have a smooth trowelled finish. The level tolerance should be no greater than +0, –25mm with respect to the slab soffit, with a slope not greater than 5mm over its width. It is important that the pile head does not project above the level of the finished sub-base.

To avoid restraint between the head and the underside of the slab it is essential that the pile head is not tied to the slab and the slip membranes should be laid over the pile head. The pile head must be constructed to an acceptable level and flatness to minimise restraint and should be finished by an experienced operative using a steel float. The pile head should be constructed flush with the sub-base as shown in Figure 5.5(a). However, it is possible that as a result of construction inaccuracies the pile head may in fact be constructed as shown in Figures 5.5(b)–(d), which would not be acceptable.

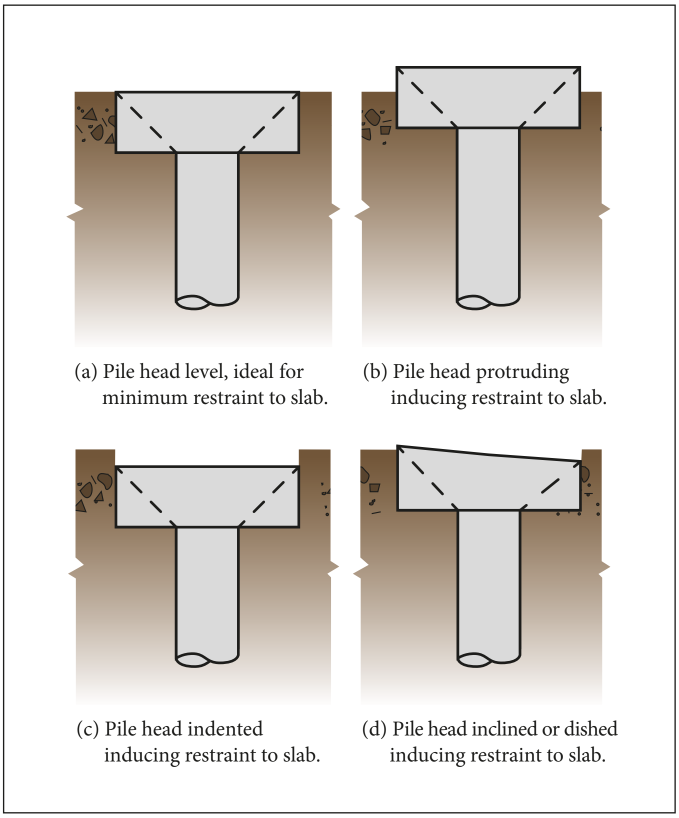


Fig 5.5: Pile heads and formation level.

6 Design – structural properties

Sections 6, 7 and 8 cover material properties, and the methods of analysis and design of ground-supported and pile-supported slabs.

The design analysis principles are generally in limit state format, in line with Eurocode 2[27]. Exceptions are the analysis of uniformly distributed loads (UDL) and line loads on ground-supported floors where a permissible stress approach is adopted with a global factor of safety being applied to the material properties of plain uncracked concrete.

Design checks are carried out on both the ultimate strength and, where appropriate, the serviceability of the slab.

6.1 Concrete

The strength properties of concrete are listed in Table 6.1, based on Eurocode 2[27], Table 3.1.

6.1.1 Flexural tensile strength

The flexural tensile strength of a plain concrete section is a function of the axial tensile strength and the section depth.

For slabs thinner than 600mm the flexural tensile strength $f_{ctd,fl}$ is obtained by multiplying the mean axial tensile strength by $(1.6 – h/1000)$, as Eurocode 2[27] expression 3.23.

$$f_{ctd,fl} = f_{ctm} \times (1.6 - h/1000)/\gamma_m \qquad \text{Equation (1)}$$

6.2 Reinforcement

6.2.1 Steel fabric and reinforcement bar

Steel fabric is commonly used in ground-supported floors and as supplementary reinforcement in steel-fibre-reinforced pile-supported slabs. In the UK it should be in accordance with BS 4483[28] with a characteristic strength of 500N/mm^2.

In the UK, steel reinforcement bar should be in accordance with BS 4449 [29], with a characteristic strength f_{yk} of 500N/mm^2. Where bars are used, for example to increase localised load capacity or for crack control purposes, structural design and reinforcement detailing should be in accordance with Eurocode 2[27] and EN 13670[30].

Steel fabric and reinforcement bar should be supplied in accordance with a recognised quality scheme such as UK CARES[31].

In areas where restraint or other factors are less than ideal, such as around dock levellers, there is greater likelihood of crack formation. In these areas, an additional top layer of fabric or bar reinforcement should be considered so as to limit crack widths and to limit crack propagation.

Fabric with flying ends should be used to avoid fabric build-up at laps.

Cover to reinforcement

Spacers and chairs for supporting reinforcement are manufactured from concrete, plastic or steel. Spacers and chairs should be in accordance with BS 7973-1[32] and their fixing and application with BS 7973-2[33].

Where wire-guided vehicles are to operate on the floor, fabric or bar reinforcement must be fixed at least 50mm (typically 75mm for 16mm diameter bar) below the surface to avoid interference with control signals.

Fabric should be installed as follows.

- Adequate numbers of spacers/chairs should be provided to support the fabric so that it does not deform during construction operations.
- For A142 and A193 fabric, the maximum distance between spacers should be 800mm (4No. 200mm fabric 'squares').
- For A252 and A393 fabric, the maximum distance between spacers should be 1000mm (5No. 200mm fabric 'squares').
- Continuous concrete block spacers may form crack inducers under the fabric and should not be used.
- Spacers should not prevent the penetration and compaction of the concrete.
- Spacers and chairs should be designed to prevent puncturing of the membrane or sinking into the sub-base.

The bottom nominal cover to the reinforcement is typically 50mm with an allowance for deviation Δc_{dev} equal to 10mm, i.e. minimum of 40 + Δc_{dev} mm. The top cover will primarily be dictated by the depth of sawn joint, depth of wire guidance or exposure class requirement, otherwise a nominal cover of 15 + Δc_{dev} mm.

6.2.2 Steel fibres and macro-synthetic fibres

Steel fibres and macro-synthetic fibres, in accordance with EN 14889 *Fibres for concrete*[34], provide post-cracking or residual moment capacity – see Section 6.3.3.

For CE marking, the fibre supplier is required to declare the quantity of fibres to achieve residual (post-cracking) flexural strength f_R of 1.5N/mm^2 at a crack mouth opening displacement (CMOD) of 0.5mm (0.47mm central deflection) and of 1.0N/mm^2 at a CMOD of 3.5mm (3.02mm central deflection). This requirement equates to a ratio of cracked to uncracked moment resistance of 30–35%, which is less than the requirements for the design in accordance with this guidance. As with conventional steel reinforcement, fibres do not generally increase the flexural tensile strength of plain concrete, as the concrete must crack before any reinforcement can have effect.

The effects of long-term creep of macro-synthetic fibres are thought to be significant and need to be considered.

For further information refer to Concrete Society technical reports TR63[35] and TR65 [36].

Fibre quantity assessment

The following procedure should be used in conjunction with fibre stock control and the recording of the number of bags/boxes added to each load to ensure that the correct quantity of fibres has been used.

Table 6.1: Strength properties for concrete.

Symbol	Property	Strength class						Units	Explanation
		C25/30	C28/35	C30/37	C32/40	C35/45	C40/50		
f_{ck}	Characteristic compressive strength (cylinder)	25	28	30	32	35	40	N/mm^2	Cylinder strength
f_{cu}	Characteristic compressive strength (cube)	30	35	37	40	45	50	N/mm^2	Cube strength
f_{cm}	Mean compressive strength (cylinder)	33	36	38	40	43	48	N/mm^2	$f_{ck} + 8$
f_{ctm}	Mean axial tensile strength	2.6	2.8	2.9	3.0	3.2	3.5	N/mm^2	$0.3\, f_{ck}^{(0.67)}$ note 1
E_{cm}	Secant modulus of elasticity	31	32	33	33	34	35	kN/mm^2	$22\,(f_{cm}/10)^{0.3}$

Note 1: For concrete strength class above C50/60 the expression for determining f_{ctm} is:
$f_{ctm} = 2.12 \ln[1 + (f_{cm}/10)]$

The test procedure for steel fibre content and homogeneity is in accordance with EN 14721[37] *Test method for metallic fibre concrete. Measuring the fibre content in fresh and hardened concrete* and for macro-synthetic fibre content EN 14488-7 [38] *Testing sprayed concrete. Fibre content of fibre reinforced concrete*. In both Standards, three samples are taken from the first, middle and last third of the load and individually tested. It is recommended that the sample container capacity is 10 litres. Test compliance criteria are given in Table 6.2.

In the light of the variability of fibre distribution, it may be necessary to specify a target fibre dosage that is higher than the required design value.

Table 6.2: Identity criteria for fibre content of fresh concrete [39].

Application	Criterion
Every sample	≥ 0.80 of specified minimum value
Average of three samples from a load	≥ 0.85 of specified minimum value

6.2.3 Micro-synthetic fibres

Micro-synthetic fibres do not provide any post-crack ductility. They do not control cracking of the hardened concrete and therefore cannot be used in lieu of other reinforcement. They are not considered in the design process.

6.3 Moment capacity

6.3.1 Plain concrete

The moment capacity of plain concrete per unit width of slab is given by:

$$M_{un} = f_{ctd,fl}\,(h^2 / 6) \qquad \text{Equation (2)}$$

where $f_{ctd,fl}$ = design concrete flexural tensile strength.

Note that neither fabric nor fibres increase the cracking moment, so where the required design limit is the onset of cracking, the moment capacity should be derived on the basis of plain (unreinforced) concrete. This applies in particular to the negative (hogging) moment in ground-supported slabs.

6.3.2 Fabric-reinforced concrete

The moment capacity M_{pfab} per unit width of slab is calculated from:

$$M_{pfab} = 0.95\, A_s f_{yk}\, d / \gamma_m \qquad \text{Equation (3)}$$

where A_s = area of steel
f_{yk} = characteristic strength of steel
d = effective depth.

The coefficient 0.95 is adopted as the reinforcement content is small enough to make a specific calculation of the lever arm unnecessary.

In ground-supported slabs, Equation 3 only applies to slabs reinforced with fabric located near the bottom surface. For ground-supported slabs reinforced with fabric located near the top surface only, the slab should be considered as unreinforced.

6.3.3 Steel and macro-synthetic fibre-reinforced concrete

EN 14889-1 *Fibres for concrete,* Part 1: *Steel fibres* [34] requires the effect of the fibre on the strength of the concrete to be determined in accordance with EN 14845[40] using a standard notched beam test in EN 14651 *Test method for metallic fibre concrete. Measuring the flexural tensile strength* [4].

Specimens 150mm wide × 150mm deep are tested under central point loading on a span of 500mm. The specimens are notched with a saw cut 25mm deep in a side face as cast, and then tested with the notch in the tension face. Either the crack mouth opening displacement (CMOD) (i.e. the increase in width of the notch) or the central deflection is measured, and the load *f* recorded at CMODs of 0.5, 1.5, 2.5 and 3.5mm (or deflections of 0.47, 1.32, 2.17 and 3.02mm). A test set should consist of at least 12 samples.

A typical graph of applied load F_R against CMOD is shown as Figure 6.1.

Note that this graph indicates the behaviour of a typical fibre reinforced concrete, exhibiting strain softening. Peak load (F_L) is achieved at the point the concrete section cracks, and thereafter the capacity of the section reduces as strain / crack width increases. F_1 is lower than F_L and F_4 is lower than F_1. Certain combinations of fibre type and dosage can exhibit strain hardening behaviour. Strain hardening is identified in a notched beam test where F_1 is equal to or greater than F_L and F_4 is greater than F_1.

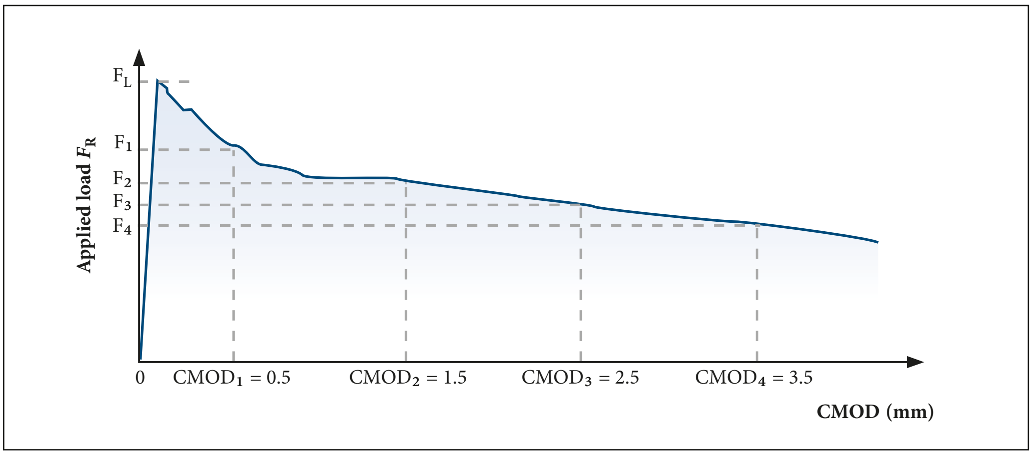


Figure 6.1: Typical graph of test load F_R vs CMOD.

Each load is used to derive a 'residual flexural tensile strength' f_R.

$$f_R = 3\,F_R\,\ell\,/\,(2b\,h_{sp}^{\,2}) \qquad \text{Equation. (4)}$$

where F_R = applied load at stage R
ℓ = the span (500mm)
b = the width (150mm)
h_{sp} = depth of the section to the tip of the notch (125mm).

The four values f_{R1}, f_{R2}, f_{R3}, f_{R4} are reported for each of the 12 samples.

The mean value of each is used.

The specimen concrete should have material constituents similar to those to be used in the floor. Where possible, the actual fibre dosage should be tested. Where this is not possible, results may be interpolated between the results of tests at a higher and lower dosage than that required; however, the range between these two dosages used should not be greater than 10kg/m^3. Results must not be extrapolated, i.e. to obtain values for a higher dosage than actually tested. Nor should the results from one fibre type be used to predict the results from another fibre type.

6.3.4 Calculation of residual moment capacity from notched beam tests

The method is explained in RILEM document TC 162-TDF (2002)[41] The mean axial tensile strengths for each of two crack widths are considered. These are σ_{r1} and σ_{r4} corresponding to CMOD 0.5mm and 3.5mm. The crack depths are taken to be 0.66 and 0.90 of the beam depth.

The following formulae are derived:

$$\sigma_{r1} = 0.45 f_{R1}$$
$$\sigma_{r4} = 0.37 f_{R4}$$

where f_{R1} = the residual flexural strength at CMOD 0.5
f_{R4} = the residual flexural strength at CMOD 3.5.

In the floor section, at ultimate limit state (ULS), it is assumed that the axial tensile strength at the tip of the crack is σ_{r1} and at the tension face (the opening of the crack) it is assumed to be σ_{r4} with a triangular distribution between the two points as shown in Figure 6.2.

6.3.5 Moment capacity calculation methods

The methods used throughout this report are intended for statically indeterminate structures only.

Historically, steel fibre concrete used in floors has exhibited reducing tensile stress in the fibre concrete as strain increases (so called 'strain softening'). More recently, fibres have been developed that exhibit strain hardening.

The ultimate moment capacity for a fibre-only slab is calculated, as is the case for traditionally reinforced sections, on the basis that failure occurs when the extreme compressive strain in the concrete reaches a limiting value of 0.0035.

Rigorous assessment of ultimate moment capacities require iterative calculations to be undertaken to determine the neutral axis depth at which strain compatibility and equilibrium of compressive and tensile forces in the section is achieved. Because of the elastic/plastic relationship between concrete compressive stress and strain, the calculation is complex. The rigorous assessments for both strain softening and strain hardening and are given in Appendix C.

For strain softening types, the following simplified method can be used.

Fibre reinforcement only

A conservative approximation of the ultimate moment capacity can be calculated by making the following simplifying assumptions;

- At the ultimate moment of the section, the concrete reaches its limiting compressive strain simultaneously with the fibre concrete reaching its limiting tensile strain. Strain compatibility is achieved, but equilibrium is not achieved, as the compressive force in the concrete will always exceed the tensile force in the fibre concrete.
- The neutral axis (NA) depth is thus a constant multiple of the section depth.

Based on these assumptions, the ultimate moment capacity per m width is calculated as follows:

$$N \geq T_{2.1} + T_{2.2}$$
$$N = 0.123h \times 0.75 \times 0.85 \times f_{ck}$$
$$N = 0.078h \times f_{ck}$$
$$T_{2.1} = 0.88h\sigma_{r4}$$
$$T_{2.2} = 0.44h(\sigma_{r1} - \sigma_{r4})$$

Taking moments about the centroid of compression zone N:

$$M_u = \left[T_{2.1}\left(\frac{0.877h}{2} + 0.075h\right) + T_{2.2}\left(\frac{0.877h}{3} + 0.075h\right)\right] \Big/ \gamma_m \quad \text{Equation (5)}$$

This can be simplified to:

$$M_u = \frac{h^2}{\gamma_m}(0.29\sigma_{r4} + 0.16\sigma_{r1}) \quad \text{Equation(6)}$$

Fibre reinforcement with steel bar reinforcement where A_s < 0.15% of gross cross-sectional area.

Where a fibre slab also includes reinforcement which will act in the tensile zone, provided A_s <0.15% of the gross cross-sectional area, the moment capacity can be calculated as follows with reference to Figure 6.3. The same simplifying assumptions are made as for the 'fibre-only' section.

$$N \geq T_1 + T_{2.1} + T_{2.2}$$
$$N = 0.078h \times f_{ck}$$
$$T_{2.1} = 0.88h\sigma_{r4}$$
$$T_{2.2} = 0.44h(\sigma_{r1} - \sigma_{r4})$$
$$T_1 = A_s E_s \varepsilon_s \leq A_s f_{yk}$$

Taking moments about the centroid of compression zone N:

$$M_u = \frac{T_{2.1}}{\gamma_m}\left(\frac{0.877h}{2} + 0.075h\right) + \frac{T_{2.2}}{\gamma_m}\left(\frac{0.877h}{3} + 0.075h\right) + \frac{T_1}{\gamma_s}(d - 0.048h)$$

Equation (7)

For low percentage of reinforcement the fabric reinforcement will yield, therefore the simplified equation is:

$$M_u = \frac{h^2}{\gamma_m}(0.29\sigma_{r4} + 0.16\sigma_{r1}) + \frac{A_s f_y (d - 0.048h)}{\gamma_s} \quad \text{Equation (8)}$$

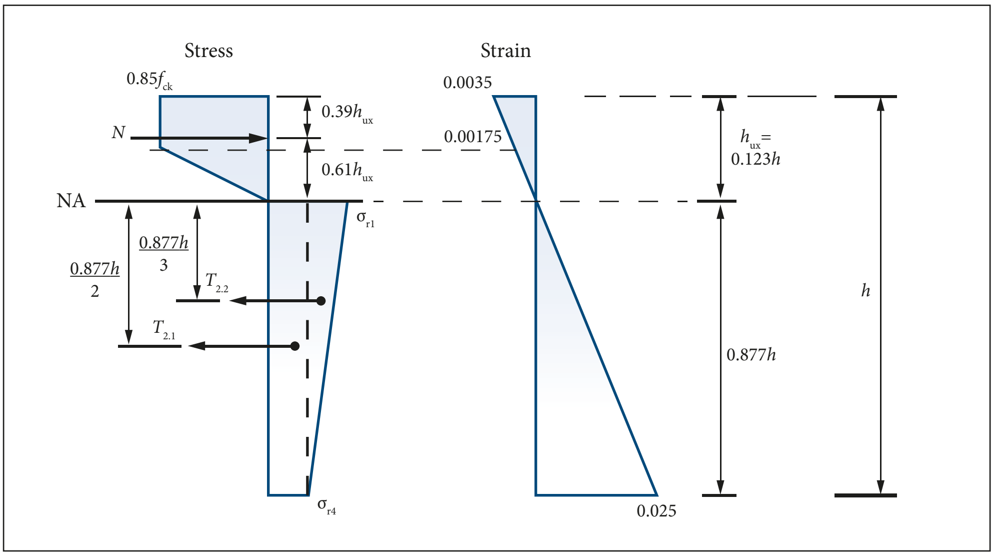


Figure 6.2: Stress block; fibre-reinforced concrete.

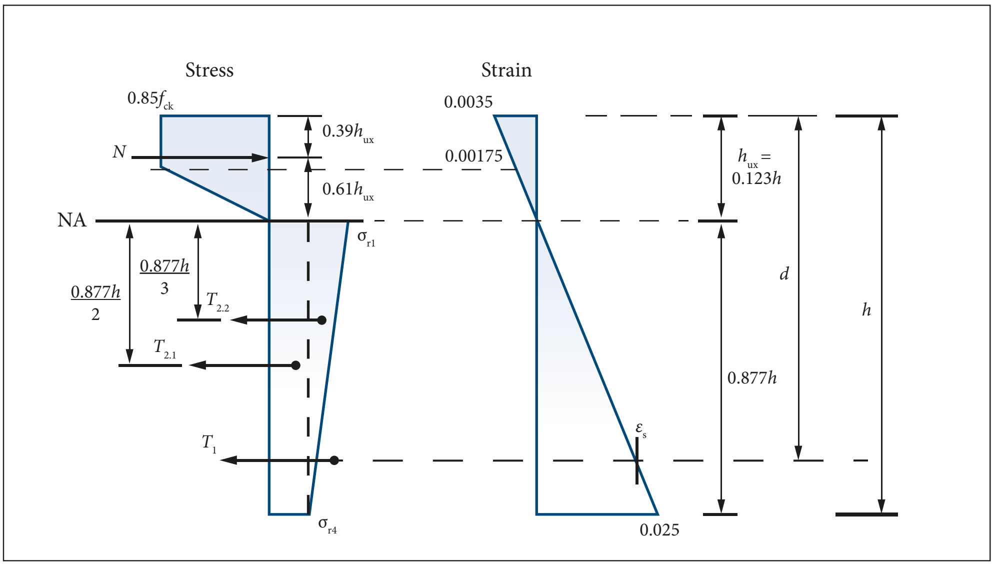


Figure 6.3: Stress block; fibre and steel bar reinforced concrete (A_s < 0.15% of gross cross-sectional area).

Fibre reinforcement with steel bar reinforcement where $A_s \geq 0.15\%$ of gross cross-sectional area

Where fibres and bar reinforcement are combined in a section and the area of reinforcement (A_s) is ≥0.15% of the gross cross-sectional area it is potentially unsafe to make the simplifying assumptions used for the 'fibre-only' and 'fibre plus fabric' sections. The neutral axis depth needs to be assessed based on equilibrium of compressive and tensile forces. In order to reduce the calculation effort, the extreme fibre tensile stress is assumed to be σ_{r4}. In reality this stress will vary between σ_{r4} and σ_{r1}, dependent on the extreme fibre tensile strain. This assumption is slightly conservative, but as the contribution of the fibres is typically relatively small in 'fibre plus bar reinforcement' sections, and reduces as the area of reinforcement increases, it is considered acceptable. See Figure 6.4

The moment capacity can be calculated as follows:

$$N = T_1 + T_{2.1} + T_{2.2}$$
$$N = 0.75h_{ux} \times 0.85 f_{ck}$$
$$T_{2.1} = (h - h_{ux})(\sigma_{r4})$$
$$T_{2.2} = 0.5(h - h_{ux})(\sigma_{r1} - \sigma_{r4})$$
$$T_1 = A_s E_s \varepsilon_s \leq A_s f_{yk}$$
$$T_1 = A_s E_s \left(\frac{d - h_{ux}}{h_{ux}}\right) 0.0035$$

Assuming the steel reinforcement yields, h_{ux} can be found using equilibrium

$$0.64h_{ux} f_{ck} = (h - h_{ux})[\sigma_{r4} + 0.5(\sigma_{r1} - \sigma_{r4})] + A_s f_{yk} \qquad \text{Equation (9)}$$

Taking moments about the centre of compression 'N'

$$M_u = [0.5(\sigma_{r1} - \sigma_{r4})(h - h_{ux})(0.28h_{ux} + 0.33h)]/\gamma_m$$
$$+ [\sigma_{r4} (h - h_{ux})(0.11h_{ux} + 0.5h)]/\gamma_m$$
$$+ [A_s f_{yk}(d - 0.39h_{ux})]/\gamma_s \qquad \text{Equation (10)}$$

To ensure sufficient ductility is available for yield line analysis to be safe, Equation 10 is only valid if $h_{ux} < 0.3d$.

Iteration is required to calculate h_{ux} for a particular quantity of bar reinforcement A_s, then calculate M_u. If necessary, repeat with higher/lower values of A_s until required M_u achieved.

6.4 Punching shear

Punching shear capacity is determined in accordance with Eurocode 2 [27] by checking the shear at the face of the contact area and at the critical perimeter distance 2.0d (where d is the effective depth) from the face of the contact area. Generally, the latter will control load capacity.

Eurocode 2[27] is written on the basis of conventional bar (or fabric) reinforcement and hence does not define an effective depth for fibre-reinforced or unreinforced concrete slabs. The effective depth for a fibre-reinforced or unreinforced slab should be taken as 0.75h, where h is the overall depth.

6.4.1 Shear at the face of the loaded area

In accordance with Eurocode 2[27], irrespective of the amount of any reinforcement in the slab, the shear stress at the face of the contact area should not exceed a value v_{max} given by:

$$v_{max} = 0.5k_2 f_{cd}$$

where f_{cd} = design concrete cylinder compressive strength $= f_{ck} / \gamma_c$

$k_2 = 0.6\,(1 - f_{ck} / 250)$ (Eurocode 2 [27] uses the symbol 'v'.)

where f_{ck} = characteristic concrete cylinder compressive strength.

Hence, maximum load capacity in punching, $P_{p,max}$, is given by:

$$P_{p,max} = v_{max}\, u_0 d \qquad \text{Equation (11)}$$

where u_0 = length of the perimeter of the loaded area based on the effective dimensions of the baseplate as described in Section 7.8.1.

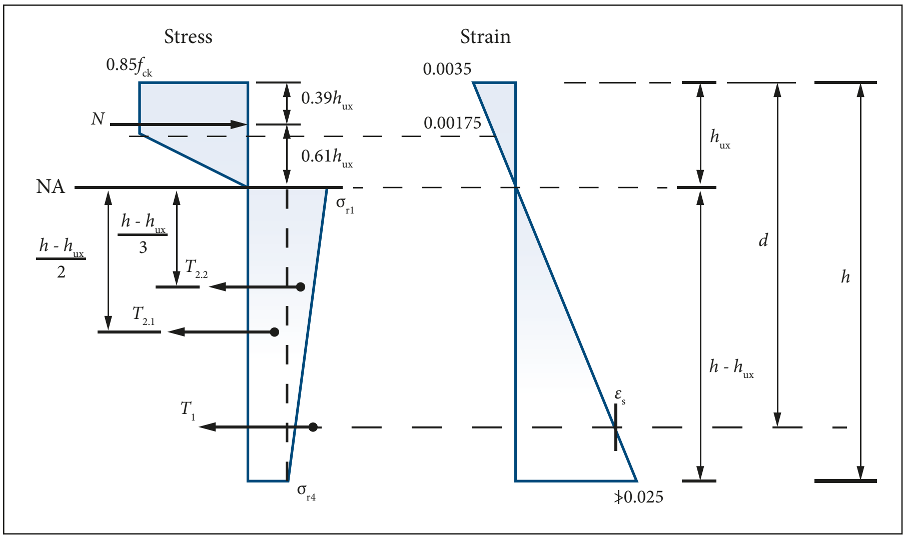


Figure 6.4: Stress block; fibre and steel bar reinforcement ($A_s \geq 0.15\%$ of gross cross-sectional area).

6.4.2 Shear on the critical perimeter

Unreinforced concrete

The minimum shear strength of concrete can be taken from Expression 6.3N in Eurocode 2[27] as:

$$v_{Rd,c,min} = 0.035\, k_s^{1.5} f_{ck}^{0.5} \qquad \text{Equation (12)}$$

where $k_s = 1 + (200/d)^{0.5}$ (Eurocode 2[27] uses k but k_s is used to avoid confusion with the modulus of subgrade reaction.)

d = effective depth.
$k_s \leq 2.0$ (see Eurocode 2[27] clause 6.2.2).

The shear stress is checked on the critical shear perimeter at a distance $2d$ from the face of the contact area.

Fabric or steel bar reinforcement

The average shear stress that can be carried by the concrete on the shear perimeter, $v_{Rd,c}$, is given by:

$$v_{Rd,c} = \frac{0.18 k_s}{\gamma_c}(100\rho_1 f_{ck})^{0.33} \geq 0.035 k_s^{1.5} f_{ck}^{0.5}$$

where $\rho_1 = \sqrt{(\rho_x \rho_y)}$
ρ_x, ρ_y = ratio of reinforcement in the x- and y-directions
$\rho_x = A_{sx}/bd \qquad \rho_y = A_{sy}/bd$
$k_s = 1 + (200/d)^{0.5} \leq 2$

Thus the slab load capacity, P_p, is given by:

$$P_p = v_{Rd,c}\, u_1 d \qquad \text{Equation (13)}$$

where u_1 = length of the perimeter at a distance $2d$ from the loaded area. For baseplates these should be based on the effective dimensions of the baseplate as described in Section 7.8.1.

Steel fibre and macro-synthetic fibre reinforcement

RILEM guidance[42] suggests that the presence of steel fibres will increase the shear capacity of a concrete section, although it states that this will be the case only in the presence of conventional reinforcement. Similar results are indicated by other researchers[43, 44]. Although some papers on steel-fibre-only slabs have suggested an increase in punching shear capacity, the results have largely been qualitative and have generally been based on higher fibre contents than would be expected in floors.

In the absence of verifiable relevant research papers, it is proposed that the RILEM guidance should continue to be followed but with a reduction of 50% in the applied RILEM values irrespective of the presence of conventional reinforcement.

RILEM[42] suggests that the increase in shear capacity is 0.12 times the residual flexural strength, where the mean flexural strength is taken from a load deflection plot up to a deflection of 3mm. This deflection is equivalent to the CMOD of 3.5mm of the notched beams as determined from EN 14651[4].

For this report, only 50% of this value is taken and this is applied to the mean of f_{r1} to f_{r4}. Thus, the increase in shear strength is given by:

$$v_f = [0.12\,(f_{r1} + f_{r2} + f_{r3} + f_{r4})/4]/2$$
$$= 0.015(f_{r1} + f_{r2} + f_{r3} + f_{r4}) \qquad \text{Equation (14)}$$

Thus the slab load capacity, P_p, is given by:

$$P_p = (v_{Rd,c} + v_f)\, \mu_1 d \qquad \text{Equation (15)}$$

where u_1 = length of the perimeter at a distance $2d$ from the loaded area. For baseplates these should be based on the effective dimensions of the baseplate as described in Section 7.8.1.

There are no data available to demonstrate that shear capacity enhancement is provided by macro-synthetic fibres and therefore no enhancement should be assumed.

6.5 Dowel capacities

The following derivation of dowel load transfer equations is given in Appendix F.

6.5.1 Conventional bar dowels and fabric

Dowels in accordance with EN 13877-3:2004 *Concrete pavements. Specifications for dowels to be used in concrete pavements*[45] are short lengths of smooth steel of either round, square or rectangular section used at free-movement joints to enable loads to be transferred from one side of the joint to the other with no significant differential deflection. One end is cast into the side of the slab cast first, while the other end is debonded so that the joint can open horizontally with no relative vertical movement.

The **shear capacity** per dowel is given by:

$$P_{sh\,dowel} = 0.6 f_{yd} A_v \qquad \text{Equation (16)}$$

where $f_{yd} = f_{yk}/\gamma_s$ from Appendix D
A_v = shear area, taken as 0.9 × area of the section ($\pi d_d^2/4$ for round dowels and d_d^2 for square bars)
γ_s = partial safety factor for steel, taken as 1.15.

The **bearing/bending** capacity per dowel, P_{bear}, is given by:

$$P_{max\,dowel} = d_d^2 (f_{cd} f_{yd})^{0.5}[(1 + \alpha^2)^{0.5} - \alpha] \qquad \text{Equation (17)}$$

where d_d = diameter of round dowel or width of a square bar
f_{cd} = concrete design compressive cylinder strength
$= f_{ck}/\gamma_c$
$f_{yd} = f_{yk}/\gamma_s$ from Appendix D
$\alpha = 3e[(f_{cd}/f_{yd})^{0.5}]/d_d$ from Appendix D, Equation D6
e = distance of application of load from face of concrete; by symmetry this equals half of the joint opening (see Appendix D, Figure D1).

This formula is based on a round bar section and gives a conservative result for square sections.

6.5.2 Plate dowels

Discrete plate dowels are commonly used as alternatives to traditional bar dowels. These are not to be confused with continuous plate dowels which have been found to perform poorly in service and are not recommended.

The following calculations are based on a constant plate cross-section. For other plate shapes, the manufacturer should adopt appropriate dimensions for the width of the plate in the opening of the joint and the area and shape of the plate within the slab on each side of the joint.

The **shear capacity** per dowel is given by:

$$P_{sh\ plate} = A \times 0.9 \times 0.6 \times p_y \qquad \text{Equation (18)}$$

where A = cross-sectional area of plate

p_y = plate steel design yield strength.

The **bearing/bending capacity** per dowel is given by:

$$P_{max\ plate} = 0.5[(b_1^2 + c_1)^{0.5} - b_1] \qquad \text{Equation (19)}$$

where b_1 = $2ek_3\, f_{cd}\, P_b$

c_1 = $2k_3 f_{cd}\, P_b^{\ 2} t_p^{\ 2} f_{yd}$

e = distance of application of load from face of concrete; by symmetry this equals half of the joint opening (see Appendix D, Figure D1)

k_3 = 3, a constant determined empirically (see Appendix D)

f_{cd} = concrete design compressive cylinder strength

= f_{ck}/γ_c

p_b = plate width

t_p = plate thickness

f_{yd} = characteristic strength of steel plate.

6.5.3 Bursting forces

The risk of conventional or plate dowels bursting/punching out of the concrete has been considered. In most floors, the principal movement joints consist of discrete plate dowels, often incorporated as part of a combined permanent formwork and joint armouring system.

The distinction between discrete or individual plate dowels as opposed to continuous plate systems needs to be emphasised as the latter have a poor record of service and are not recommended.

In TR34 3rd edition[2], a calculation method based on Eurocode 2[27] was adopted.

It is now recognised that there is considerable eccentricity in the application of the loads, as the loads on the dowels are applied to the side of the dowel-to-concrete interface. At present there is no accepted method of modifying the Eurocode 2[27] method to take account of this eccentricity. However, there is now considerable experience of the use of discrete plate dowels with little, if any, record of failure.

Limited laboratory testing[46] has indicated that bursting could occur at lower loads than indicated by the TR34 3rd edition method which used the actual depth below the plate and the face of the slab in the Eurocode calculation.

It is therefore recommended that the Eurocode method should continue to be used but using the more conservative effective depth of 0.75 times the plate depth between the dowel/plate and the surface of the slab.

The loaded length for conventional bar dowels should be taken as not greater than 8 times the bar diameter.

Where the dowel spacing is such that the critical shear perimeters overlap, the shear capacity of the slab along a perimeter encompassing the loaded dowels must be checked.

6.5.4 Effect of steel and macro-synthetic fibres on bursting forces

Shear enhancement associated with fibre-reinforced concrete should not be relied on in the vicinity of dowels [47, 48] and is therefore not taken into account in calculating load transfer at joints.

7 Structural design of ground-supported slabs

This section provides guidance on the structural design of ground-supported slabs. The slab is fully supported by the ground and it is assumed that there is no access below the slab, either on completion or in the future, so the slab remains fully supported.

The scheme designer is advised to agree with the checking authority that this design method is appropriate for providing foundation support to mezzanines.

7.1 Introduction

The primary design objectives are to carry the intended loads and to avoid surface cracking.

For the commonly found point loads from storage racking, mezzanines and materials handling equipment (MHE), two ultimate strength modes of failure are possible: flexure and punching.

Slab design for flexure under point loads at the ultimate limit state (ULS) is based on yield line theory, which requires adequate ductility to assume plastic behaviour. Clearly, there is a requirement for sufficient rotation capacity of the sagging yield lines so that the hogging moment capacity is mobilised.

At the ULS, the bending moment along the sagging (positive moment) yield lines is assumed to be the full plastic (or residual post-cracking) value. However, as a principal serviceability requirement is the avoidance of cracks on the upper surface, the bending moment of the slab along the hogging yield lines is limited to the design cracking moment of the concrete, although with the partial safety factor appropriate to the ULS. This is not a true ULS as the floor will not have collapsed and the design process is in reality meeting a serviceability requirement. It follows that there are no separate design checks for serviceability.

The design against punching shear of the slab around concentrated loads is based on the approach in Eurocode 2[27] for suspended slabs. Allowance is made for the fact that some of the load will be transferred directly through the slab to the ground.

Line loads and uniformly distributed loads are evaluated using an elastic analysis with reference to Hetenyi's *Beams on elastic foundation*[49].

The recommended minimum design thickness for a ground-supported slab is 150mm.

The designer should take account of the reduction of thickness caused by mat wells, induction loops, guide wires and other features.

Most floors have joints as there are practical limits on how much concrete can be placed in one day. In most cases, the critical loading case is a point load close to a joint between slab panels. Hence the load-carrying capacity of the floor alongside joints must be checked in all designs. This capacity will depend significantly on the ability of the joint mechanism to transfer load to the other side of the joint. This is particularly the case for MHE, which, unlike static loads, cannot be positioned away from joints. The joint mechanism can consist of the fabric reinforcement in the slab, bar or plate dowels and aggregate interlock.

Floors are subjected to stress from both loads and potentially from restraint to drying shrinkage. This combination can cause cracking. Realistic assessment of the combined effects of load-induced stresses and shrinkage is problematical and could produce conservative designs without significantly reducing the risk of cracking.

The approach taken in this report for ground-supported floors is therefore to not take into account the effect of shrinkage-induced stresses and to minimise shrinkage by careful attention to concrete mix design and to minimise restraint to shrinkage by careful attention to sub-base design and construction, the use of slip membranes and limiting distances between joints, and by not tying the floor to walls, columns or other fixed elements.

Using this approach it has been shown that for fabric-reinforced ground floors with sawn joints, in the order of 6m apart, there is a very low risk of in-panel cracking induced by drying shrinkage. For floors with fewer joints at greater distances, in what are commonly referred to as 'jointless' floors, the theoretical risk of cracking increases as a result of greater restraint to movement from the sub-base. Limiting the distance between joints to the order of 35m may reduce that risk. This also provides the benefit of limiting joint openings.

The risk of cracking of heavily and/or early loaded jointless floors will be significantly increased in wide aisle formations of racking or block stacking.

In the case of fibre-reinforced slabs it has been found that once cracked to full depth, such cracks may open further, resulting in the progressive reduction of load transfer capacity and in the worst case the formation of a free edge. Load capacity is then reduced significantly and deflections increase, particularly under the actions of materials handling equipment (MHE). This situation can be difficult to remedy by repair where a crack becomes a dominant movement joint. The possibility of unplanned cracks should therefore be taken into account in the design of jointless floors.

Sawn joints in fibre reinforced floors are at risk of opening to an extent where load transfer capability is progressively lost under the dynamic actions of MHE, see Section 4.3. This may result in significant deflections, cracking and joint arris damage. Vertical movement at joints can lead to sub-base compression and loss of slab support. It is therefore recommended that sawn joints should be avoided in fibre reinforced floors unless additional load transfer measures are used.

7.2 Partial safety factors

The partial safety factors used in ground-supported floors are as follows.

Materials

Concrete	1.5
Concrete with fibre	1.5
Reinforcement (bar or fabric)	1.15

Loads

Defined racking	1.2
Other	1.5
Dynamic loads	1.6

The partial safety factor of 1.6 for dynamic loads allows for the braking and cornering effects as well as the normal allowance for the uncertainty of the magnitude of the load.

Where very heavy MHE is in use, fatigue effects need to be considered – see Section 7.3.

For UDLs and line loads, a global safety factor of 1.5 is used. As a partial safety factor of 1.5 is applied to the material properties, a partial safety factor of 1 should be applied to the UDL or the line load.

Where a mezzanine is supported by a slab then the partial safety factor for the mezzanine structure dead load should be taken as 1.35 and for any imposed loads on the mezzanine structure taken as 1.5.

7.3 Fatigue effects of heavy dynamic loads

TR34 3rd edition[2] has been shown to be well calibrated for most warehouse and distribution centre ground-supported floor slabs supporting conventional pallet racking and the associated MHE. However, it is now considered that where very heavy MHE is in use, fatigue effects need to be considered. Typically, this will arise where heavy counterbalance trucks are used for applications such as double pallet handling, paper reel handling with clamps and loads in heavy engineering works. A method for checking the slab thickness required to resist fatigue effects in ground-bearing slabs is described in Appendix E.

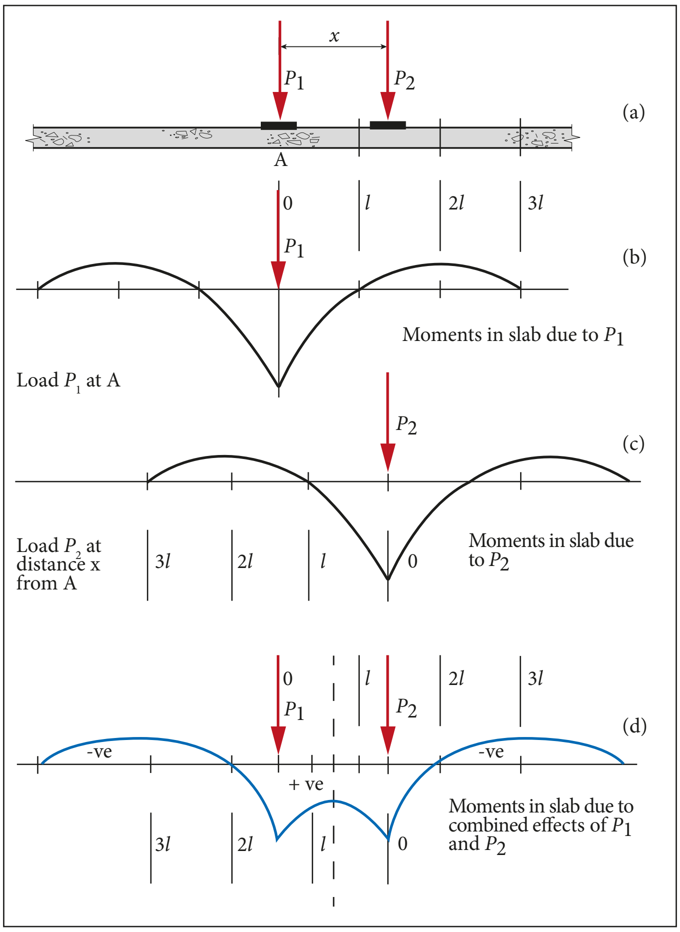


Figure 7.1: Schematic of distribution of elastic bending moments for internal loads, a) typical load case, b) for load P_1 c) for load P_2 and d) for the combined loads P_1 and P_2.

7.4 Reinforcement requirements

The reinforcement content should be such that the ratio of cracked to uncracked factored moments of resistance is not less than 50%[50].

The moment capacity of a steel-fibre-only, or steel fibre combined with fabric or bar reinforcement should be calculated as described in Section 6.3. For fabric reinforcement it is recommended that the reinforcement cross-sectional area (A_s) should be at least 0.08%, with an upper limit of 0.125% across sawn restrained movement joints. The fabric should be in the bottom of the slab and should be installed on spacers to provide sufficient nominal cover as described in Section 6.2.1.

7.5 Radius of relative stiffness

Westergaard [24, 25] introduced the concept of the radius of relative stiffness *l* which is determined as:

$$l = [(E_{cm} h^3) / (12 (1 - v^2) k)]^{0.25} \qquad \text{Equation (20)}$$

where

E_{cm} = short-term modulus of elasticity of the concrete (N/mm^2)
h = slab thickness (mm)
k = modulus of subgrade reaction (N/mm^2/mm, taken as N/mm^3)
v = Poisson's ratio, taken as 0.2.

The physical significance of *l* is discussed below and by reference to Figure 7.1.

7.6 Bending moments for internal point loads

The bending moment under a concentrated load P_1 is at a maximum and positive (tension at the bottom of the slab) directly under the load. As the distance from the load increases, the circumferential moment remains positive and decreases to zero at a distance 1.0 *l* from the load. It then becomes negative and is at its maximum at 2.0 *l* from the load. The maximum negative moment (tension on the top of the slab) is significantly less than the maximum positive moment. The moment approaches zero at 3.0 *l* from the load.

The influence of an additional load P_2 at any distance x from A is as follows:

- If $x < l$, the positive bending moment at A will increase.
- If $l < x < 3l$, the positive bending moment at A will decrease, but by a relatively small amount.
- If $x > 3l$, the additional load will have negligible influence on the positive bending moment at A.
- If $2l > x < 6l$, the additional load will increase the negative bending moment.

It is also useful to examine how the factors included in Equation 20 will influence the value of l.

- In Eurocode 2[27], Poisson's ratio for concrete is taken as 0.2. Thus $(1 - v^2) = 0.96$ and has little influence on the value of l.
- The modulus of elasticity of concrete (short term) may be obtained from Eurocode 2[27] as shown in Table 6.1. Therefore l increases with E_{cm}.
- The smaller the value of k (i.e. the more compressible the soil), the higher the value of l.
- The value of l will increase with increase in the slab depth h.

Figure 7.2 shows the case of a single point load applied internally over a small circular area on a large concrete ground-supported slab. As the load increases, the flexural stresses below the load will become equal to the flexural strength of the concrete. The slab will begin to yield, leading to radial tension cracks in the bottom of the slab caused by positive tangential moments.

With further increases in load, it is assumed that the moments are redistributed and there is no further increase in positive moment but a substantial increase in circumferential moment some distance away from the loaded area. Tensile cracking will occur in the top of the slab when the maximum negative circumferential moment exceeds the negative moment capacity of the slab (i.e. as a plain concrete section). When this condition is reached, failure is considered to have occurred as the design criterion is to avoid surface cracks.

In 1962, Meyerhof [51] used an ultimate strength analysis of slabs based on plastic analysis (yield line theory) and obtained design formulae for single internal, edge and corner loads. He also considered combinations of two and four loads.

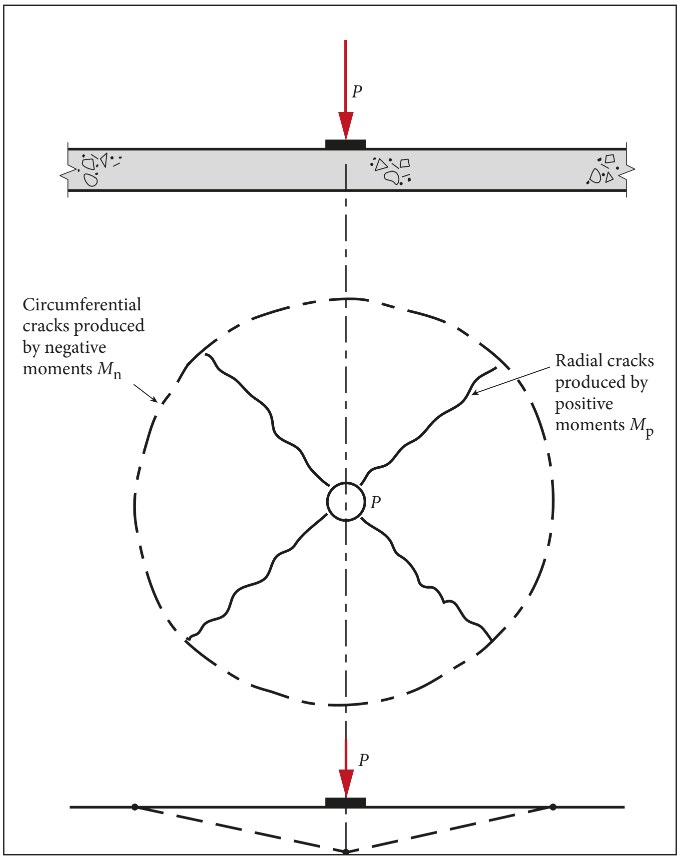


Figure 7.2: Development of radial and circumferential cracks in a concrete ground-supported slab.

For each location, a pair of equations is given to estimate the capacity (P_u) of ground-supported slabs subjected to a single concentrated load – see Equations 21 to 30. The first equation of each pair is for a theoretical point load, i.e. with $a = 0$, where a = equivalent radius of contact area of the load. The second is for a 'patch' load and is valid for $a/l \geq 0.2$. Meyerhof is not explicit in dealing with values of a/l between 0 and 0.2. However, test results reported by Beckett [52] and by Beckett *et al.*[53] have shown that reasonable agreement between theoretical and test values is obtained by linear interpolation between values of a/l between 0 and 0.2.

7.7 Load locations

Three load locations (see Figure 7.3) are considered in design as follows:

Internal – the centre of the load is located more than $(a + l)$ from an edge (i.e. a free edge or a joint).

Edge – the centre of the load is located immediately adjacent to a free edge or joint more than $(a + l)$ from a corner (i.e. a free corner, the intersection of a free edge and a joint, or the intersection of two joints).

Corner – the centre of the load is located a from each of the two edges or joints forming a corner.

where a = equivalent radius of contact area of the load
l = radius of relative stiffness. See Equation 20.

It should be noted that loads at edges adjacent to joints are considered in the same way as those at true edges to be found at, for example, the perimeter of a building. However, effective loads at joints are reduced by load transfer through aggregate interlock and or dowels – see Section 7.9.

Although the theoretical load capacity at a true corner, as found at the perimeter of a building, is much lower than at a true edge, experience has shown that the actual capacity at a joint intersection appears to be as great as that at a joint, provided that there are the same conditions of joint opening and provision of dowels. It is therefore generally not necessary to consider potential loads at intersections provided that appropriate design considerations are applied to the single joints in the floor.

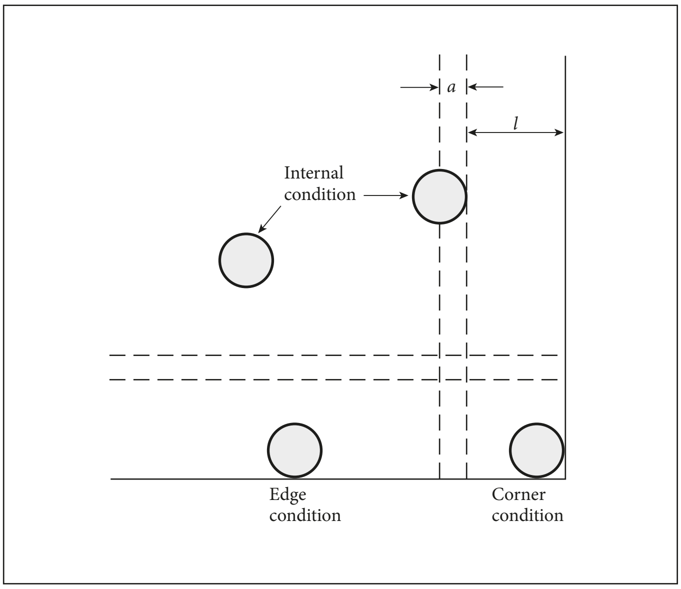


Figure 7.3: Definitions of loading locations.

7.8 Point loads

7.8.1 Single point loads

In order to calculate the stresses imposed by a load it is necessary to know the size of the load and the radius of the contact area, a. As baseplates and the footprints of truck wheels are generally rectangular, the actual contact area is first established, from which the radius of the equivalent circle (i.e. with the same area) is calculated. In the absence of contact area details for pneumatic wheel loads, the contact area can be calculated using the load and the tyre pressure. For other types of wheel, the manufacturer should be consulted for information on the load and contact area.

The dimensions of any baseplates should only be taken as the area which is sufficiently stiff to transfer the load to the slab. Unless a larger area can be justified by appropriate analysis, taking account of the relative stiffness of the slab and baseplate, the baseplate dimensions should be taken as the lesser of the actual dimensions and the effective dimensions calculated in accordance with Figure 7.4.

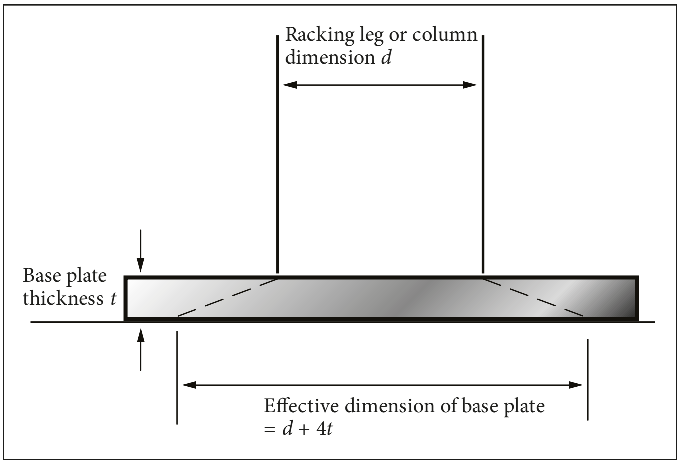


Figure 7.4: Calculation of effective dimension of baseplate.

In the absence of project-specific detail for adjustable pallet racking, an effective dimension of 100mm × 100mm should be used.

7.8.2 Closely spaced point loads

Where point loads are in close proximity, they can be considered to act jointly as a single load on a contact area that is equivalent to the individual loads expressed as circles plus the area between them, as shown in Figure 7.5. This will, for example, apply to back-to-back racking uprights which are typically 250–350mm apart. This method may be used for pairs of loads at centres up to twice the slab depth. Otherwise the combined behaviour should be determined from Equations 27 and 28.

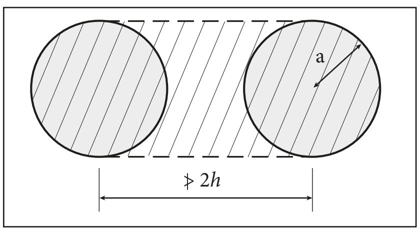


Figure 7.5: Calculation of equivalent contact area for two adjacent point loads.

This can also apply to combinations of forklift wheels and racking uprights when picking or placing pallets. In these positions, the load-side front wheel is often carrying the maximum load of the forklift. A typical layout for very narrow aisles is shown in Figure 7.6. Note that a more onerous condition could occur when dimension H is at a minimum when the truck is passing the racking upright with the carried load centrally positioned.

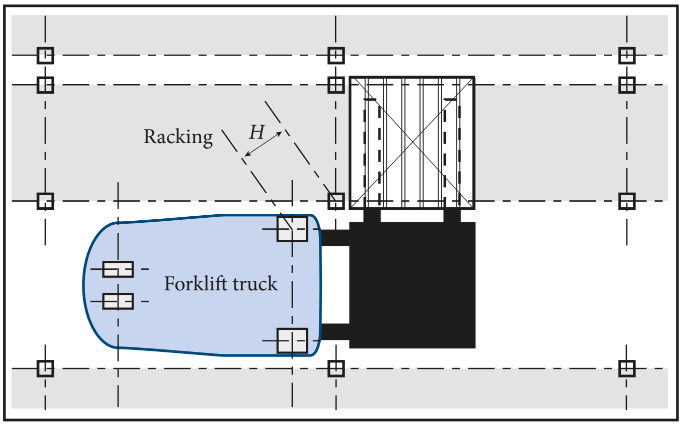


Figure 7.6: Adjacent point loads in very narrow aisles.

7.8.3 Design equations for single point loads

The following equations for internal loads (Equations 21 and 22), edge loads (Equations 23 and 24), and corner loads (Equations 25 and 26), are taken from Meyerhof [51].

Interpolate for values of a/l between 0 and 0.2.

For an **internal load** with:

$a/l = 0$:

$$P_{u,0} = 2\pi\,(M_p + M_n) \qquad \text{Equation (21)}$$

$a/l \geq 0.2$:

$$P_{u,0.2} = 4\pi\,(M_p + M_n)\,/\,[1 - (a/3l)] \qquad \text{Equation (22)}$$

For a free **edge load** with:

$a/l = 0$

$$P_{u,0} = [\pi\,(M_p + M_n)/2] + 2M_n \qquad \text{Equation (23)}$$

$a/l \geq 0.2$:

$$P_{u,0.2} = [\pi\,(M_p + M_n) + 4\,M_n]/\left[1 - \frac{2a}{3l}\right] \qquad \text{Equation (24)}$$

For a free **corner load** with:

$a/l = 0$:

$$P_{u,0} = 2M_n \qquad \text{Equation (25)}$$

$a/l \geq 0.2$:

$$P_{u,0.2} = 4M_n\,/\,[1 - (a/l)] \qquad \text{Equation (26)}$$

where M_n = negative (hogging) resistance moment of the slab (kNm), taken to be that of the plain unreinforced concrete – see section 6.3
M_p = ultimate positive (sagging) resistance moment of the slab (kNm), taken to be that of the reinforced concrete – See section 6.3.

Although it may be possible to position static loads away from joints, this is unlikely to be the case with dynamic loads such as MHE. The slab should therefore be checked for static and dynamic loads where applicable at joints.

7.8.4 Design equations for multiple point loads

The following equations should be used for multiple **internal loads**.

For **dual point loads**, where the centre-line spacing x is less than $2h$ (twice the slab depth), use the simplified approach given above. Otherwise, the total failure load approximates to the following:

Interpolate for values of a/l between 0 and 0.2.

For $a/l = 0$:

$$P_{u,0} = [2\pi + (1.8\,x/l)][M_p + M_n] \qquad \text{Equation (27)}$$

For $a/l \geq 0.2$:

$$P_{u,0.2} = \left[\frac{4\pi}{1-(a/3l)} + \frac{1.8x}{l-(a/2)}\right][M_p + M_n] \qquad \text{Equation (28)}$$

As the spacing of the dual point loads increases, the total failure load approaches the upper limit given by the sum of the separate failure loads obtained from Equations 21 and 22.

Meyerhof did not provide equations for dual point loads at an edge. Where dual point loads are found near an edge and where it is inappropriate to use the simplified approach, the internal load can be factored down by the ratio of the edge to internal load for a single point load.

For **quadruple point loads** with centreline spacing of x and y, the total failure load is given by the sum of the failure loads of the individual point loads (Equations 21 and 22) or by the sum of the failure loads of the individual dual concentrated loads or by the following approximate total failure load, whichever gives the smaller value:

For $a/l = 0$:

$$P_{u,0} = \left[2\pi + \frac{1.8\,(x+y)}{l}\right][M_p + M_n] \qquad \text{Equation (29)}$$

For $a/l \geq 0.2$:

$$P_{u,0.2} = \left[\frac{4\pi}{1-(a/3l)} + \frac{1.8(x+y)}{l-(a/2)}\right][M_p + M_n] \qquad \text{Equation (30)}$$

The failure patterns resulting from these load arrangements are illustrated in Figure 7.7.

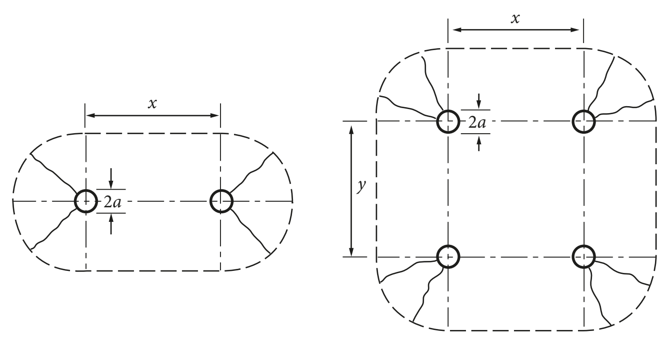


Figure 7.7: Failure patterns for multiple point loads.

It will be found that the failure load of a group of loads will be smaller than for the sum of the individual loads unless the loads are spaced well apart (e.g.at least $3.5l$ for two loads in line). However, experience indicates that for pallet racking in the common back to back configuration it is sufficient to check for the combined load of the two inner back to back rack uprights and for the two outer rack uprights individually in the set of four uprights comprising the two rack frames.

For mezzanines columns or other similar point loads, checks for the combined effects should be carried out where loads are closer than $3.5l$.

For dense racking such as automated storage and retrieval systems, the combined effect of the point loads should also be assessed as a uniformly distributed load in accordance with 7.12.

7.9 Load transfer at joints

True free edges or corners that are required to carry load are relatively unusual, as they generally occur only at the periphery of a building. Joints between panels and the intersections of these joints are of greater importance and therefore provision must be made to transfer load across them without causing differential vertical movement.

It is not possible to transfer more than 50% of the load across a joint.

Load transfer needs to be considered separately for the following joint types. For joint details see Section 11.

- ***Formed free-movement joints.*** Joint mechanisms consist of round or square dowels or individual plate dowels.
- ***Sawn free-movement joints.*** Debonded dowels are set into position in dowel cages. Full-depth cracks are induced by saw cuts and load transfer is provided by aggregate interlock and the dowels.
- ***Formed restrained-movement joints.*** The restraint is usually provided by lengths of reinforcing bar placed in the same way as dowels but with a full anchorage length at each side of the joint. The amount should correspond to the reinforcement in the slab, generally in the range 0.08–0.125%. Connecting dowels are typically 12mm at 450–600mm centres.
- ***Sawn restrained-movement joints (fabric-reinforced slabs only).*** Full-depth cracks are induced by saw cuts. Fabric reinforcement is continuous across the joint and load transfer is provided by aggregate interlock and the reinforcement.

7.9.1 Load transfer by aggregate interlock

Aggregate interlock is the ability of a narrow irregular crack to transfer load from one side to the other by contact between the particles of aggregate exposed when the crack forms. The effectiveness of this depends on the joint opening width, the slab thickness, the subgrade support, the load and the way it is applied, and the angularity of the aggregate. Clearly, aggregate interlock can only take place at a crack formed deliberately at a sawn restrained-movement joint or at a narrow random crack.

Based on the work of Colley and Humphrey[54], for design purposes at a 1.5mm crack opening, 15% of the capacity can be transferred across a joint. Where joints or cracks open more widely than 0.9mm in areas of heavy traffic or loading, they should be filled to reinstate aggregate interlock.

Thus the design approach is:

- Calculated edge capacity (from Equation 23) = X.
- Assume 15% load transfer, so effective edge capacity = X / (1 – 0.15) = 1.176X.
- Add in the capacity of any dowels (see Section 6.5) = Y.
- Thus total effective edge capacity = 1.176X + Y (but not greater than internal capacity from Equation 22).

7.9.2 Load transfer by dowels or bars

Calculation methods for bending, shear and bursting capacities for dowels and bars can be found in Section 6.5. For ground-bearing slabs, the effective numbers of dowels which will contribute to transferring the load are taken as those within a distance of 1.8 l either side of the centreline of the applied load, where l is the radius of relative stiffness (Yoder and Witczak [55]). The amount of load carried by each dowel is assumed to reduce linearly with distance from the centreline. This is equivalent to assuming that all the dowels within a distance of 0.9 l each side of the centreline work at their full capacity.

7.10 Punching shear capacity and ground support

7.10.1 Shear capacity of slab

As the dominant design load for industrial ground-floor slabs is point loads from racking and forklifts, punching shear needs to be considered.

Punching shear capacity is determined in accordance with Section 6.4 by checking the shear at the face of the contact area and at the critical perimeter a distance 2.0d from the face of the contact area, where d is the effective depth. See Figure 7.8.

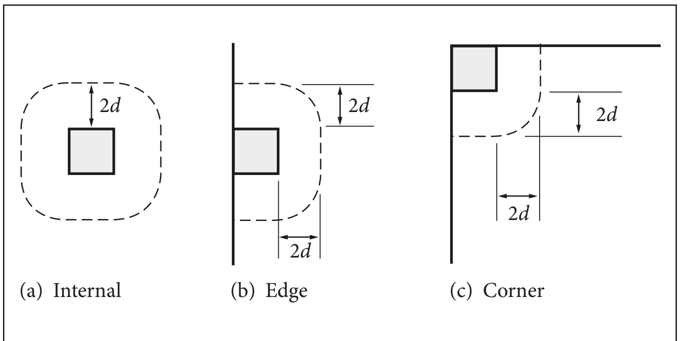


Figure 7.8: Critical perimeters for punching shear for internal, edge and corner loading.

7.10.2 Ground support

As the slab is assumed to be in contact with the sub-base, a proportion of the load within the punching shear perimeter can be considered to be applied directly to the subgrade, thus reducing the design force. A method for calculating the ground reaction is set out below.

For point loads applied through a stiff bearing (where a/l < 0.2), the reaction is:

Internal

$$R_p = 1.4\left(\frac{d}{l}\right)^2 P + 0.47(x+y)\frac{dP}{l^2} \qquad \text{Equation (31)}$$

Edge

$$R_{cp} = 2.4\left(\frac{d}{l}\right)^2 P + 0.8(2y+x)\frac{dP}{l^2} \qquad \text{Equation (32)}$$

where P is the point load
d is the effective depth
x and y are the effective dimensions of the bearing plate – see Section 7.8.1. For the edge condition x is the dimension parallel to the edge
l is the radius of relative stiffness.

Note: where dimensions x and y result in an 'equivalent radius of contact area' $a > 0.2l$, the effective dimensions of the baseplate should be reduced such that a is not greater than $0.2l$.

The total reaction can then be deducted from the imposed shear load.

The full derivation for the above expressions can be found in Appendix F.

7.11 Line loads

The elastic analysis based on the work of Hetenyi [49] is adopted. This analysis has traditionally used a global safety factor of 1.5. As a factor of 1.5 is already applied to the material properties, an additional factor should not be applied to the load. The equations for determining moments in ground-supported slabs incorporate the term λ where:

$$\lambda = \left(\frac{3k}{E_{cm} h^3}\right)^{0.25} \qquad \text{Equation (33)}$$

where k = modulus of subgrade reaction (N/mm^2/mm, taken as N/mm^3)
E_{cm} = modulus of elasticity of the concrete (N/mm^2).

The factor λ is referred to as the 'characteristic' of the system and since its dimension is (length)$^{-1}$ the term $(1/\lambda)$ is referred to as the 'characteristic length'.

The working load capacity of the slab under the action of a line load per unit length, P_{lin}, is determined from:

$$P_{lin} = 4\,\lambda M_{un} \qquad \text{Equation (34)}$$

As this is based on an elastic distribution of bending moment, M_{un} should be taken as the cracking moment, i.e. the value from Equation 2. The residual moment for fibre-reinforced or fabric reinforced concrete (from Equation 4) should not be used.

Equation 34 is applicable to line loads remote from joints or slab edges. Where a line load is located adjacent to a free edge, the capacity is $3\lambda M_{un}$ increasing to $4\lambda M_{un}$ over a distance of $3/\lambda$. For a joint with a minimum load transfer capacity of 15%, the capacity increases to $4\lambda M_{un}$ at a distance of $1/\lambda$ - see Figure 7.9. This can be explained by the fact that for a line load remote from an edge, the zero moment position is at a distance of approximately $1/\lambda$ from the load, which is analogous to a joint with shear capacity but no rotational stiffness.

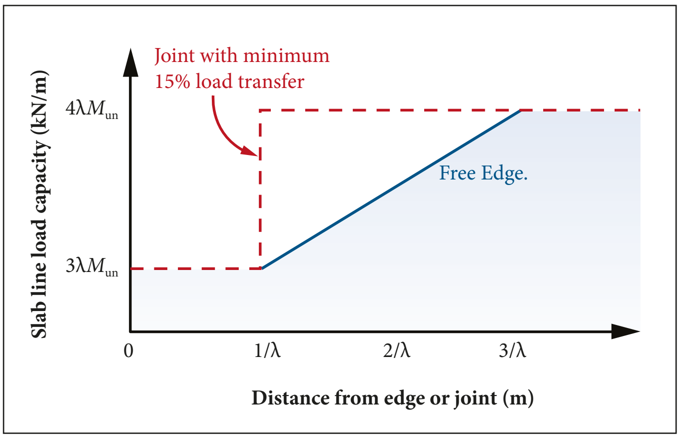


Figure 7.9: Line load capacity near free edges or joints.

7.12 Uniformly distributed loads

The elastic analysis based on the work of Hetenyi[49] is adopted. This analysis has traditionally used a global safety factor of 1.5. As a factor of 1.5 is already applied to the material properties, an additional factor should not be applied to the load. The equations for determining moments in ground-supported slabs incorporate the term λ as for line loads (Equation 33).

The equations below do not take account of UDL loads near to joints. Hetenyi[49] provides a method of analysis for loads near joints but this is very complicated. Traditionally, joints have been ignored for UDL calculations and this has not been known to result in failures. It is suggested that this should continue although designers can analyse slabs more rigorously by reference to Hetenyi[49].

A common example of uniformly distributed loading is block stacking. For the general case where the slab will be subjected to a random pattern of uniformly distributed loading, it has been found that the maximum positive (sagging) bending moment in the slab is caused by a load of breadth $\pi/2\lambda$ as shown in Figure 7.10(a).

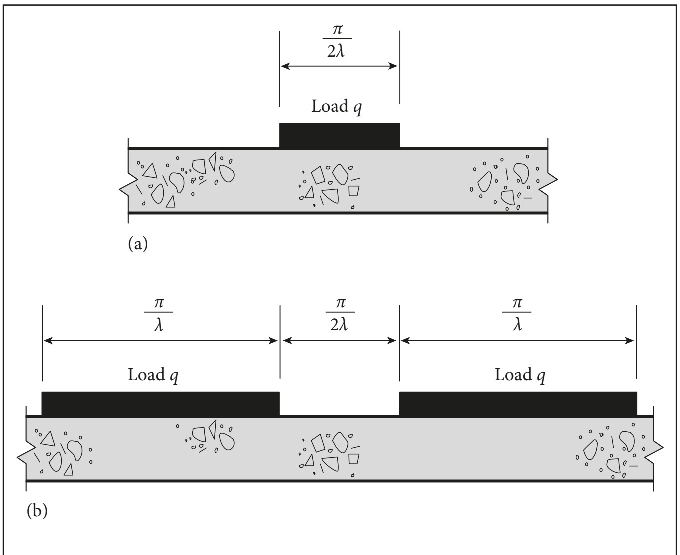


Figure 7.10: (a) Loading patterns for uniformly distributed load q causing maximum positive bending moment; (b) maximum negative bending moment.

The maximum negative (hogging) moment is induced between a pair of patch loads each of breadth π/λ spaced a distance $\pi/2\lambda$ apart, as shown in Figure 7.10(b). This spacing is commonly known as the critical aisle width.

The load capacity per unit area, q, is given by:

$$q = 5.95\,\lambda^2 M_n \quad (\text{kN/m}^2) \qquad \text{Equation (35)}$$

where M_n = moment capacity of plain concrete from Equation 2.

If the position of the loading is well defined, Hetenyi [49] has shown that the positive bending moment induced under a load of width $2c$ (shown in Figure 7.11(a)) is given by:

$$q = \frac{2}{B_{\lambda c}}\,\lambda^2 M_p \qquad \text{Equation (36)}$$

where $B_{\lambda c} = e^{-\lambda c} \sin \lambda c$
$e = 2.7182$.

At a distance a_1 from the near face and b_1 from the far face of the loaded area, see Figure 7.11(b), the induced negative moment, M_{n1}, is given by:

$$M_{n1} = \frac{1}{4\lambda^2}\,(B_{\lambda a1} - B_{\lambda b1})q \qquad \text{Equation (37)}$$

where $B_{\lambda a1} = e^{-\lambda a1} \sin \lambda a_1$
$B_{\lambda b1} = e^{-\lambda b1} \sin \lambda b_1$

If a second load is located close to the first, this will induce an additional bending moment M_{n2}, again determined from Equation 36 but with modified values of a and b. Hence q can be determined from the maximum value of $(M_{n1} + M_{n2})$, equating this to the concrete capacity M_n.

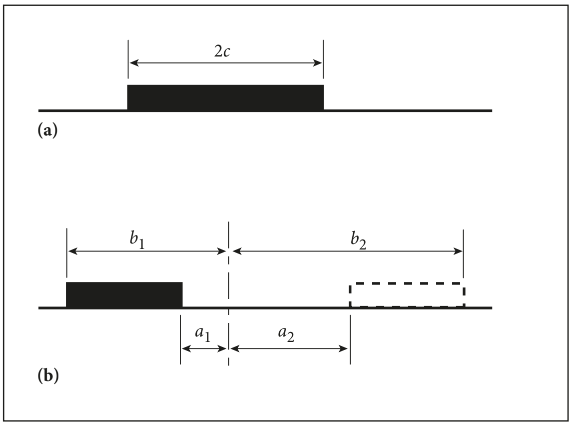


Figure 7.11(a) and (b): Defined areas of uniformly distributed load.

8 Structural design of pile-supported slabs

Pile-supported slabs are designed to be supported by the piles with no support provided by the soil. They are constructed using the soil as temporary support to the slab but it is assumed that either the soil will settle with time, to leave a void beneath the slab, or the soil stiffness is such that it effectively provides no support.

The scheme designer is advised to agree with the checking authority that this design method is appropriate for providing foundation support to mezzanines.

8.1 Introduction

It is assumed that there is no access below the slab, either on completion or in the future. In situations where such access is required or envisaged, this guidance is inappropriate and the design should be undertaken strictly in accordance with Eurocode 2 [27].

The primary design objectives are to carry the intended loads and to minimise surface cracking.

The recommended minimum design thickness for a pile-supported slab is 200mm.

The designer should take account of the reduction of thickness caused by mat wells, induction loops, guide wires and other features.

Two ultimate modes of failure are possible, namely flexure and punching under both point loads and the slab-to-pile interface.

Slab design for flexure at the ultimate limit state (ULS) is based on yield line theory, which requires adequate ductility to achieve the assumed plastic behaviour. It follows that at ultimate loads, they are in a cracked state, that is to say, the load-induced stresses are being resisted by either conventional bar reinforcement or steel fibre or combinations of the two. Sufficient moment capacity is not provided by macro-synthetic fibre-reinforced concrete.

The design against punching shear of the slab around point loads and piles is based on the approach in Eurocode 2[27] for suspended slabs.

The slab is supported on the piles but is not normally connected to them and a slip membrane is provided between the pile head and the slab soffit, so as to reduce restraint to shrinkage. Careful detailing of isolation joints around columns and other intrusions is important to allow sufficient movement of the slab.

A primary design objective is to minimise surface cracking. However, clients should be made aware that cracking is still possible in suspended slabs and the implications of such cracking on the performance of the slab should be explained. The age at which the slab is loaded is important in this regard as experience suggests that early loading of slabs is one of the major causes of cracking.

Where the client requires a completely crack-free floor, the only practical alternatives are to use an appropriately designed 'post-tensioned' slab (see Section 11.1.7), or to provide a debonded concrete screed designed as a ground-bearing slab cast on top of the suspended slab. Such solutions are not commonly adopted, as the additional cost is high compared to the cost of dealing with limited cracking.

The objective of minimising surface cracking is challenging as the greatest load-induced stresses are on the top of the slab over the piles. This is in contrast with a ground-supported slab where the greatest load stresses are on the underside of the slab. The stresses are additive to the stress induced by restraint to shrinkage, which is also at its greatest on the top surface.

For conventionally reinforced concrete structures it is possible to design with the intention of limiting crack widths. This is not recommended for piled floor slabs as excessive very fine cracking is generally more difficult to treat than more limited wider cracking. It should be noted that there is no accepted method for calculating crack widths in fibre-reinforced concrete[35].

The approach in this guidance is to limit cracking by careful attention to design and construction detailing such that cracks that do occur are not excessive in terms of extent and surface crack width and are repairable.

All practical steps should be taken to minimise shrinkage by careful attention to concrete mix design. Restraint should be reduced by careful attention to the design and construction of pile heads and sub-bases, the use of slip membranes and by not tying the floor to walls, columns or other fixed elements. Bay sizes should be limited to 35m and ideally should be as close to square as possible or if not possible, the aspect ratio should not exceed 1:1.5. The limitation on bay size also provides the benefit of limiting joint openings.

The risk of cracking is reduced by limiting the moment over the pile at service limit state (SLS) in relation to the elastic capacity of the plain concrete and by minimising shrinkage and restraint to shrinkage.

The risk of cracking is also reduced by ensuring that the hogging moment capacity over the pile is equal to or greater than the sagging moment capacity in the span in the ULS check.

8.2 Partial safety factors

The partial safety factors used in pile-supported floors are as follows.

Materials

Concrete	1.5
Concrete with fibre	1.5
Reinforcement (bar or fabric)	1.15

Dead loads

Self-weight of slab (design density of concrete 2500kg/m^3)	1.2
Permanent dead load, e.g. floor toppings/screeds	1.35

Imposed loads

Defined racking	1.2
Other	1.5
Dynamic loads	1.6

The partial safety factor of 1.6 for dynamic loads allows for the braking and cornering effects as well as the normal allowance for the uncertainty of the magnitude of the load.

Where a mezzanine is supported by a slab then the partial safety factor for the dead load from the mezzanine structure should be taken as 1.35 and for any imposed loads on the mezzanine as 1.5.

8.3 Fatigue effects of heavy dynamic loads

Where very heavy MHE is in use, fatigue effects need to be considered. Typically, this will arise where heavy counterbalance trucks are used for applications such as double pallet handling, paper reel handling with clamps and loads in heavy engineering works. Failure under the action of repeated passes of such MHE across the slab will take the form of excessive cracking, thus the fatigue design check is at the SLS, and no partial safety factor (load) is applied.

In such applications, the maximum elastic bending moment due to the unfactored MHE load should be assessed and compared with the moment capacity of the slab, reduced to take account of fatigue effects, calculated in accordance with Equation 39.

At locations where MHE movements are constrained, such that they will frequently be braking or cornering at a particular point on the slab, the static axle load should be multiplied by a dynamic enhancement factor of 1.4.

The maximum elastic bending moment can be calculated approximately using Equation 38, or alternatively may be assessed by undertaking an appropriate elastic analysis. Note that the coincident bending moments due to the slab self-weight or adjacent storage loads do not need to be added to the moments due to the MHE load when checking for fatigue effects.

Elastic moment from MHE load (kNm/m)

$$= (0.2 \times W_a \times L_{eff}) / (WB + 0.6L_{eff}) \qquad \text{Equation (38)}$$

where W_a = maximum static axle load (unfactored) (kN)
L_{eff} = effective span (m)
WB = wheelbase of loaded axle (m).

Reduced moment capacity = Moment capacity (from Section 6.3.5) × reduction factor (F)

$$F = [105 - 6.7 \log_{10}(N)] / 100 \qquad \text{Equation (39)}$$

where N = number of cycles of load application over a particular point on the slab (N not greater than 5,000,000).

Table 8.1 is derived from Equation 39.

Table 8.1: Number of cycles of load application (*N*) and reduction factor (*F*).

N	*F*
100,000	0.72
500,000	0.67
1,000,000	0.65
2,000,000	0.63
5,000,000	0.60

This fatigue-related moment capacity reduction applies to steel-fibre-reinforced slabs and to hybrid (steel fibre plus bar or fabric) slabs unless the minimum quantity of steel bar reinforcement required by Eurocode 2 is incorporated. Where the minimum bar reinforcement is incorporated, no reduction in moment capacity due to fatigue effects is required.

8.4 Reinforcement requirements

The reinforcement content should be such that the ratio of cracked to uncracked factored moments of resistance is not less than 85%.

The moment capacity of a steel fibre only, or steel fibre combined with fabric or bar reinforcement should be calculated as described in Section 6.3. For conventionally reinforced sections with no steel fibres, the moment capacity should be calculated in accordance with Eurocode 2[27].

8.5 Pile heads and effective spans

Piles are commonly enlarged to form a pile head, thus reducing the punching shear stress, reducing the peak elastic hogging moments, and shortening the effective span for yield line analysis. An added advantage is that it is easier to achieve a flat and level support (and thus minimise restraint) using an enlarged head than is the case when preparing the top of a pile. Approaches for the construction of pile heads are outlined in Section 5.7. The pile head or bearing should be designed to provide full support over the contact area.

For yield line design, the effective span of the slab is defined in Figure 8.1(a) to (d).

Also see Appendix H for optimised pile layouts.

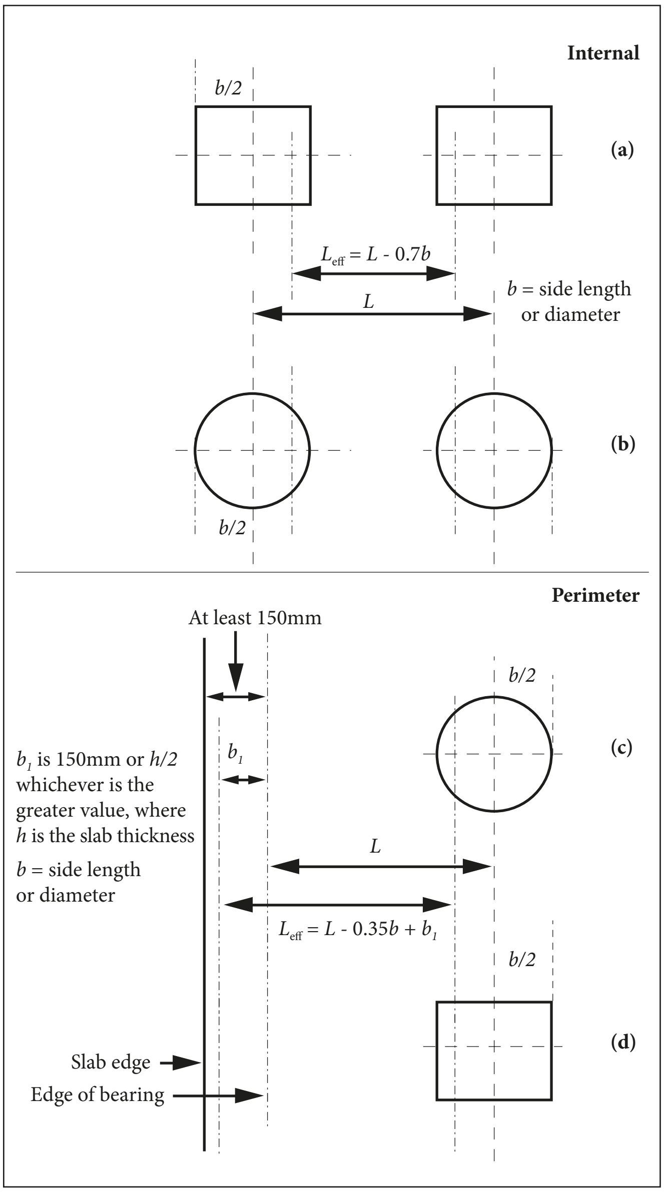


Figure 8.1: (a) and (b) Effective span for yield line design – internal; (c) and (d) effective span for yield line design – perimeter.

For rectangular grids, L is the greater pile spacing.

8.6 Design for flexure

For slabs supported on a regular rectangular or square grid of piles carrying a uniformly distributed or knife-edge load (or point loads that can be represented as a knife-edge load), yield line analysis provides a simple and quick assessment of the ultimate capacity of the slab, and is the recommended method of analysis. The process of yield line design involves identifying a pattern of yield lines that results in the critical collapse mechanism and calculating the corresponding load resistance. A full explanation of the method is available in *Kennedy and Goodchild* [56].

For a slab supported on a rectangular grid of piles, two potential yield line patterns should be considered: folded plate and fan pattern.

8.6.1 Folded plate - UDL

Yield lines are assumed to form close to the face of the piles – see Figure 8.2(a) and (b) for internal and perimeter panels respectively. The effective spans are shown in Figure 8.1(a)–(d).

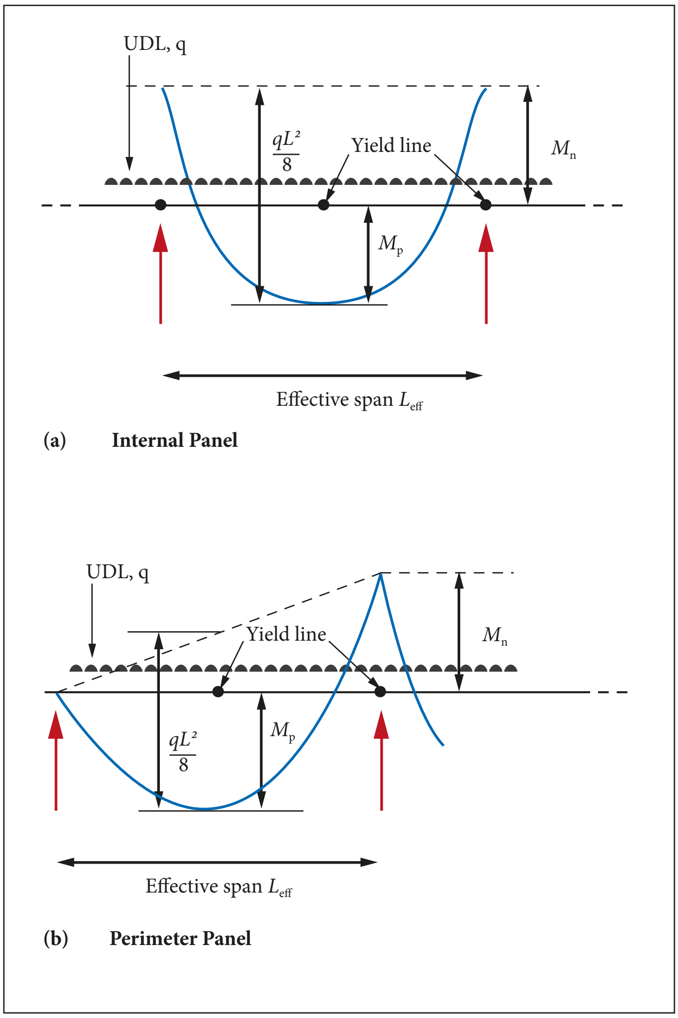


Figure 8.2: Folded plate yield line mechanism (uniformly distributed load) for (a) internal panel and (b) perimeter panel.

The collapse load for a UDL in an internal panel is given by:

$$M_p + M_n = q_u (L_{eff})^2 / 8 \qquad \text{Equation (40)}$$

where

M_p = positive moment capacity
M_n = negative moment capacity
q_u = uniformly distributed dead plus imposed load (factored)
L_{eff} = effective span.

The collapse load for a UDL in a perimeter panel is given by:

$$2M_p\{1+[1+(M_n/M_p)]^{0.5}\}^2 = q_u(L_{eff})^2 \qquad \text{Equation (41)}$$

For the particular case where $M_p = M_n$, this can be simplified to:

$$M_p + M_n = q_u (L_{eff})^2 / 5.83 \qquad \text{Equation (42)}$$

8.6.2 Folded plate – concentrated line load

Yield lines are assumed to form close to the face of the piles – see Figure 8.3(a) and (b) for internal and perimeter panels respectively. The effective spans are shown in Figures 8.1(a)–(d).

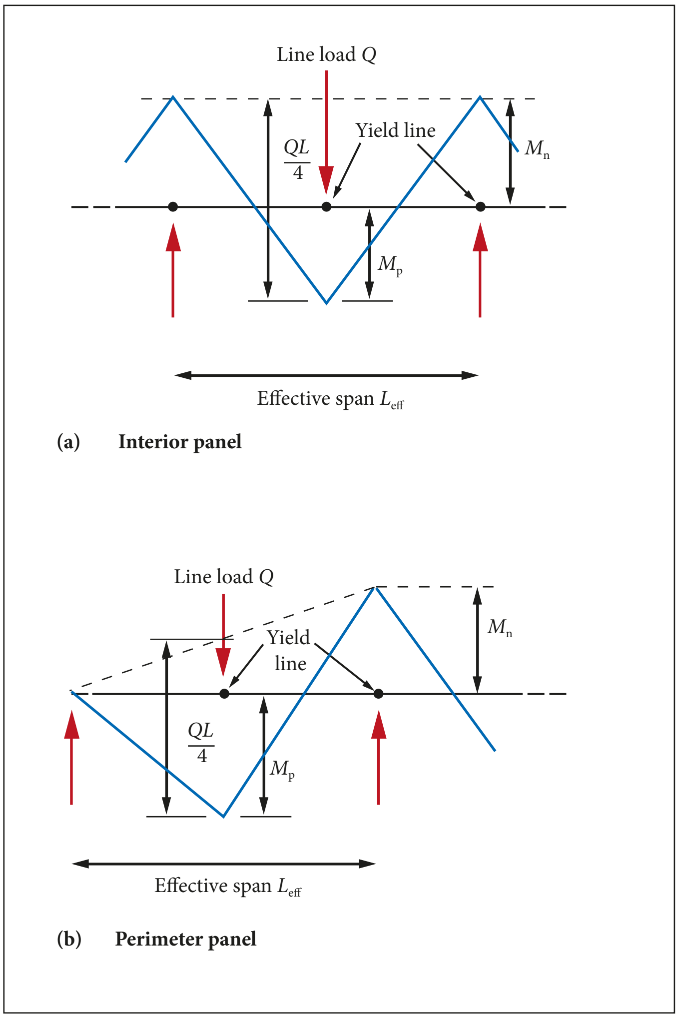


Figure 8.3: Folded plate yield line mechanism (line loads) for (a) internal panel and (b) perimeter panel.

The collapse load for a line load in an internal panel is given by:

$$M_p + M_n = Q_\ell L_{eff}/4 + q_{sw}(L_{eff})^2 / 8 \qquad \text{Equation (43)}$$

where Q_ℓ = imposed line load (factored)

q_{sw} = uniformly distributed dead load/slab self-weight (factored).

The collapse load for a line load in a perimeter panel is given by:

$$M_p + \frac{M_n}{2} = \frac{Q_\ell L_{eff}}{4} + \frac{q_{sw} L^2_{eff}}{8} \qquad \text{Equation (44)}$$

For the particular case where $M_p = M_n$ this can be simplified to:

$$M_p + M_n = Q_\ell L_{eff}/3 + q_{sw}(L_{eff})^2 /6 \qquad \text{Equation (45)}$$

Fan pattern

The second yield line pattern is a fan centred on the support pile –see Figure 8.4.

The collapse load is given by:

$$M_p + M_n = q_u L_1 L_2 \{1 - [A/(L_1 L_2)]^{0.33}\}/2\pi \qquad \text{Equation (46)}$$

Other terms are defined in Figure 8.4.

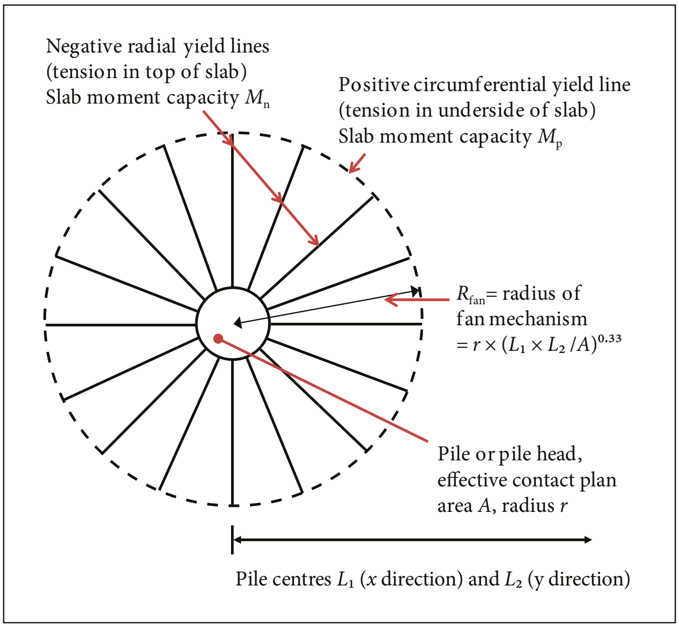


Figure 8.4: Fan yield line mechanism at pile.

The fan mechanism as shown in Figure 8.5 should also be checked at single point loads such as mezzanine columns, using Equation 47. In this case, the radial yield lines are positive and the circumferential yield line is negative, and R_{fan} is the distance from the centre of the point load to the centre of the nearest pile.

The collapse load is given by:

$$M_p + M_n = (P_u / 2\pi) + (q_u R_{fan}{}^2 / 6) \qquad \text{Equation (47)}$$

Other terms are defined in Figure 8.5.

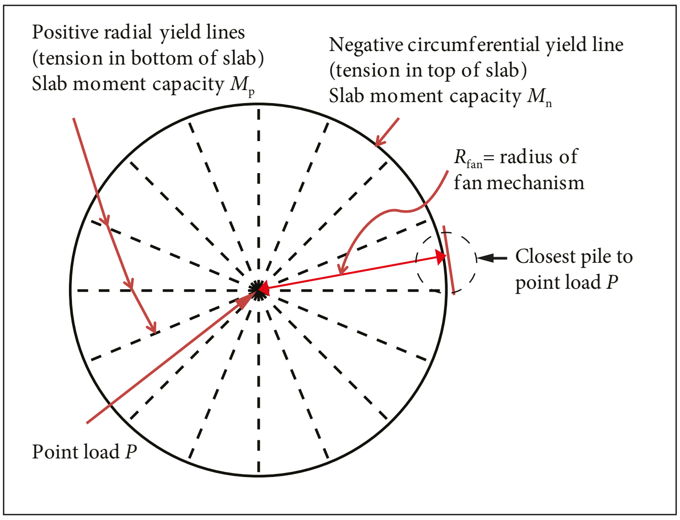


Figure 8.5: Fan yield line mechanism at point load.

At locations where a 'fan' mechanism could develop adjacent to a free edge or movement joint, the mechanism will be semicircular in shape and the capacity of the slab will be approximately half that calculated using Equation 46 or 47.

For both the pile fan mechanism and the point load fan mechanism, careful consideration is required before taking into account the effect of any supplementary reinforcement in the assessment of M_p and M_n. Only reinforcement that is adequately anchored beyond R_{fan}, the radius of the fan mechanism, should be included in the moment capacity of the slab (refer to Section 8.8 for reinforcement anchorage requirements).

For slabs with a complex or irregular arrangement of supports, unusual patterns of loading or areas of slab incorporating trenches or pits, a yield line analysis can be used but is more difficult to apply. As an alternative, finite element (FE) methods might be used. A full description of FE or yield line theory and practice is outside the scope of this report, and for more information reference should be made to Concrete Society Technical Report 64[57] or Kennedy and Goodchild[56]. The following guidance is provided on the appropriate use of FE:

- FE packages should only be used by those who understand the principles underlying the analysis and have a thorough understanding of the package they are using, especially with regard to its limitations.
- As with any analysis, it is necessary to validate the results to ensure there are no gross errors in the input or output data. Equilibrium checks on loads and reactions, and a check on the 'free' bending moment diagram, should always be undertaken.
- Simple elastic FE software cannot properly model the effects of cracking/yielding that will occur over supports. The use of non-linear FE analysis packages that model progressive yielding and redistribution is therefore preferred, although great care is needed in selecting appropriate material properties for the uncracked, cracked and yielding sections. If simple elastic FE is used, redistribution of peak moments at supports, or an iterative approach (whereby section properties at locations where the cracked moment capacity is exceeded are adjusted and the analysis repeated) will be required. The need to make such adjustments may largely negate the advantages of FE.
- The effects of pattern loading may need to be considered for ULS FE analysis of block stacking or other imposed loads, if they are likely to occur in an arrangement that will result in higher bending moments.
- For FE analysis, the actual centre-to-centre pile dimension should be used as opposed to reduced effective spans.

8.7 Punching shear

Punching shear should be checked at the pile or pile head and point loads. Punching shear capacity is determined in accordance with Section 6.4 by checking the shear at the face of the contact area and at the critical perimeter a distance 2d from the face of the contact area, where d is the effective depth.

Shear links are rarely provided in pile-supported slabs. If punching shear stresses are critical, the slab should be made thicker, top reinforcement over the pile increased or a larger pile head provided.

The critical perimeter u_1 is the length of the perimeter at a distance 2d from the face of the load contact area – see Figure 8.6.

8.8 Curtailment

Where reinforcement is used, curtailment needs to be considered. Figure 8.7 indicates anchorage requirements for reinforcement in a typical internal panel supporting a line load. The same principles apply for perimeter panels and panels carrying uniformly distributed loads. For the fan pattern, the extent of the fan is defined in Figures 8.4 and 8.5.

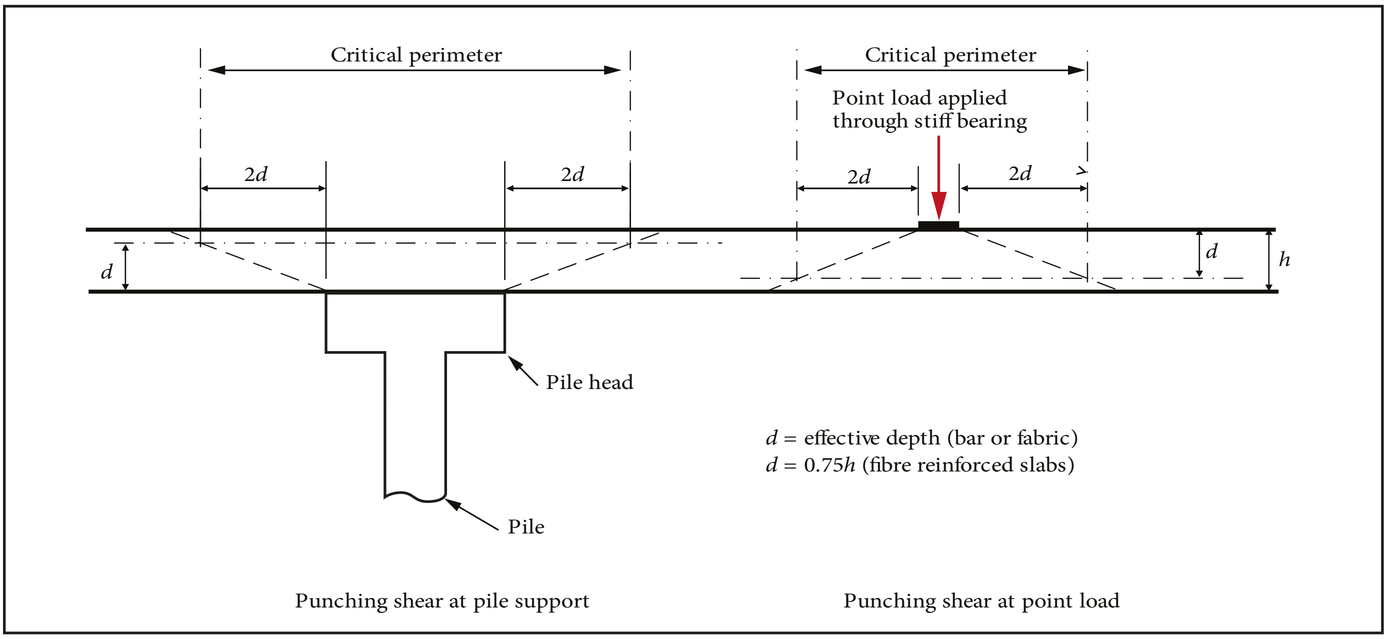


Figure 8.6: Critical perimeter for punching shear at pile support and point load.

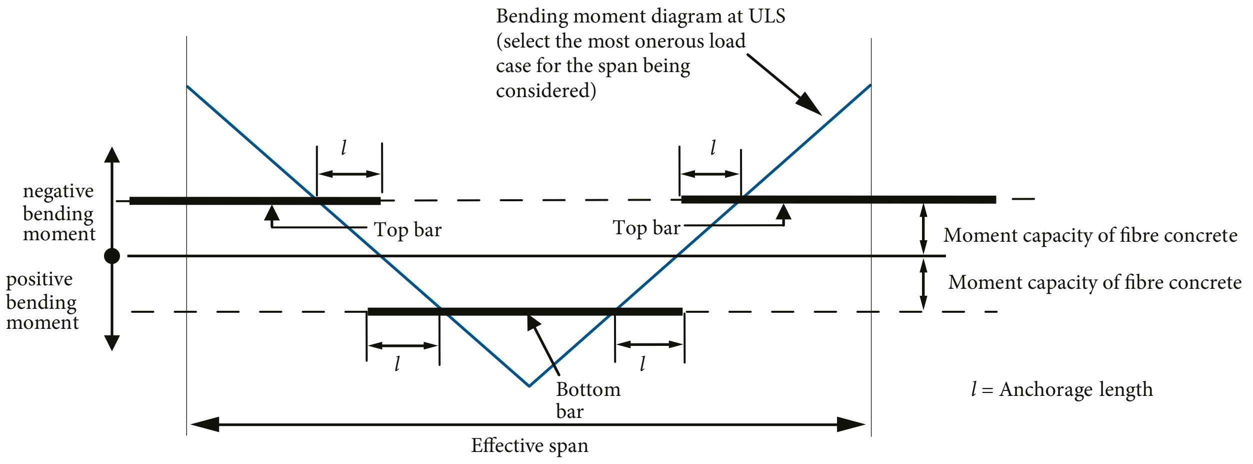


Figure 8.7: Anchorage requirements for steel bar reinforcement.

The required anchorage length may be calculated from Eurocode 2[27] or, for concrete of strength class C25/30 or greater, the following dimensions used:

For slabs up to 275mm thick

Deformed bars		Top or bottom bars, anchorage length
	=	(40 × bar diameter) + (effective depth d)
Fabric		Top or bottom bars, anchorage length
	=	(31 × bar diameter) + (effective depth d)

For slabs > 275mm thick

Deformed bars		Top bars, anchorage length
	=	(58 × bar diameter) + (effective depth d)
		Bottom bars, anchorage length
	=	(40 × bar diameter)+ (effective depth d)
Fabric		Top bars, anchorage length
	=	(44 × bar diameter) + (effective depth d)
		Bottom bars, anchorage length
	=	(31 × bar diameter) + (effective depth d)

8.9 Design load conditions

For the ultimate limit state design condition, loads should be multiplied by the appropriate partial load factors as set out in Section 8.2.

Uniformly distributed loads should include the slab self-weight, the imposed load and any additional imposed dead loads due to finishes etc. Pattern imposed loading (e.g. alternate spans loaded) is not normally considered for racking upright loads.

Concentrated loads for the yield line folded plate mechanism should be treated as a knife-edge line load across the mid-span of the panel, the line load per metre width being the total of the point loads divided by an appropriate effective spread width for the loads.

For the simple folded plate failure mechanism, the internal work yield line resisting the load extends across the full panel width between joints and therefore a point load or series of point loads can theoretically be spread over the full panel width. This may however result in unacceptable cracking as a result of elastic stress concentrations due to the point loads.

Therefore for concentrated loads, such as back-to-back racking, it is reasonable to assess the effective spread width using an appropriate elastic distribution of the load [$2.4x\ (1 - x/L)$] where x is the distance from the support to the load and L is the distance between support centres in the direction of span (refer to Figure 8.8). For a line load at mid-span, the effective spread width becomes ($0.6L$ + the load width).

For typical back-to-back racking, the spread width should therefore be restricted to:

minimum [($0.6L$ + load width) or (aisle width + load width)]

The spread width is therefore a function of the aisle width subject to the above limiting value. Therefore, VNA racking could impose a higher line load than wide aisle racking with the same upright load. For wide aisle installations, the specifier should consider the possible future change of use to VNA if full flexibility of the installation is to be accommodated.

It is important that the potential implications of such a change of use are explained to clients who may wish to make allowance for this within the slab design.

For all racking systems, allowance should be made for the MHE by applying a loaded axle width with the appropriate dynamic partial safety factor within the load spread width calculated as above. Allowance should also be made for pallets placed on the floor beneath the racking.

For single point loads, such as mezzanines, for slabs with pile spacings up to 3.5m, the spread width can be taken as L. For greater pile spacing, it is recommended that an FE analysis is undertaken to assess the bending moments in the slab due to the point load.

Note that the 'fan' yield line mechanism at the point of the load and punching shear should be checked for point loads.

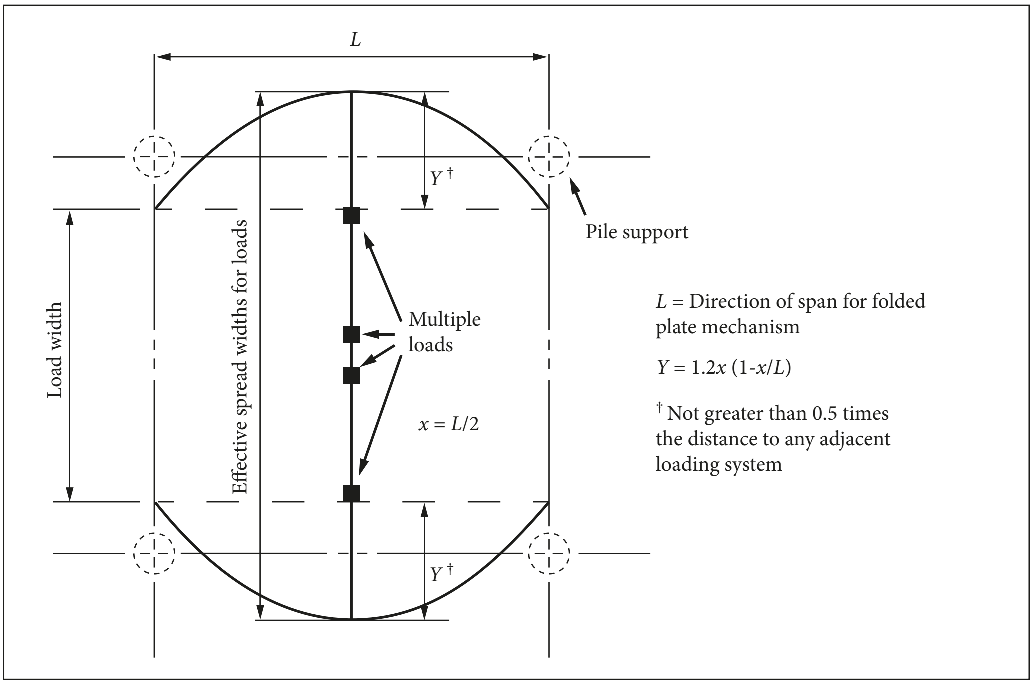


Figure 8.8: Load spread width for racking.

8.10 Construction joints

Joints in suspended slabs are typically constructed as free-movement joints formed using sleeved bar or plate dowels. The joints should be provided at no more than 35m centres in each direction. The flexural stiffness of these joints is very low relative to the stiffness of the slab and they are designed assuming they act as a 'hinge' which transfers vertical shear but no moment. The shear capacity of dowel bars or plates should be determined in accordance with Section 6.5.

The structural design of the floor slab adjacent to the joints requires special consideration. One of the following two options is typically selected for the position of the joint relative to the supporting piles:

Option 1: Locate joint on centreline of a line of piles
This location is only suitable if a pile cap is provided (or if the pile is of sufficient cross-section to provide an adequate bearing, taking account of pile installation tolerances) – see Figure 8.9. The spans either side of the joint are designed as 'end spans', resulting in locally higher positive and negative bending moments. This may dictate the slab thickness, unless supplementary reinforcement or a local reduction in the pile spacing to approximately 75% of the 'internal' pile spacing is provided.

Option 2: Locate joint offset from a line of piles
The most efficient location for the joint is at the 'point of contraflexure' (zero moment) of the slab, calculated assuming full moment continuity exists across the joint. The difficulty is that the point of contraflexure will vary with different load cases. Ideally, each load case should be considered, and the joint located at the point of contraflexure of the load case that results in the most onerous positive and negative bending moments on either side of the joint. Typically, this will result in the joint being located between ¼ and ⅙ of the span from the pile centreline. The slab panels on each side of the joint will be designed in accordance with Figure 8.10.

Note that no redistribution of the negative bending moment in the short cantilever span is possible. Structural failure will occur when this moment exceeds the negative moment capacity of the slab and a yield line forms along the line of piles closest to the joint.

Capacities of dowel bars or plates should be determined in accordance with Section 6.5.

8.11 Serviceability checks

8.11.1 Elastic flexural stresses and cracking

The risk of flexural cracking at SLS can be reduced by applying an upper limit on the elastic negative (hogging) moment over the pile in relation to the moment capacity of the plain, uncracked concrete section see Figure 8.11, calculated from Equation 2.

In addition, the sagging moment capacity in the span should not be greater than the hogging moment capacity over the pile. See Section 8.6.

For a slab supported on a regular grid of piles with at least six continuous spans, carrying a uniformly distributed load, a minimum slab thickness, h_{min}, can be calculated using Equation 48.

$$h_{min} = 21L_{eff}\,(q\,/\,f_{ctd})^{0.5} \qquad \text{Equation (48)}$$

where L_{eff} = effective span
q = uniformly distributed load, including self-weight, in kN/m^2 (unfactored)
f_{ctd} = design flexural tensile strength of the concrete, in N/mm^2 (factored).

The derivation of this equation can be found in Appendix G.

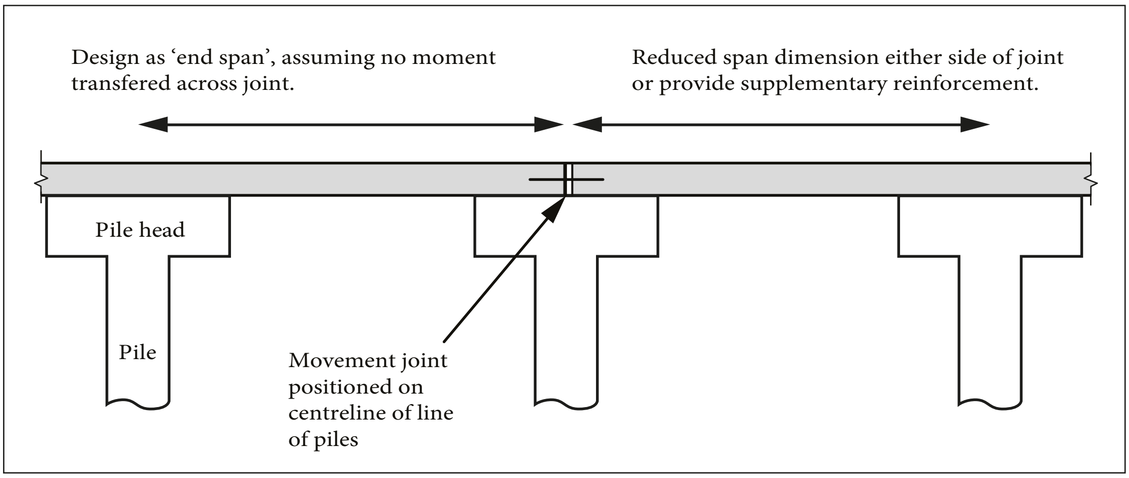


Figure 8.9: Effective span, joint on centreline of pile.

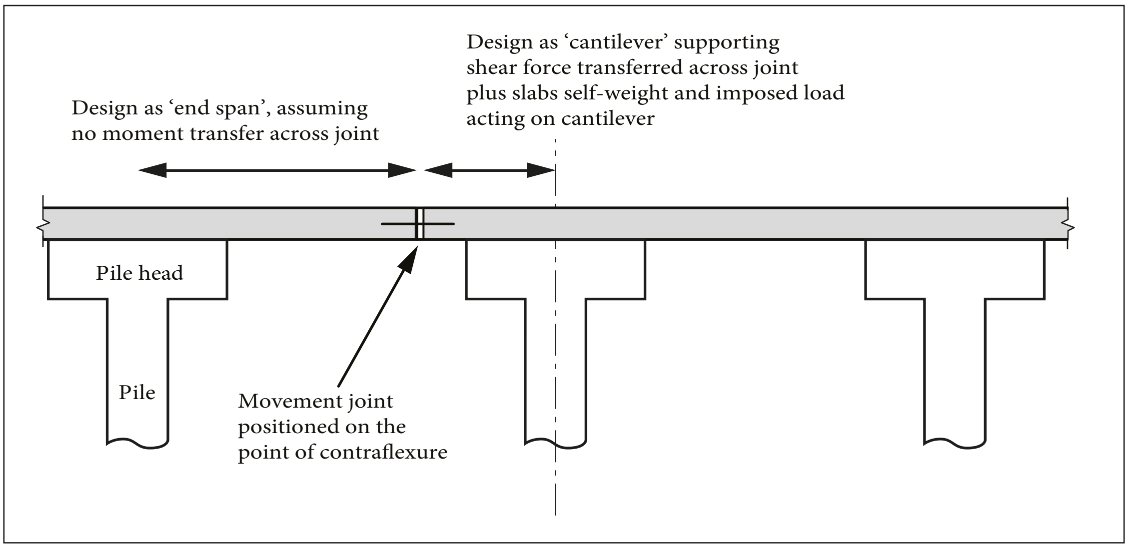


Figure 8.10: Effective span, joint offset from centreline of pile.

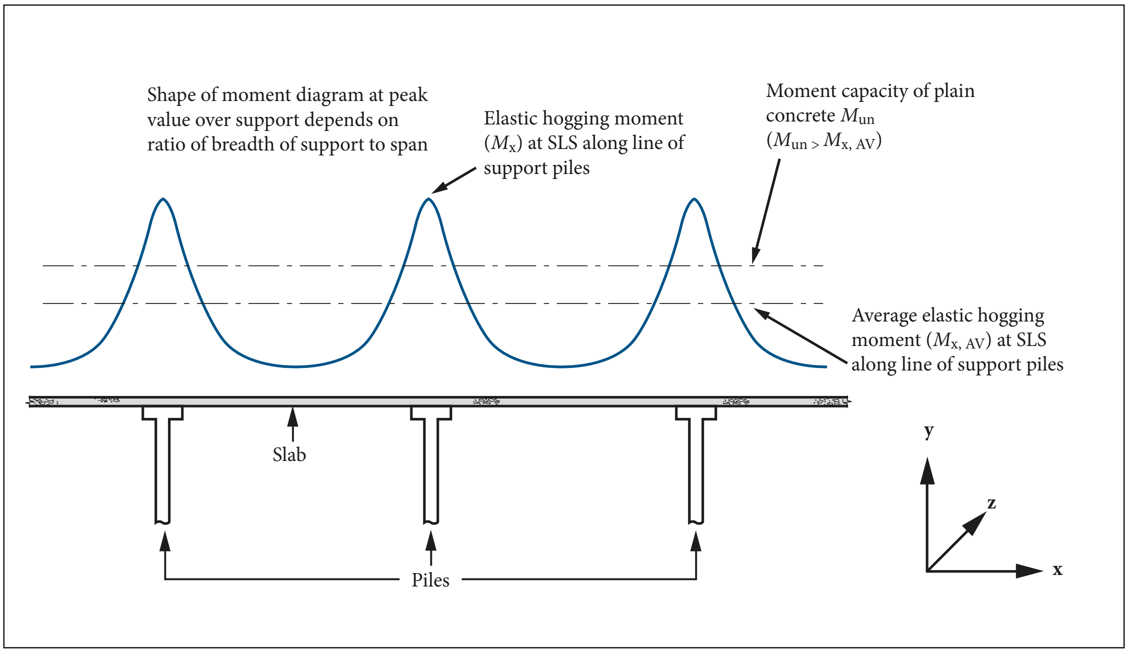


Figure 8.11: Recommended minimum M_{un}.

Equation 48 is also used to calculate the minimum slab thickness required for line loads (or point loads represented as a line load), by calculating the uniformly distributed load that would create an equivalent flexural stress:

$$q = [1.5 \times \text{line load (kN/m)}]/L_{eff} + [\text{slab self-weight (kN/m}^2)] \qquad \text{Equation (49)}$$

For more complicated arrangements of pile support or loading condition, or where there are less than six spans, an elastic analysis of a strip of slab equal to the panel width should be undertaken, using a 'continuous beam' analysis, to determine $M_{x,av}$ and therefore the minimum slab thickness required to ensure $M_{un} > M_{x,av}$. All credible arrangements of pattern loading should be included in this analysis.

Perimeter spans should be approximately 25% shorter than internal spans such that the perimeter span moment and first internal support moment are approximately the same as the span and support moment in interior panels. Alternatively, reinforcement can be added in the perimeter spans and over the first internal support piles to increase the moment of resistance.

Adding reinforcement to the bottom only of perimeter or internal spans would not accord with the serviceability requirement described above and could result in unacceptable crack widths at the surface due to excessive redistribution of support moments.

A minimum design thickness of 200mm is recommended, and pile centres should generally not exceed 3.5m.

8.11.2 Deflection and cracking

A slab designed in accordance with these recommendations will have span/depth ratios that will be much lower than the limits in Eurocode 2[27] for flat slabs and will exhibit deflections of a small order. Where deflection checks are required, FE methods are recommended.

If simple elastic FE packages are used for checking deflections, a reduction in the elastic modulus must be made to account for creep effects and reduced inertia due to cracking over supports. A modulus equivalent to ⅐ times the mean short-term elastic modulus from Table 6.1 is considered appropriate. An estimate of the elastic shortening and settlement of the piles should be incorporated in the analysis, by modelling the piles as springs in the FE model, to obtain realistic differential deflections between loaded and unloaded areas.

In most cases, explicit calculation of crack widths and spacing is not warranted, provided the formation of cracks is anticipated, planned for and accepted by the client and end-user. For hybrid 'steel fibre and bar reinforcement' slabs and 'bar reinforcement only' slabs, crack widths due to flexural stresses and restrained shrinkage stresses can be calculated if required, although it should be appreciated that such calculations provide no more than an estimate of crack width and spacing.

Guidance on crack width calculation is provided in Eurocode 2[27]. Caution should be exercised before providing additional reinforcement to achieve very narrow cracks at close centres. Such cracks can be difficult to repair and although narrow, can break down under the action of repeated trafficking by small-diameter hard wheels.

For 'steel fibre only' slabs, no theoretical estimate of crack width and centres is possible. The 'strain softening' response of fibre concrete means that when shrinkage cracks do occur, further movements tend to be concentrated at the existing crack rather than forming a new crack.

For all types of pile-supported slab it is strongly recommended that enlarged pile heads are provided. Careful attention should be given to the detailing and construction of pile heads, isolation details, joints, construction platform finish and surface tolerance and slip membranes so as to minimise restraint to shrinkage. Two layers of slip membrane should be used over pile heads. Shrinkage potential should be minimised by careful attention to concrete mix design. As with all jointless floors, early loading will increase the risk of restraint and associated cracking.

9 Concrete specification

Concrete should be specified in accordance with EN 206[39] and BS 8500[58]. Product conformity certification is required for designated concrete and is recommended for designed and all other types of concrete.

9.1 Specification considerations

The mix design considerations should address the performance objectives described in this section, which are:

- strength and related characteristics
- shrinkage
- placing and finishing needs
- durability (resistance to abrasion, chemicals).

9.2 Strength and related characteristics

9.2.1 Compressive and flexural strength

The standard method of specifying concrete for most structural applications is by characteristic cube strength. However, the important strength parameter for ground-supported slabs is flexural tensile strength – see Section 6.1.1.

Routine flexural tensile testing of concrete is not common practice. Eurocode 2[27] uses fixed relationships, to calculate flexural and axial tensile strength from the concrete compressive strength class.

9.2.2 Concrete in cold store floors

Concrete floors are used in cold stores with temperatures as low as –40°C. Fully matured concrete performs well at constant low temperatures. Immature concrete with a compressive strength of less than 5N/mm^2 may be damaged by freezing. Immature concrete with strength higher than 5N/mm^2 may have its strength development curtailed by too early a reduction in temperature. It is therefore essential that cold store slabs are allowed to mature to develop the required in-situ strength of the concrete under external ambient temperature before the temperature is drawn down. Fourteen days is a minimum and may have to be extended.

Concrete not subject to wetting will resist both continued exposure to temperatures below freezing and freeze–thaw cycles. Therefore there is generally no need to consider enhanced performance.

Air entrainment is normally used to resist damage to exposed saturated concrete by freeze–thaw action and is therefore not applicable to industrial floors. In cold stores the concrete is not saturated and the number of freeze–thaw cycles is very small, so air entrainment is not needed. Importantly, the use of air entrainment in power-trowelled concrete floors should be avoided due to the high risk of delamination.

9.3 Shrinkage and movement

Shrinkage is a reduction in size or volume; for concrete floors, several generic types of shrinkage are of concern. These are:

- drying shrinkage
- early thermal contraction
- crazing
- plastic shrinkage.

All forms of shrinkage can lead to cracking, although drying shrinkage is the most relevant to concrete floor slabs.

9.3.1 Drying shrinkage

All concrete shrinks as the water in the concrete evaporates to the atmosphere. The prediction of drying shrinkage is complicated (see Hobbs and Parrott [59]). Concrete floors usually lose more water from the upper surface, resulting in non-uniform shrinkage and, potentially, curling. Any steps taken to reduce shrinkage will reduce curling.

Although curing is of great importance in achieving a durable concrete floor, it does not reduce shrinkage. A floor will eventually dry and shrink by an amount that is almost independent of when that drying begins.

The main factors influencing drying shrinkage are the volume of cement paste and its water content. Cement and water contents should be as low as possible, consistent with the specified maximum free-water/cement ratio and the practicalities of placing and finishing. The maximum water/cement ratio should be 0.55. The use of water-reducing admixtures (see Section 10.3) is strongly recommended.

Although the cement paste is usually the only component of concrete that undergoes significant shrinkage, some aggregates are known to have high levels of drying shrinkage (see BRE Digest 357 [60]). Aggregate shrinkage should be determined according to EN 1367-4 [61].

The combined grading of the coarse and fine aggregates should be adjusted to minimise the water demand. The largest available size of aggregate should be used, consistent with the thickness of the slab. In practice this is a nominal maximum size of 20mm in the UK.

9.3.2 Early thermal contraction

The hydration of cement generates heat. If the rate of heat generated by cement hydration is greater than the loss of heat from the concrete surfaces, the concrete will expand. Conversely, as the rate reduces with time, there will be net cooling and the concrete will contract.

Early thermal contraction can be reduced by minimising heat generated. Cement content should be kept to a minimum (consistent with strength requirements) and low-heat cements (such as those containing fly ash or ground granulated blastfurnace slag (ggbs)) could be used, particularly in hot weather. In hot weather, consideration should be given to producing and placing concrete in the coolest part of the day.

9.3.3 Crazing

Crazing is the result of differential shrinkage of the surface zone of a concrete slab relative to the bulk and is a common feature of power-finished floors. Experience suggests that, despite its appearance, crazing generally has no effect on the performance of a floor surface. Crazing appears to occur less in concretes with lower water content. See Section 2.6 for further information.

9.3.4 Plastic shrinkage

Plastic shrinkage occurs before the concrete hardens. The main cause of plastic shrinkage is rapid drying of the exposed concrete surface. If the rate of evaporation from the surface exceeds the rate at which bleed water rises to the surface, net shrinkage will occur (with the possibility of subsequent plastic cracking).

Materials and mix design normally have a limited influence but highly cohesive concretes with very low bleed characteristics are particularly susceptible to plastic shrinkage cracking. Concretes with low water/cement ratios or containing fine additions such as limestone powder or silica fume may be at higher risk.

Loss of moisture from the surface can be reduced by protecting the surface from drying air flows, particularly in warm weather. Protection from wind and sun is essential and floors should be constructed after the walls and roof are in position and openings are sealed. Section 12 discusses best site practice.

There are practical difficulties in applying curing measures early enough to prevent plastic shrinkage cracking completely.

9.4 Mix design for placing and finishing

9.4.1 Mix design

Mix design should aim to create a homogeneous and moderately cohesive concrete that will not segregate when being compacted and finished. Excessively cohesive concrete can be difficult to place, compact and finish. Excessive bleeding should be avoided but some limited bleed water is required to assist with the formation of a surface mortar layer that can be levelled and closed by the power-finishing process. Where dry-shake toppings are used, sufficient water is required at the surface for the wetting of the dry material and hydration of the cement component, as well as allowing the air to escape.

Aggregate content should be maximised by using an overall aggregate grading that provides the optimum packing and the minimum effective surface area. In practice, there may be limitations on the aggregate grading available – see Section 10.2. However, it is important to have a consistent grading.

After batching, the designed consistence can reduce as a result of absorption by the aggregates and by evaporation. Delays in the arrival of ready-mixed concrete trucks and the influence of warm weather will both increase these effects as hydration accelerates. A practical way of dealing with this is for the concrete producer and contractor to make provision for the consistence to be adjusted under controlled conditions on site.

Water additions should be supervised by a competent technician and should be limited to that required to increase the consistence to that originally specified. The procedure should ensure that the maximum specified water/cement ratio or the water/cement ratio required for the specified strength, whichever is the controlling value, is not exceeded. When water is added on site, the concrete should be adequately remixed. Site records of water additions and final consistence should be kept.

9.4.2 Consistence and finish

Consistence is normally measured using the slump test (EN 12350-2)[62]. A flow table test (EN 12350-5)[63] may be applicable for flowing concrete but this is unusual for constructing warehouse floors. A representative sample of the concrete must be taken in accordance with EN 12350-1[64]. Consistence is either specified by class or target.

The sample should either be a spot sample taken from the initial discharge of a ready-mixed truck or a composite sample consisting of sub-samples taken throughout the load as it is discharged. Table 9.1 (from BS 8500: 2015 [58]) gives the permissible range for selected consistence tests based on both spot and composite samples.

Specifying by target rather than class is recommended. As a general guide, concrete should be specified as a target slump of 100mm or 140mm where fibres are to be subsequently added. However, for the floor finishing process it is recommended that the preferred range is that shown in brackets in Table 9.1. This level of consistence control will need to be agreed with the concrete supplier.

Table 9.1: Slump class and target slump (from BS 8500: 2015)[58].

Slump class and target slump	Permissible range (mm)	
	Spot sample	Composite sample
S2	30–110	40–100
S3	80–170	90–160
Target 100	50–150 (60–140)	60–140 (70–130)
Target 140	90–190 (100–180)	100–180 (110–170)

The processes for finishing concrete floors (power-floating, power-trowelling, etc.) are particularly susceptible to changes in consistence and setting characteristics of concrete. Therefore, avoiding variability in these aspects of performance should be a high priority. It is essential that concrete is well mixed and that workability is consistent within and between batches. Variations in setting can cause problems in maintaining the working face and lead to problems with levels and appearance.

9.4.3 Fibre addition

Depending on the overall grading of the available aggregates and the volume and type of fibre used, it may be necessary to increase fine aggregate content to improve fibre dispersion and to make the concrete easier to compact and finish. The fibres themselves will also have some effect on consistence.

Fibres can be susceptible to agglomeration into balls in the concrete. Suitable procedures to ensure thorough dispersal should be undertaken.

For further information on practical aspects of concreting see Concrete Society, Good Concrete Guide No. 8 *Concrete practice* [65].

9.5 Abrasion resistance

Achieving adequate abrasion resistance of a concrete floor depends primarily on effective use of power trowels on the concrete as it sets and, to a lesser extent, on the fine aggregate and cement used in the concrete. Fine aggregate in the surface zone can be either present in the bulk concrete used for the floor or a constituent of a dry-shake topping applied to the surface.

The finishing process, in particular the power trowelling, is a skilled activity that should take into account the ambient conditions. Achieving appropriate abrasion resistance and other surface characteristics requires careful timing and control. Although repeated power trowelling is a significant factor in developing abrasion resistance, excessive trowelling will adversely affect appearance.

Effective curing is very important in creating abrasion resistance. This is typically by spraying resin-based curing compounds on the surface as soon as practicable after the finishing process. During the finishing process it is important to minimise surface drying. One of the key factors affecting drying is air movement across the concrete surface and therefore buildings should be totally enclosed before the floor is constructed. See Section 12.

Abrasion resistance develops over time, so even if a floor has gained enough strength to allow it to be loaded, it may not have developed adequate abrasion resistance. This should be considered where construction programmes are very short.

See Section 2.1 for further information.

9.6 Chemical resistance

Any agent that attacks hydrated cement will ultimately damage a concrete floor surface if it stays in contact with the floor for long enough. Frequent cleaning to remove aggressive agents will reduce deterioration, but repeated cycles of spillage and cleaning will still cause long-term surface damage. For further information see Section 2.2 and Appendix B.

10 Concrete materials

The constituents of concrete vary regionally hence it is important to understand their influence on the concrete supplied. For further information on practical aspects of concreting see *Concrete practice* [65].

10.1 Cement

Portland cement and other factory-produced cements (composites) or combinations of Portland cement with fly ash (fa), ground granulated blastfurnace slag (ggbs), or silica fume are primarily specified with reference to EN 197-1[66] and BS 8500[58]. Combination cements are produced at the concrete batching plant where the two powders are combined in accordance with standardised procedures.

The choice of the most appropriate type of cement will be dictated by:

- Availability – batching plant may stock only one type of addition.
- Ambient temperature (winter/summer) – different cements give differing concrete setting characteristics, which are sensitive to temperature.
- Setting time – window available for completing the power finishing.
- Strength development – early strength improves the tensile strain capacity and reduces the risk of cracking.

Table 10.1 summarises the relevant properties of cements and combinations commonly used in floors.

It is recommended for floors that, in normal circumstances, the percentage of fa and ggbs should be limited to 30% and 35%, respectively. The limiting level of the addition must be specified.

For further information on the properties of cements see Technical Report 74, *Cementitious materials* [67].

10.2 Aggregate

Most concrete is made from natural aggregates that are usually specified to conform to the requirements of EN 12620[68] together with the UK guidance document, PD 6682-1[69]. Recycled aggregate (RA) and recycled concrete aggregate (RCA) conforming to BS 8500-2[58] can be used in certain circumstances with care.

The C-M-F (coarse-medium-fine) classification for fine aggregate is useful for selecting appropriate proportions of fine and coarse aggregates in a mix, because the optimum proportion of fine aggregate is partly related to its fineness. Fine aggregates with gradings at the coarse end of grade C or the fine end of grade F should be avoided. If crushed rock sand is used, either on its own or in combination with other sands, the proportion of the combined sand by mass passing the 63µm sieve should not exceed 9%.

Aggregates should be free from impurities such as lignite that may affect the integrity or appearance of the finished surface of a floor. It may not be possible to eliminate impurities entirely but if there are concerns about potential impurities in an aggregate source, the contractor should seek assurances from the concrete producer about procedures adopted to minimise this risk. Information on the history of use of sources should be sought. Information on dealing with surface defects can be found in Section 2.

10.2.1 Mechanical performance

With the exception of ground and polished floors, coarse aggregates have no direct influence on the abrasion resistance of the surface, and so all normal concreting aggregates are suitable. For floors in exceptionally aggressive environments where the surface of the floor is likely to be worn away, the mechanical properties of the coarse aggregate are important.

Table 10.1: Effects of different cements on concrete properties.

Concrete property	Broad cement designation (from Table A.6 BS 8500-1[58])			
	CEM I *	IIA-LL	IIA-L and IIB-V	IIA, IIB-S and IIIA
Addition range	–	6–20% limestone	6–35% fa	6–65% ggbs
7 to 28 day strength	Approx. 80%	Approx. 80%	Approx. 50–80%	Approx. 60–80%
Consistence	–	Similar	Reduced water demand for given consistence	Similar
Cohesiveness	–	Reduced bleed	Reduced bleed rate, longer bleed time	Can bleed more than CEM I
Setting time	–	Similar	Increased. May be significantly extended in cold weather	Slightly longer. May increase significantly at lower temperatures and higher replacement levels
Heat of hydration	–	Similar	Reduced at higher additions	Reduced at higher additions
Curing requirements	All cements require adequate curing to develop abrasion resistance			
Other comments	–	–	Extended finishing window in hot weather. Can delay finishing in cold weather, especially with higher percentages of additions	
Note: * For structural concrete this is taken as being Class 42,5 and above.				

EN 12620[68] has adopted a classification system based on the Los Angeles test. A maximum Los Angeles coefficient of 40 is recommended for aggregate for normal floors. For floors in exceptionally aggressive environments, it may be appropriate to specify a lower value of 30 or 35.

Fine aggregates are present at the surface and can affect performance. Fine aggregates that contain larger particles of friable materials that are likely to break down under mechanical action should not be used.

10.2.2 Drying shrinkage

The principal effect of the aggregate is to restrain the contraction of the cement paste, thereby helping to reduce the likelihood of cracking. In general, aggregates with a higher modulus of elasticity (greater 'stiffness'), cubic shape and rough particle surface textures are likely to offer more restraint to concrete shrinkage.

The magnitude of shrinkage can vary substantially with type of aggregate. Quartz, granite and limestone are frequently associated with low concrete shrinkage, whereas sandstone and some basic igneous rock aggregates are more likely to cause or permit comparatively higher shrinkage.

BS 8500-2[58] limits the aggregates drying shrinkage to not exceed 0.075% when tested to EN 1367-4[61]. The drying shrinkage of the majority of concrete in the UK is of the order of 0.03% to 0.045%.

10.3 Water-reducing admixtures

Water-reducing admixtures are used to reduce the free water content for a given concrete consistence. This reduces drying shrinkage in the concrete and hence cracking and curling of the slab.

Shrinkage of concrete occurs mainly in the cement paste. To limit shrinkage, both the water and the cement content should be kept as low as possible without compromising its placing and finishing ability, strength or durability. The water reduction using these admixtures should also allow cement content to be reduced while still achieving the required concrete strength class.

Dispersion of the admixture throughout the concrete is key to the performance of the concrete. Incorrect use and inadequate mixing can lead to variable concrete setting characteristics and poor performance.

Water-reducing admixtures are typically dosed at 0.30 to 0.7 litres per 100kg of cement and give water reductions of up to 30% without reducing consistence.

It is recommended that the water-reducing admixture is specifically designed for flooring application.

10.4 Dry-shake toppings

Dry-shake toppings are dry blends of cements, fine aggregates, admixtures and sometimes pigments. They are usually factory blended and supplied in bags. They are commonly used to provide colour and or to help suppress fibres at the surface.

Dry-shake toppings depend upon bleed water from the underlying concrete for hydration and for them to be worked monolithically into the base concrete. Although excess bleed water should be avoided by appropriate mix design, it is equally important to have enough moisture at the surface when dry-shake materials are applied. If there is insufficient bleed water available to wet the dry-shake, there is a high risk of delamination.

When a coloured floor is required, the appearance of small laboratory-produced samples will not be representative of the finished floor. The colour of a concrete floor with a coloured dry-shake topping is more variable than a resin coating or other applied coatings such as paint – see Section 2.4.

Some dry-shake toppings may enhance abrasion resistance.

10.5 The importance of curing

All cements require effective curing to develop optimum properties in the hardened concrete. Excessive moisture loss may result in surface dusting and poorer abrasion resistance. It is therefore important to apply appropriate curing techniques as soon as practical after finishing is completed.

Figure 10.1: After application of spray on curing compound and saw cutting, curing is continued using polythene sheet.

11 Construction and joints

The construction method and the joint layout should be planned to suit the intended floor use. The factors to be considered should include building geometry, equipment layout, joint width, levelness across the joints and the stability of the joint edges and surface regularity. Detailed guidance on surface regularity is given in Section 3.

Some cracking of slabs between joints may be expected, particularly in larger slab panels and in 'jointless' construction – see Section 11.1.1. The significance of any such cracking in terms of operational requirements or appearance should be considered at the design stage.

Since concrete shrinks, it is not possible to dispense with joints completely. Joints are also required because there are practical limitations on the area of floor that can be constructed at any one time.

11.1 Construction methods

There are two main construction techniques: large area and long strip.

Large areas of up to several thousand square metres can be laid in a continuous operation (Figure 11.1). Levels are controlled either manually using a target staff in conjunction with a laser level transmitter or by direct control of laser-guided spreading machines.

Figure 11.1: Large area construction.

Tighter surface regularity tolerances can be achieved by using additional measures such as manual levelling techniques, often referred to as 'highway straightedge', which can be used on the stiffening concrete surface to remove 'high spots', see Figure 11.2.

Long strip is laid in strips typically 4 to 6m wide using the formwork as the principal method of flatness control.

Shrinkage of the concrete is inevitable and any restraint to that shrinkage has the potential to cause cracking. It follows that in order to reduce the risk of cracking, steps should be taken to reduce both the potential shrinkage and potential restraint.

Shrinkage can be minimised by avoiding high cement contents and reducing water content. Note that there are practical limitations on both aspects of concrete mix design – see Section 9.4. Restraint to shrinkage can be minimised by careful attention to isolation details around columns and other intrusions, by the use of sawn-induced joints and by minimising sub-base restraint.

The general approach to minimising the risk of shrinkage-induced cracking adopted by TR34 is to reduce the restraint to movement. Drying shrinkage induced cracking can be minimised and thereby allow larger areas between joints by combining:

- the control of sub-base flatness
- provision of a slip membrane
- optimisation of the concrete mix design
- careful consideration of joint layouts and joint types
- isolation of hard spots, e.g. column bases.

11.1.1 Large area construction

Two main methods are used, categorised by the reinforcement and joint type.

Jointed – ground supported
Formed free-movement joints are provided at the perimeter of each bay at up to 50m but typically 40m intervals. These typically open in the order of 20mm.

Sawn restrained-movement joints are cut, typically on a 6m grid in both directions, as early as possible after casting, resulting in an induced crack under the saw cut. At the surface these joints typically open to 4–5mm.

The floor then becomes a set of smaller panels that continue to shrink as they dry out. If the sub-base has been constructed in accordance with the recommendations in Section 5 and has been provided with a slip membrane, the frictional restraint will be relatively low, and the panel will shrink with a low risk of cracking.

Jointless – ground and pile supported
Formed free-movement joints are provided at the perimeter of each bay at up to 35m intervals. These typically open in the order of 20mm.

Jointless floors are built using large area construction methods, generally with steel fibres although reinforcing steel can be used or a combination of both. The word 'jointless' can be misleading, as there is a practical upper limit to the area of concrete that can be placed in a single continuous operation. No additional joints are provided within the formed bay joints.

For jointless slabs, particular attention should be given to minimising shrinkage and restraint.

Jointless slabs are more susceptible to the restraining effects of the applied loads, such as racking, and so loading should be delayed for as long as possible. Joints should be provided between zones of racking, for example in transfer aisles.

For jointless pile supported floors the effect of joint type and location on the structural performance of the slab must be taken into account. There will be effectively two types of joint:

- Tied joints, e.g. at construction joints, where there is a requirement for full continuity of the slab reinforcement through the joint.
- Free-movement joints, where there is provision for horizontal movement and load transfer, where the joint is effectively a hinge.

Sawn-restrained movement joints should not be provided in pile-supported slabs.

11.1.2 Long strip construction

The floor is laid in a series of strips typically 4 to 6m wide, with forms along each side (Figure 11.2). Strips can be laid alternately, with infill strips subsequently placed. Strips are laid in a continuous operation and joints are sawn transversely across each strip up to 6m apart to accommodate longitudinal shrinkage.

As formwork can be set to tight tolerances, and as the distance between the forms is relatively small, this method lends itself to the construction of floors with a high standard of surface regularity – see Section 3.

Figure 11.2: Long strip construction.

11.1.3 Wide bay construction

Wide bay construction is a variation on large area construction but with bay widths limited to 12 to 15m. Limiting the bay width permits access for the use of a 'highway straightedge' on the concrete surface to control the surface tolerances more accurately

11.1.4 Overlay construction

Floor slab overlay construction is commonly used where the existing floor slabs have become unserviceable. Overlays commonly take the form of bonded or unbonded construction.

Bonded overlay floor slabs are typically constructed to thicknesses of 75-100mm. Careful control of construction tolerances is required to ensure a minimum thickness is maintained throughout. The effectiveness and performance of the bonding mechanism is critical. This edition of TR34 does not provide technical guidance on the design and construction of such overlays. Industry guidance and specialist technical advice should be gained to design such floor types.

Unbonded overlay floor slabs, generally constructed on a membrane, should be designed and constructed as a ground-supported floor in accordance Sections 6 and 7.

11.1.5 Two-layer construction

Greater floor thicknesses may require two-layer-type construction to provide the necessary surface regularity. Such floors require a structural bond between layers throughout their area to ensure the floor maintains its structural integrity and serviceability requirements.

11.1.6 In-floor heating systems

Where heating systems are incorporated in the floor itself, the following need to be considered:

- floor design, e.g. reduction in sectional thickness, load capacity
- placing and fixing of the pipework
- joint and pipe interface
- pour size
- placing and finishing operations
- embedment of subsequent fixings.

Specialist advice should be gained for such installations to ensure all aspects of the floors design are considered.

11.1.7 Post-tensioned floors

Post-tensioning can be used to construct jointless floors and is one method for overcoming some or all of the tensile stress that normally occurs. Panel sizes can be up to 100m × 100m and there is a low risk of cracks opening as tendons cast within the slab keep the concrete in compression. However, the free-movement joints around the post-tensioned panels may open considerably by several centimetres.

The design of post-tensioned floors is not covered in this publication. Suppliers of post-tensioning systems should be consulted. Consideration should also be given to the use of proprietary joint systems that can cope with potentially wide joint openings.

11.2 Joints

The number and type of joints in a floor will depend on the floor construction method and its design and the chosen method should be related primarily to the planned use of the floor. Joints can be a potential source of problems because the edges of slab panels are vulnerable to damage caused by the passage of materials handling equipment, with wider joints particularly susceptible. Joints will need to be maintained during the life time of the floor – see Section 13.4.

Joints are provided for two reasons, 1) to relieve tensile stresses induced by drying shrinkage or temperature changes and 2) to cater for breaks in the construction process.

Joints in concrete floors are created in two ways; 1) by sawing and 2) by forming with temporary demountable formwork or permanent proprietary joint systems.

Expansion joints are not used in internal floors, except those subject to above-ambient temperatures and to large temperature fluctuations. In most floors, the dominant movement is that caused by drying shrinkage and any ongoing thermal-related movements are much smaller. Cold store floors have greater thermal movements but the slabs do not expand beyond their as-constructed dimensions. Therefore expansion joints are not required. Designers should satisfy themselves that there is a definite need for expansion joints, avoiding their unnecessary installation and the resulting wide gap required between floor panels. Expansion joints require the provision of compressible filler and load transfer by debonded dowels or other mechanisms.

11.3 Joint types

Joints are classified according to the movement they allow and the method by which they are formed, as follows:

- free-movement joint
    - sawn
    - formed
- restrained-movement joint
    - sawn
    - formed
- tied joint
- isolation joint.

11.4 Free-movement joints

Free-movement joints are designed to provide a minimum of restraint to horizontal movements caused by drying shrinkage and temperature changes in the slab, while restricting relative vertical movement. Free-movement joints have the potential to open wider than restrained-movement joints. There is no reinforcement across the joint therefore dowels or other mechanisms provide load transfer. Load-transfer mechanisms including dowels and dowel sleeves should be engineered to reduce vertical movement to a minimum.

A free-movement joint (not an isolation joint) should be provided between a floor slab and an adjoining structure where the adjoining structure, for example external pavement, dock leveller (Figure 11.3) or machine base, forms part of the floor surface trafficked by MHE.

Figure 11.3: Dock levellers.

11.4.1 Sawn free-movement joints

Sawn free-movement joints are cut as soon as the concrete is strong enough to be cut without damaging the arrises –see Figure 11.4. For more detail on sawing joints see Section 11.8. Debonded dowels set in position in dowel cages before the concrete is placed provide load transfer. Steel fabric does not cross the joint. Care must be taken to ensure that the dowels are horizontal and perpendicular to the line of the joint and that their positions are not disturbed during the placing of the concrete. If this is not done, the joint will become tied, thereby increasing the risk of a crack forming nearby or a larger opening of an adjacent restrained-movement joint – a dominant joint. Sawn joint edges can be damaged by intensive traffic with small hard wheels, such as from pallet trucks.

The use of cast-in bottom crack inducers below the location of saw cuts is not recommended, as cracks can occur above the crack inducer before sawing commences. The alternative use of plastic crack inducers pushed down into the wet concrete is also not recommended as they create poorly defined arrises.

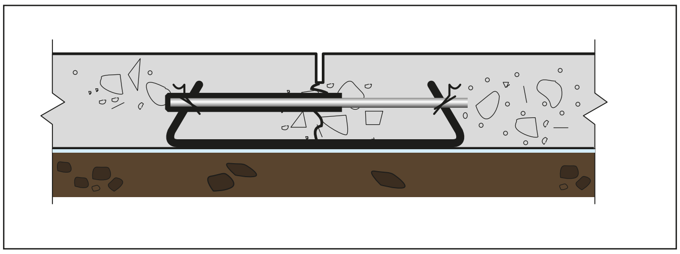


Figure 11.4: Sawn free-movement joint, can be used in fabric and fibre reinforced concrete.

11.4.2 Formed free-movement joints

Formed free-movement joints are created by formwork and are provided at the perimeter of each bay and use debonded dowels or discrete plate dowel systems to provide load transfer.

Joint formers should extend as near as possible to the full depth of the joint face and should not permit excessive extrusion of concrete beneath their lower edges. A small gap is useful as it will allow air to escape and will provide visual confirmation of full compaction when concrete paste is evident at the base of the formwork.

Dowels can be round, square or plate types. Round bars allow longitudinal movement only. The sleeves of square dowels have compressible side inserts to allow lateral as well as longitudinal movement. Sleeves should be of a shape compatible with the dowel and with a good fit and sufficient stiffness to prevent vertical movement.

Discrete plate dowel systems (see Figure 11.5) of various shapes are commonly used as alternatives to dowel bars to allow movement in the horizontal plane. These are not to be confused with the continuous plates which have been found to perform poorly in service and are therefore not recommended.

The edges of formed free-movement joints should be protected with steel plates of adequate thickness to provide sufficient stiffness and resistance to bending from wheel impact when the joint has opened, which would cause breakdown of the concrete behind the steel section. These can be incorporated into permanent formwork systems.

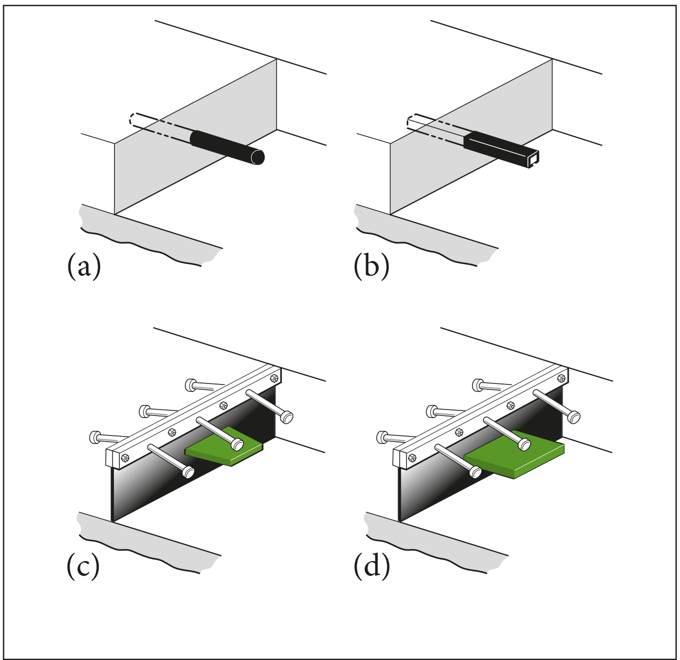


Figure 11.5: Formed free-movement joints with load transfer using (a) round dowels, (b) square dowels, (c) and (d) proprietary systems with arris protection and plate dowel configurations.

11.5 Restrained-movement joints

Restrained-movement joints are provided in fabric reinforced floors to allow limited movement to relieve shrinkage-induced stresses at predetermined positions. The fabric reinforcement is continuous across the joint.

11.5.1 Sawn restrained-movement joints

Sawn restrained-movement joints are sawn as soon as the concrete is strong enough to be cut without damaging the arrises – see Figure 11.6. For more detail on sawing joints see Section 11.8.

For slabs cut into (typically) 6m panels, these joints can be expected to open by an extra 1–2mm beyond their initial width at the top surface of 3–4mm as the reinforcement across the joint yields under the stresses created by the shrinkage of the concrete. The sawn joint edges are relatively resistant to damage for narrow joint openings, but can be damaged by intensive traffic with small hard wheels, for example from pallet trucks.

Load transfer is provided by the fabric reinforcement across the joint and by aggregate interlock, see Section 7.9. The steel area is typically 0.08% to 0.125% of the slab cross section. It is assumed that the reinforcement across the restrained joint yields as the panels shrink.

Reducing the percentage of steel carries the risk of wide joint openings as the steel yields and load transfer capability is progressively lost under the dynamic actions of MHE, see Section 4.3. This may result in significant deflections, cracking and joint arris damage. Vertical movement at joints can lead to sub-base compression and loss of slab support.

Increasing the percentage of steel carries the risk of more mid-panel cracking, as the steel may not yield at each joint.

The use of cast-in bottom crack inducers below the location of saw cuts is not recommended, as cracks can occur above the crack inducer before sawing commences. The use of plastic crack inducers pushed down into the wet concrete in place of saw cuts is also not recommended as they create poorly defined arrises.

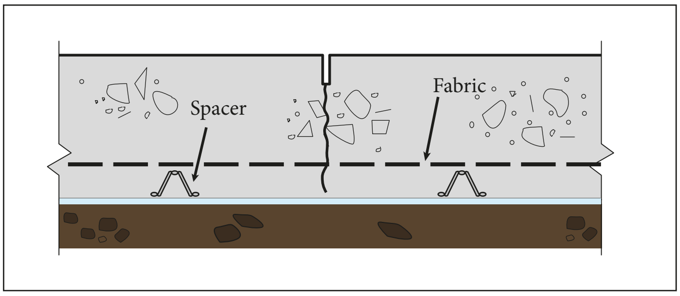


Figure 11.6: Sawn restrained-movement joint.

11.5.2 Formed restrained-movement joints

Formed restrained-movement joints are created by using formwork through which reinforcing bars are inserted – see Figure 11.7. The joint is designed for some limited horizontal movement, similar to that expected in a sawn restrained-movement joint, the bar dimensions and spacing giving approximately equivalent cross-section per metre length of joint to that of the fabric in the slab. The reinforcing bars provide load transfer.

Like other formed joints, there will be weaknesses in the arrises but the potential for damage is reduced where the arrises are in close proximity – see Section 11.8.

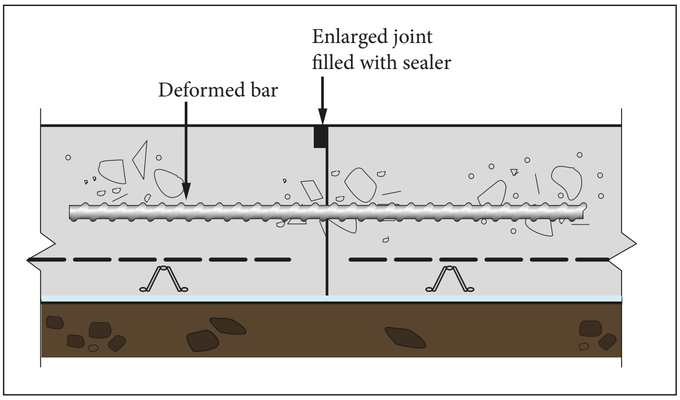


Figure 11.7: Formed restrained-movement joint.

11.6 Tied joints

Tied joints (Figure 11.8) are sometimes provided to facilitate a break in construction at a point other than at a free-movement joint. The joint is formed and provided with a cross-sectional area of steel reinforcement high enough to prevent the joint opening. That is, the load capacity of the steel used should be greater than the tensile capacity of the concrete section. Tied joints should not open significantly and a well-designed and formed joint should not suffer significant damage from MHE traffic.

The reinforcement bars also provide load transfer.

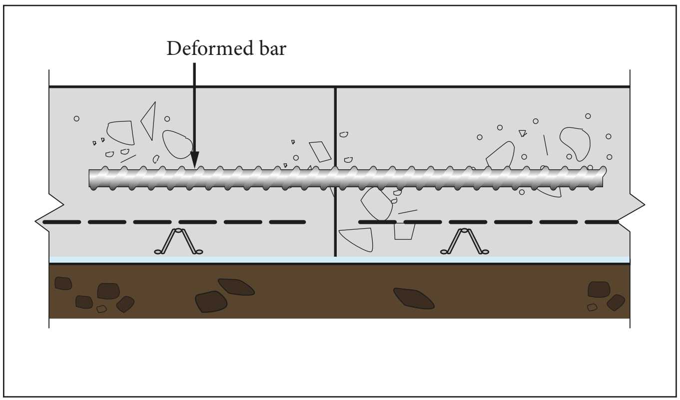


Figure 11.8: Tied joint.

11.7 Isolation joints

The purpose of isolation joints is to avoid any restraint to the slab by fixed elements at the edges of or within the slab, such as columns, walls, machinery bases or pits. They can also be used to isolate the slab from machinery bases that are subject to vibration. However, where a floor slab adjoins a fixed structure that is itself to form part of a trafficked area over which MHE will pass then a free-movement joint should be provided so that there is adequate load transfer without restraint. This will typically be the case at dock levellers (see Figure 11.3) and alongside conveyor tunnels.

Where there is any risk of movement towards a fixed element – for example, laterally against a column, pit or base – a flexible compressible filler material should be used (see Figure 11.9). These materials are typically 10–20mm thick and the choice of material and thickness should be based on an assessment of the likely movement. They should not be bent around right-angled corners, as the effective thickness at the corner will be much reduced by pinching.

Isolating materials should extend throughout the full depth of the slab and be sealed effectively to prevent the ingress of grout into the space between the slab and adjoining structure. Typical joint details are shown in Figure 11.10.

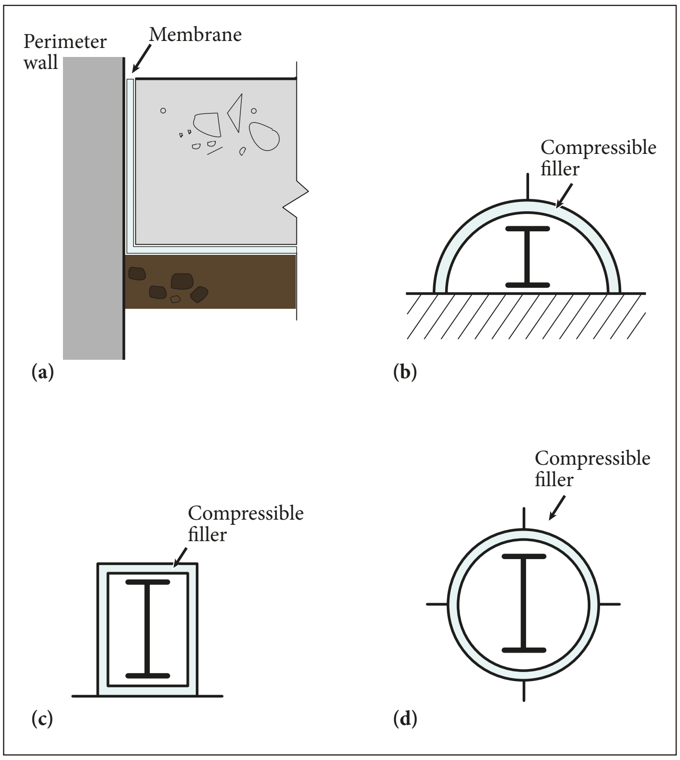


Figure 11.10: Slab isolation details at (a) slab perimeter and (b)–(d) columns.

11.8 Performance of sawn and formed joints

11.8.1 Sawn joints

Sawn joints are usually 3 to 4mm wide and are cut as soon as practicable after placing the concrete when the concrete is strong enough to avoid damage to the arrises (Figure 11.11), nominally 24 hours after placing. They are cut to a depth of typically 25 -30% of the slab depth, creating a line of weakness in the slab that induces a crack below.

Figure 11.9: Isolation details around column.

Figure 11.11: Joint sawing.

It should be noted that deeper saw cuts will reduce aggregate interlock and the associated load-transfer capacity of the joint, see Section 7.9.

The concrete at the arrises of a sawn joint is representative of the slab as a whole, being fully packed with aggregate and without excess cement paste, see Figures 11.12. Sawn joint edges are relatively resistant to damage where the joint opening is limited, but can be damaged by intensive traffic with small hard wheels, such as from pallet trucks.

Surface levels across a sawn joint are consistent with the profile of the floor to either side of the joint. Generally, there will be minimal interruption to wheeled traffic across sawn restrained-movement joints. However, sawn free-movement joints can be expected to have wider openings.

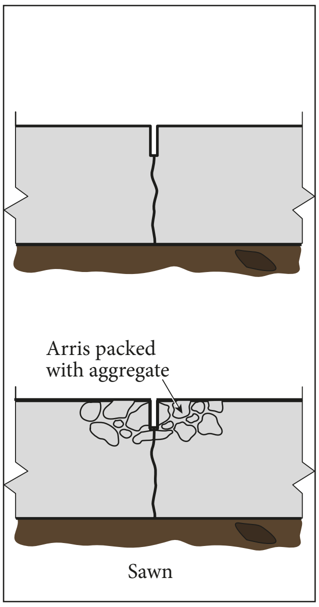


Figure 11.12: Concrete integrity at sawn joints.

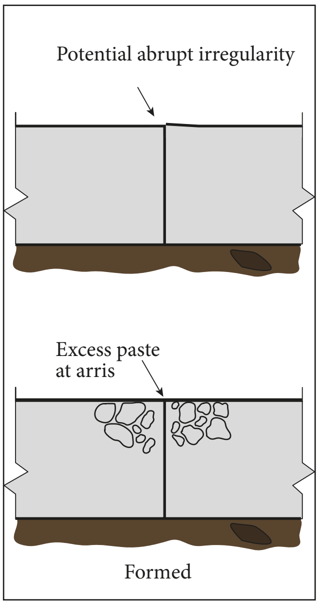


Figure 11.13: Concrete integrity at formed joints.

11.8.2 Formed joints

The concrete at the arris of a formed joint will have less aggregate and more relatively weak cement paste. The concrete at the edges may be less well worked by the power trowel. Care is needed when removing temporary formwork to ensure that the arris is not damaged – see Figure 11.13.

Care is also needed to obtain the required surface regularity immediately adjacent to either side of, and therefore across, a formed joint.

11.9 Armouring of joints

Free-movement joint edges should be armoured if the joints are to be trafficked by vehicles with small hard wheels, e.g. pallets trucks. The arrises at formed free-movement joints can be protected by steel armouring, as shown in Figure 11.5(c). Most armouring systems are combined with permanent formwork and load-transfer systems. Factors to consider are the width, grade and flatness profile of the steel arris and the capacity of the load transfer mechanism at potentially wide openings which can be typically 20–30mm.

To be effective, the steel arris must be sufficiently stiff and well fixed to the concrete to resist and distribute the impact forces of the materials handling equipment wheels. The steel should be thick enough to resist deformation at its arris and have a right-angled profile on the face adjacent to the concrete.

Long-term performance of armoured joints can be improved by monitoring the joints over the first year or two of life and filling as required – see Section 13.4.

Anchorage fittings such as shear studs need to provide adequate stability without creating planes of weakness in the concrete close to the joint. They should extend to the full length of the joint and be provided close to the ends of each rail and near to joint intersections. In addition, consideration can be given to welding the armouring sections at corners and intersections on site or by the use of prefabricated sections. However, it is important to ensure that at intersections, all four corners are free from connection to each other. It must be possible to compact the concrete fully under and around the anchorages and any other steel sections used for a load-transfer mechanism.

11.9.1 Installation

The armouring system should be provided with a means of fixing with sufficient accuracy to provide a smooth transition across the joint. Matching halves of the system must have temporary locating devices to provide stability during construction and accuracy across and along the joint when in service. These devices should be removed during construction or be self-separating. Inaccuracies are not easily remedied after construction.

Anchorage fittings such as shear studs need to provide adequate stability without creating planes of weakness in the concrete close to the joint. They should extend to the full length of the joint and be provided close to the ends of each rail and near to joint intersections. In addition, consideration can be given to welding the armouring sections at corners and intersections on site or by the use of prefabricated sections. However, it is important to ensure that at intersections, all four corners are free from connection to each other. It must be possible to compact the concrete fully under and around the anchorages and any other steel sections used for a load-transfer mechanism.

11.10 Joint layout

In an ideal joint layout plan the objective is to minimise the risk of cracks. This is achieved by:

- ideally having square panels, particularly in fibre reinforced floors, but limiting the length-to-width ratio (aspect ratio) to 1:1.5
- avoiding re-entrant corners
- avoiding panels with acute angles at corners
- avoiding restraint to shrinkage by using isolation details around fixed points
- avoiding point loads at joints
- limiting the longest dimension between sawn joints to typically 6m
- limiting dimensions to 35m for jointless bays and 50m for jointed bays. These limitations are not applicable to long strip or wide bay construction.

In practice, the floor plans of most buildings dictate that conflicting requirements have to be balanced. Columns, bases and pits do not always conveniently fit to predetermined grids, and areas around dock levellers pose particular difficulties. Therefore, basic panel grids may need to be modified to accommodate column spacing and other details that depart from the ideal joint layout.

Joint openings will increase as joint spacing increases.

Ideally, joints should align with each corner of fixed construction elements. Where this is not practical, it may be necessary to have an internal (re-entrant) corner in the panel. There is a risk of cracking at such corners. Cracks at corners can be controlled but not prevented by placing additional trimming reinforcement in the top across the corner. The area of steel should be a minimum of $300mm^2$ within a 0.5m zone adjacent to the corner. The bars should be placed at 45° to the corner itself, a minimum 100mm apart and 1m long or 80 × bar diameter, whichever is longer. The minimum requirement is equivalent to, say, 4 × 10mm diameter bars. Additional saw cuts can also be provided to confine anticipated cracking to predetermined positions.

Slabs should be isolated from fixed elements such as ground beams, dock levellers, column surrounds, slab thickenings and machine pits. Load transfer should be provided if trafficked and to restrain curling.

Floors should not be used to resist horizontal kick-out forces from portal frames, as this will induce restraint and result in significant cracking.

Joints should be positioned so as to minimise the trafficking from MHE while taking into account other factors such as minimising restraint to shrinkage and the structural requirements for pile-supported slabs.

In narrow aisle warehouses, longitudinal formed joints should be ideally positioned to avoid the wheel tracks of materials handling equipment.

11.11 Wire guidance systems

Where wire guidance is to be installed across free-movement joints, in particular in jointless construction, the wire needs to have 'slack' to accommodate the movement. This may be achieved by providing a loop between the joint faces. Coordination in conjunction with the joint type and layout is very important.

The saw cuts for the wire can induce cracks.

11.12 Joint sealants

Joint sealants are provided to prevent ingress of debris and to support the joint arris while allowing for movement.

11.12.1 Properties

Joint sealants are supplied as liquids or paste-like materials that cure to create a flexible seal. They can have one component, which cures by reaction with the environment, or two components, which cure by reaction of the components after mixing.

Sealants are characterised by their movement accommodation factor (MAF), which is the total movement the sealant can accept in service expressed as a percentage of the original joint width, and by their Shore A hardness value. Typically, floor sealants have MAF values in the range 5–25%, and Shore A hardness values in the range 20–60.

Sealant selection should be based on the level of anticipated movement in service and the need for arris support. Anticipated joint openings should be such that the MAF value is not exceeded. Flexibility and hardness are conflicting qualities in any material and product selection is therefore a compromise.

Aspects to consider are the:

- movement required (MAF)
- joint or opening width at time of installation
- hardness (Shore A)
- installation temperature
- adhesion to surface substrate
- cure rate.

11.12.2 Joint sealants in new floors

Sealing should be left as late in the construction process as possible, and ideally just before building handover. Typically, high MAF is associated with softer sealants, which will give only limited joint arris support.

Initially, a sealant with a MAF in the range 25–35% and a Shore A in the range 30–50 should be used. This sealant should be considered as temporary and may need to be replaced later with a harder sealant that will provide support for the joint arris, while still providing capacity for movement. These may debond in due course and they should be replaced as required. It should be noted that all joints will open and close by small amounts in response to slab temperature and moisture variations.

Joints and sealants should form part of the long-term monitoring and maintenance regime for the floor – see Section 13.4.

11.12.3 Sealant application

Joint faces should be cleaned to remove cement slurry, mould oils or any loose materials. The concrete surface needs to be dry before applying the sealant. Ideally, the joint should be filled flush with the surface.

The sealant should be allowed to cure fully before the joint is trafficked. The rate of cure of sealant is dependent on the ambient temperature and the sealant type. The time for full cure depends on humidity and the dimensions of the sealant section. The sealant manufacturer should be consulted.

11.12.4 Joints in cold stores

Joints in cold stores will close when slab temperatures are raised from the frozen condition to ambient, potentially causing damage to joint sealants.

The thermal movement, Δ_j, in mm, can be estimated as follows:

$$\Delta_j = (\alpha T L)10^3$$

where L = distance between free-movement joints (m)

T = change in temperature (°C)

α = coefficient of thermal expansion of concrete.

The default value for coefficient of thermal expansion of concrete in Eurocode 2[27] Clause 3.1.3(5) is 10×10^{-6}/°C (microstrain/°C). Specific values for concretes made with various aggregates are given in CIRIA 660 Table 4.4 [70]

12 Design and construction best practice

The best outcome for an industrial floor results from a well-integrated design and construction process.

The design of a floor is a specialist activity and should be undertaken by designers who have a thorough working knowledge of TR34 and who have practical experience of floor construction techniques. Specialist flooring contractors should be consulted during the design process.

Design should be based on a detailed examination of the proposed use of the building using a design brief such as that shown in Appendix A. Future change of use of the building should be anticipated.

A geotechnical appraisal is essential, as described in Chapter 5. This appraisal should include predictions of settlement and proposals for ground improvements including piling proposals where appropriate.

Construction of the floor should only commence once the shell of the building envelope is completed. The concrete specification and testing should be flooring specific.

It is strongly recommended that supervising engineers and contractors develop quality control plans and that these should include checking and reporting procedures.

12.1 Preconstruction planning

An essential part of a successful floor slab project is the preconstruction planning process. During this phase, the principal contractor, main suppliers and specialist flooring contractor should address the construction and quality areas listed below.

The lines of communication between the parties to the project should be identified along with a clear understanding of individual responsibilities, including:

- Overall building programme enabling construction of the floor in a completed building envelope, totally protected from the weather.
- Sub-base level tolerances.
- Pile head tolerances.
- Maintenance of pile head integrity during construction.
- Floor slab construction programme, including access to clear building and storage areas, relationship with other trades, proximity of working, slab access requirements, and curing.
- Post-construction access, including plans to avoid surface damage and overloading of newly completed slab.
- Timescale for permanent loading.
- Timescale for lowering temperature in cold stores.
- Material supply, delivery and storage arrangements determined and back-up contingencies organised.
- Method of working established, including numbers of personnel, plant type and quantities, concrete supply and emergency joint detailing procedures in the event of breakdown in concrete supply.
- Quality control procedures and compliance testing.
- Calibration of specialist levelling, transmitting and receiving equipment.

These points can be covered by a number of means, the most common being the tender correspondence and pre-start meetings.

12.2 Construction

Areas of practice that should be addressed during the floor construction process should include the following:

- Health and safety compliance and methods of work, including provisions for noise, dust and fume control, clean-up and waste disposal.
- Provision of adequate lighting for construction operations after daylight hours in buildings with poor natural light.
- Provision of adequate ventilation for works in confined, poorly ventilated spaces.
- Delivery documentation check procedures against the specification for materials delivered, e.g. concrete strength class, reinforcement type.
- Sub-base surface regularity and stability check procedures, i.e. level grid prior to pouring and determination of resistance to rutting by construction traffic, including concrete delivery trucks.
- Integrity and level of any slip or gas membrane.
- Stable set-up of specialist laser levelling transmitters and receivers.
- Level checking procedures for formwork, optical levels and laser equipment.
- Installation of fabric or bar reinforcement to provide stable and suitable detailing, including correct use of chairs and spacers.
- Control of allowable standing time for concrete delivery trucks with careful attention to delivery range and weather conditions.
- Dosing and mixing procedures for steel fibres and admixtures where added at site.
- Thorough mixing of concrete before discharge.
- Application equipment and procedures for dry-shake finishes.
- Procedures for sampling and testing concrete and other materials, including concrete cubes, dosage and distribution of steel fibres, and spreading rates of dry-shake finishes including dust and emission control.
- Protection of adjacent works or perimeter walls or columns from splashes of concrete.
- Assessing concrete before start of power floating and finishing operations.
- Procedures for sawing restrained-movement joints.
- Selection and application of curing compounds.
- Prevention of contamination of concrete surfaces by waste materials.

See Appendix I for an example daily work activity check sheet.

12.3 Protection of a new floor

The new floor should be left uncovered and undisturbed after construction for long enough for the concrete to gain strength, so that damage to the surface and joint arrises is avoided. Ideally, this should be for three days, or longer in cold weather. If earlier access is required then additional care must be taken.

Where the long-term appearance of a floor is particularly important, such as in retail premises, specific measures are required. These floors may incorporate dry-shake finishes, the appearance of which can be seriously compromised by damage or staining to the floor.

Where protection is required, it should be left in place for as short a time as possible and preferably removed at the end of each work shift. This will permit the concrete to lose moisture to the atmosphere without build-up of condensation, which may react with protective boarding and cause staining. Trapped moisture under polythene can also temporarily mark the surface. Hoists and other vehicles should be fitted with tyre covers and oil drip catchers. The appearance of a new floor will improve over time with regular mechanised cleaning. This process can be accelerated, if required, by repeated early cleaning.

12.4 Post-construction

After construction is complete, sampling and compliance testing reports (including the following) should be completed:

- Surface regularity survey.
- Construction quality control reporting.
- Information required under the Construction (Design and Management) Regulations [71].
- Information required for the operating and maintenance manuals.

13 Maintenance

Floors provide an operational platform for storage and materials handling equipment and these operations will create wear and tear that must be addressed on an ongoing basis.

13.1 Introduction

Failure to maintain concrete floors and joints will ultimately lead to higher long-term costs and lower efficiency. A philosophy of planned inspection, maintenance and repair should be adopted as soon as the floor is constructed.

Even if the building is left empty, some maintenance will still be necessary; for example, joint movement due to natural concrete drying shrinkage can lead to joint sealant failure.

Issues such as joint deterioration, debonded or split joint sealant and impact damage should be treated under an adopted inspection and maintenance plan.

A defect is defined as a feature or matter causing an obvious serviceability or structural issue that directly prevents safe and efficient use of the floor. Normal wear and tear should not be confused with construction defects.

Examples of typical defects may be identified as loose sections of steel joint protection, shrinkage cracks, loose surface aggregate or aggregate pop-out, concrete contamination, surface delamination, cement/sand balling etc.

Most building contracts have a period of defects liability, typically 12 months, which commences at the point in time where a client effectively takes possession of the building from the main contractor. At the end of that period an inspection determines the defects to be made good and on satisfactory making good of those defects a certificate is issued to that effect.

The function of this period is to identify those defects that become apparent during initial use of the floor in order that they can be repaired, either during the period if the issue is one of concern, or at the end of the 12-month period. The period of defects liability is not a maintenance-free period for the building user.

13.2 Cleaning

Regular cleaning is essential to stop dirt and dust building up, as increased surface wear or susceptibility to slips can result if a floor is not clean and dry. Power-trowelled surfaces can normally be easily cleaned with a wet scrubber/dryer using neutral cleaning agents. Dry cleaning can scratch the surface sealer coat. A wet scrubber drier is preferred to lift fine dust. Larger debris (e.g. nails, wood shards from pallets, steel banding etc.) should be removed from the floor as soon possible as significant damage can occur when jammed under wheels, especially at joints.

Where road vehicles enter the warehouse, additional cleaning measures may be necessary to remove dirt, water, salts, oil/fuel or other spillage.

Figure 13.1: Mechanical cleaning.

13.2.1 Cleaning frequency

Frequency will largely depend on the type of contamination and level of cleanliness required. For maximum effectiveness, cleaning should be carried out on a daily or weekly basis as part of standard housekeeping procedures.

13.2.2 Cleaning materials

There is a wide range of materials for the cleaning of floors: many are a complex blend of chemicals and some have specific application requirements. Most are formulated to be effective against a range of materials and some are very specific to the contamination they are designed to remove, e.g. bio products which are targeted against fats and oils. Similarly, some cleaning products may have an adverse reaction on the floor surface if used in the wrong concentration, giving rise to etching or wear. This may be a one-off effect or as a cumulative result of repeated activity. Trials should be undertaken on small areas away from sensitive areas prior to widespread use.

13.2.3 Spillages

Spillages of any liquid should be wiped up or absorbed and removed as quickly as possible. Not only is this important for health and safety (slip hazard) but it will also help minimise staining or chemical attack of the floor surface. Once the spillage is removed, the floor should be cleaned thoroughly.

13.2.4 Tyre marks

Non-marking tyres should be used on all materials handling equipment where possible to reduce excessive marking, especially at turning locations. Marking can also result from the wheel skidding and acceleration interaction with the surface floor sealer, often showing as clean patches of floor leading up to a darker marking. To remove these marks, the floor surface sealer has to be removed (usually by a specialist floor cleaning contractor) but only after it is deemed the floor has cured sufficiently and no detrimental effect will result from the removal.

13.3 Surface wear and damage

How the floor surface will wear is dependent on the type of materials handling equipment, cleanliness of the floor and traffic intensity. Most power-trowelled floors are finished with an acrylic curing and sealing agent that will provide some resistance to normal floor use. These agents are designed to gradually wear to reveal the concrete surface but in the case of heavily trafficked areas, they can be reapplied using a roller or spray (after the floor has been thoroughly cleaned).

Areas of impact damage (e.g. dropped goods) or scouring (e.g. dragged fork tines) should be treated to prevent further degradation under trafficking. Scraping of pallets or tines will damage the surface and, especially, the arris of any joints. Pushing of pallets and steel stillages should be avoided and pallets should be well maintained as protruding nails or timber shards can lead to significant surface damage. Under-chassis stabilisers on trucks should be adjusted to prevent dragging when manoeuvring.

If heavy goods (e.g. paper rolls or steel sections) are dropped on the floor, serious cracking may result, requiring a section of slab to be removed and reinstated.

13.4 Joints

Joints typically require most attention in any maintenance plan. The exposed edges of any joint in a concrete floor are prone to damage or wear and protective measures are needed to prevent serviceability issues. For day or formed joints, typically 10mm thick steel plate armouring is cast into the concrete during construction. For sawn-induced joints, sealant of varying hardness can be installed at any time. All joints are susceptible to wear from trafficking, especially by small, hard wheels and an unprotected joint arris will suffer significant damage if left unprotected and unmaintained.

13.4.1 Joint inspection

Joints should be regularly inspected for signs of wear, damage or split/debonded sealant. The ability of the sealant to protect the joint arris should be assessed. Deterioration in the sealant should be treated quickly before significant damage to the joint arris occurs. Any arris damage that has occurred should be quickly repaired as deterioration will accelerate once it has started. It may be necessary to replace joint sealant in more heavily trafficked areas, more frequently, e.g. defined-movement aisles or collation areas.

13.4.2 Joint sealant

Soon after the slab is constructed, a 'soft' elastomeric sealant is normally installed to the sawn-induced joints; this material permits a degree of movement or stretching as the joint opens but offers little protection to the joint itself. Once the sealant reaches the limit of its elasticity, it will split or debond and should be replaced under general maintenance.

Normally, the specification will require replacement of this initial 'soft' sealant with a 'hard' material that can provide significantly more protection but is susceptible to minor joint opening. Generally there is a balance between ability to accommodate joint opening and hardness of the sealant, i.e. a tough 'hard' sealant will not accommodate significant joint opening but a 'soft' sealant can at the expense of arris protection. See Section 11.12. There are, however, sealants that can offer higher protection whilst having the same movement accommodation as softer materials (e.g. one-part high-modulus polymer sealants). Sealants for use in chemical exposure or cold store environments should be specific for their use and manufacturers should be consulted.

13.4.3 Joint deterioration

Slight ravelling or wear of the joint arris will occur under repeated trafficking and/or insufficient support from the mastic sealant (either because the sealant is too 'soft' or not in proper contact with the joint itself). Minor wear will not affect serviceability of the floor and joint but will need regular inspections and assessment of deterioration.

Sealant installation (or replacement) can fill in smaller areas of ravelling (e.g. max 10mm) but larger or more significant wear/damage should be repaired using an epoxy or resin material with a proven cut-back and fill method. The ability of a given joint to resist wear is also related to how wide the joint has opened and the relative size of the wheeled traffic it receives. The wider a joint opens and the smaller the wheels used on the floor, the greater propensity there is for joint damage. Small hard wheels, often found on collation trolleys or small pallet movers, can inflict significant damage despite being relatively lightly loaded.

13.5 Cracks

As with joints, any cracks that develop should be monitored and, where appropriate, repaired. Durability of a trafficked crack arris is subject to the same wear characteristic of a trafficked joint with the same relationship to opening and movement. With regard to serviceability, if a crack withstands trafficking without wear, it may be better to leave it untreated. Cracks should be monitored as part of the normal floor inspection and maintenance procedures.

It is important to remember that in the case of shrinkage (restraint) cracks, the crack opening can result in smaller openings at joints, simply transferring the maintenance attention accordingly. If the arris of a crack begins to spall or ravel, it should be treated to prevent further deterioration (in the same manner as discussed regarding joints).

However, the requirement to treat or repair a crack should be balanced against the dormant status of the crack, i.e. ideally the crack should not be subject to further opening after treatment as a hard, durable sealant/resin material will perform well under trafficking but will not accommodate future opening. Providing wear at the crack arris does not hamper floor use nor the opening of a crack lead to structural issues, it is advisable to leave treatment as late as possible (e.g. end of the period of defects liability for a new floor) as the treatment will remedy the arris damage and restore serviceability. Repeated treatment of the same crack whilst the floor continues to shrink can lead to a less effective repair in the long run. Where cracks are not dormant but some arris support is considered essential, semi-flexible sealants can be used.

Cracks may be separated into two classes (see Concrete Society Technical Report 22, *Non-structural cracks in concrete*[72]) for the purpose of deciding on potential repair:

- Dormant cracks which are unlikely to open, close or extend further. The crack widths (minimum value throughout the crack depth) can be subdivided as follows:
  - fine cracks: <0.5mm wide (full aggregate interlock and load transfer)
  - medium cracks: 0.5–1.5mm wide (partial load transfer; approximately 15% at 1.5mm)
  - wide cracks: >1.5mm wide (limited or no load transfer).
- Live cracks which may be subject to further movement, due to changes in the temperature and/or moisture state of the concrete, loading etc.

13.6 Inspection and action schedule

The following are guidelines for when inspections and treatment should be carried out based on a typical warehouse with average usage (e.g. a wide aisle rack based warehouse with a marshalling area in front of loading docks, working a 12-hour shift, 6 days per week). The more intense the working of the warehouse floor, the shorter the intervals between actions.

Daily:

- Cleaning regime to remove dust, dirt and debris.
- Use floor scrubber or vacuum scrubber drier.

Every 3 months:

- General and visual inspection of trafficked areas.
- Repair any spalling or ravelling of joint edges and replace joint sealant (as required).

Every 12 months:

- Inspection and report including typical photographical evidence of the floor's condition.
- Replace sealant in floor joints or cracks if debonded or split due to movement (as required).

Every 5 years:

- Thoroughly clean the floor, remove tyre marking and surface sealer issues and reseal the surface.

13.7 Applied coatings

The application of resin or painted coatings will be subject to their own cleaning and maintenance recommendations; these should be provided by the manufacturer or installer of the coating. It may be necessary to reapply paint coatings periodically as they will wear under trafficking. Line markings will also wear under trafficking and should be regularly inspected and maintained. Some line marking involves shot blasting the surface in preparation and it is important to seal any exposed shot blasted areas to prevent accelerated wear of the concrete.

13.8 Textured surface

Acid etching or shot blasting at the surface of the floor can be used to increase slip resistance (e.g. near vehicle external doors or in wet environments); however, under vehicle trafficking, these textured surfaces will wear smooth and will require further attention to restore and maintain the desired level of roughness. With minor shot blasting designed to expose the fine aggregate and sand, there will be a limited number of times this process can be carried out before larger aggregate is exposed and the desired finish can no longer be provided. This will vary according to aggressiveness of the process, vehicle use and material performance of the base concrete.

It should be noted that cleaning of a floor with a textured surface will be more difficult than a smooth power-trowelled finish.

13.9 Repair

The repair of concrete structures is covered by EN 1504 [73]. The various parts of the Standard cover both the requirements for the repair materials and for the methods of application. Further guidance is given in Concrete Society Technical Report 69, *Repair of concrete structures with reference to BS EN 1504* [74].

13.10 General tips and advice

To maintain the appearance and service life of the floor, the following basic tips are recommended.

Good practice:

- Clean regularly.
- Remove debris before it causes damage.
- Give higher frequency of maintenance and care to heavily trafficked areas.
- Clean up spillages immediately.
- Remove oil and grease immediately.
- Install spill and clean-up kits at regular locations.
- Ensure cleaning agents are suitable for concrete surfaces – trial areas before use.
- Follow instructions from manufacturers.
- Remember all floors need maintenance once in use.

Bad practice:

- Using excess concentrations of cleaning agents.
- Mixing cleaning chemicals and agents.
- Ignoring initial and minor joint damage – get it treated to prevent more significant issues.
- Using aggressive brushes on cleaning equipment.
- Leaving brush heads in lowered position while machine is stationary.
- Feathering out or using thin layers of repair materials – cut vertical and reinstate with recommended layer thickness.
- Using acid or alkali cleaning agents – over time, damage will occur.

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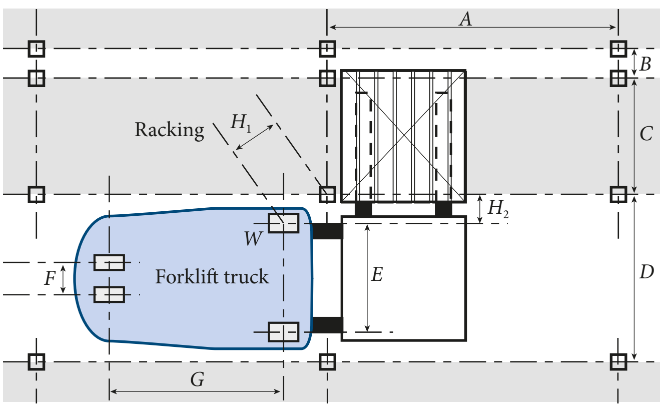
Appendix A: Model design brief for concrete industrial ground-floors

Area name/description .... Planned use ....

PART ONE: GENERAL INFORMATION

LOAD TYPE	DATA REQUIRED	VALUE	UNITS
Pallet racking	Single upright load		kN
	Back-to-back spacing *B*		mm
	Rack depth *C*		m
	Rack length *A*		m
	Aisle width *D*		m
	Upright to MHE wheel spacing (maximum static load) H_1		mm
	Upright to MHE wheel spacing (maximum moving load) H_2		mm
MHE	Maximum static wheel load *W*		kN
	Wheel contact area		mm^2
	Load axle width *E*		m
	Rear axle width *F*		m
	Front to rear axle length *G*		m
	No. of passes for fatigue (if required)		no.
UDLs	Load per square metre		kN/m^2
	Aisle width if to be fixed		m
	Load width if to be fixed		m
Line loads	Load per linear metre		kN/m
Mezzanine	Mezzanine column load		kN
	Spacing		m × m
	Baseplate size		mm × mm
Other loads			



Key
- ***A*** Upright spacing along rack
- ***B*** Back-to-back upright spacing
- ***C*** Upright spacing across rack
- ***D*** Upright spacing across aisle
- ***E*** Truck load wheel spacing
- ***F*** Truck drive wheel spacing
- ***G*** Truck wheel base
- H_1 Distance of truck wheel from rack upright when the wheel load *W* is at its maximum value
- H_2 Distance of truck wheel from rack upright when the truck is in motion
- ***W*** Maximum wheel load

PART TWO: SURFACE REQUIREMENTS CHECKLIST

	Check or N/A
Abrasion resistance	
Chemical resistance	
Colour and appearance	
Slip resistance	
Fibre visibility at the surface	
Joint types, layout and spacing	
Special requirements:	

Flatness	

PART THREE: GENERAL

Floor to be loaded ………………… days after construction
Operating temperature/range ………………………………………………

Environmental considerations
(e.g. ground conditions, gas venting)

Other

Appendix B: Chemical attack

B1. Introduction

Well-designed and constructed concrete will perform satisfactorily when exposed to many kinds of chemical. However, in some chemical environments the useful life of even the best concrete will be shortened unless specific measures are taken. It is important to remember that concrete is rarely attacked by solid, dry chemicals. For significant attack to occur, the chemical must be in solution and sufficiently concentrated or reach a critical concentration after evaporation of the solution.

B2. Sulfates

All sulfates are potentially harmful to concrete. Sulfates occur naturally in soils, rocks and groundwater. For example, gypsum (calcium sulfate) is present in some clay soils in the south of England. Soils and waters containing sulfates are often described as 'alkali'.

Sulfate attack on hardened concrete generally appears in two forms:

- Expansive formation of ettringite or gypsum in the hardened concrete causing cracking and exfoliation.
- Softening and dissolution of the hydrated cementing compounds due to direct attack on these compounds by sulfate or by their decomposition when calcium hydroxide reacts with the sulfates and is removed.

Either or both of these can occur, depending on the temperature, types and concentrations of sulfate available for reaction and the composition of the concrete.

Potentially the most serious problem with sulfate is that it can be drawn upwards into the slab from the sub-base or subgrade by the wicking action caused by drying from the slab surface. Sulfate attack can cause slab heave. Care is needed if waste or recycled materials are used for sub-bases which contain sulfate or sulfide which could oxidise to sulfate. If present, the slab should be isolated from the source of sulfate by an effective membrane.

The literature on sulfate attack is complex and confusing and there is no consensus on some of the mechanisms. For more information, sources such as *Lea's chemistry of cement and concrete*[75] and BRE Special Digest 1[76] may be consulted.

B3. Chlorides

The most significant and common sources of chlorides are marine environments and de-icing salts applied to road surfaces in winter, which may be brought into the building on vehicle wheels. Chlorides have little effect on hardened concrete but they increase the risk of reinforcement corrosion.

B4. Physical salt weathering

Salt weathering is designated in the literature as 'physical salt weathering' to distinguish it from reactions of concrete with chlorides. The mechanism is similar to freezing and thawing of water in concrete in that salts (usually salts of sulfates and possibly chlorides) crystallise in the pores of the concrete close to its surface. The crystal growth exerts pressure in a similar manner to ice forming within the pores.

Salt in solution from groundwater or damp soil is transported by capillary action vertically through the concrete member. Above ground level, the moisture is drawn to the surface and evaporates, leaving crystals of salt growing in the near-surface pores. The result is an area of deterioration just above ground level. This form of attack is common in hot, dry areas and may also occur in marine structures.

B5. Acids and alkalis

The hydrated and unhydrated cement compounds and calcareous aggregates in concrete are attacked by most acids to a greater or lesser extent. Strong alkalis can also be a problem and can attack siliceous aggregate, however disintegration is typically slow. Table B1 indicates the rate of attack of various common acids. Acids may become more concentrated due to evaporation, leading to an increased rate of attack. It is important to realise that many acids are the products of reactions of other substances which may, in themselves, be harmless to concrete or may only be present in low concentrations in these substances.

Table B1: Rate of attack of concrete by acids (after PCA [77]).

Effect on concrete	Inorganic (mineral) acids	Organic acids
Rapid	Hydrochloric Nitric Sulfuric	
Slow	Carbonic Phosphoric	Acetic* Formic* Humic Lactic** Tannic
Negligible	–	Oxalic Tartaric

* Slow although greater when concentrated. ** Negligible, as milk.

B6. Other substances

Some common substances that may come into contact with concrete, particularly when the structure is used for processing or storage, are shown in Table B2. The information is based on *Effects of substances on concrete and guide to protective measures*[77] which gives an exhaustive list of materials and protective measures. In addition, information is given in Lea's chemistry of cement and concrete[75]. In all cases the effects described should only be seen as indicative as they depend on the concentration of the products formed.

Table B2: Effect of some common substances on concrete (after PCA).[77]

Material	Comment on products formed	Effect
Ashes/cinders	If wet, sodium sulfate may leach out	Disintegrate concrete without adequate sulfate resistance
Beer	Fermentation products may contain acetic, carbonic, lactic or tannic acids*	Disintegrates concrete slowly
Cider	Contains acetic acid*	Disintegrates concrete slowly
Coal	Sulfides leaching from damp coal may form sulfurous or sulfuric acid*	None unless sulfides present, then disintegrates concrete rapidly
Common salt		Not harmful to dry concrete. Harmful to embedded steel in the presence of moisture
Creosote	Contains phenol	Disintegrates concrete slowly
Exhaust gases (diesel or petrol)	Form various acids in the presence of moisture	Disintegrate concrete slowly
Flue gases	Form various acids in the presence of moisture	Disintegrate concrete slowly; temperature differentials may cause significant stresses
Fruit juices (and fermenting fruit)	Contain sugar and various acids	Disintegrate concrete slowly
Manure		Disintegrates concrete slowly
Milk	Not harmful unless sour, which contains lactic acid*	Disintegrates concrete slowly
Peaty water	Contains humic acid	Disintegrates concrete slowly
Petroleum oils		Disintegrate concrete slowly, if fatty oils are present
Silage	Contains a wide range of acids	Disintegrates concrete slowly
Sugars		Disintegrate concrete slowly
Urine		Attacks steel in porous or cracked concrete
Wine	Not harmful. Solutions in process may cause slow disintegration	Negligible
Wood pulp		Negligible

* See Table B1

Appendix C: Rigorous assessment of moment capacity of fibre-reinforced section, with and without supplementary fabric or bar reinforcement

Assessment of the moment capacity is based on the simplified stress–strain relationship. The ultimate moment capacity is dependent on the strain at the extremity of the section. On the compression face, the strain is limited to 0.0035, as is the case for conventional reinforced concrete sections. On the tension face, the strain is limited to 0.025.

Assessing the true ultimate load capacity of a statically indeterminate structure requires a non-linear analysis, which is overly complicated for everyday use. In this report, a simplified approach is taken. The moment – crack width (M-w) response of the section is derived in terms of the residual strengths f_{R1} and f_{R4} obtained from the BS EN 14651 beam test. f_{R1} and f_{R4} represent the flexural tensile stresses at a Crack Mouth Opening Displacement (CMOD) of 0.5mm and 3.5mm respectively in the 150mm deep test beam. Although in sections deeper than 150mm, the strain at a CMOD of 3.5mm will be lower than in the test beam, the maximum tensile strain is set at the value resulting from a CMOD of 3.5mm, subject to a limiting maximum strain of 0.025.

For a slab with a low (cracked) flexural tensile capacity, the compressive strain in the concrete may remain in the elastic range, below 0.00175, in which case the concrete stress block is triangular. As the flexural tensile capacity increases, by increasing the dosage or performance of the fibres or adding fabric or bar reinforcement to the section, the compressive strain in the concrete increases and the compressive stress block becomes bi-linear , as shown in Figures C1.1 and C2.1. Where a particular fibre behavious exhibits strain hardening the stress strain relationship is modified as in Figure C1.2 and C2.2.

The characteristics exhibited by the stress strain softening and hardening are illustrated in the following stress and strain diagrams at the ultimate moment. Based on these stress and strain diagrams, the moment capacity of the section, where w = 3.5mm, can be calculated. Note that the compressive stresses and strains are considered positive in the following expressions.

This method of assessing the moment capacity of steel fibre reinforced sections is not valid for sections exceeding a depth of 600mm or where $h_{ux} > 0.3d$, as such sections may not be sufficiently ductile to be safe for Yield Line analysis.

Strain softening

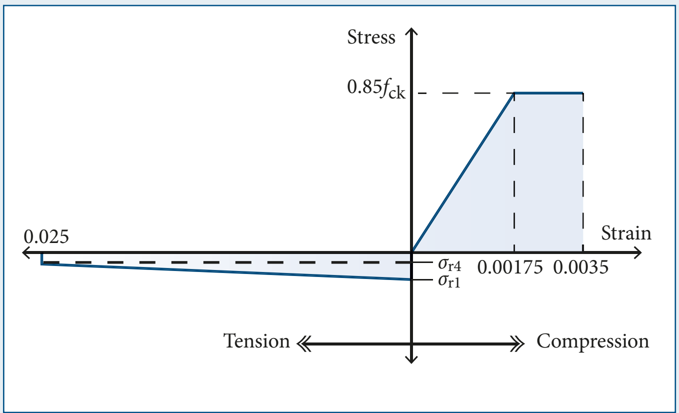


Figure C1.1: Simplified stress–strain relationship for strain softening.

Strain hardening

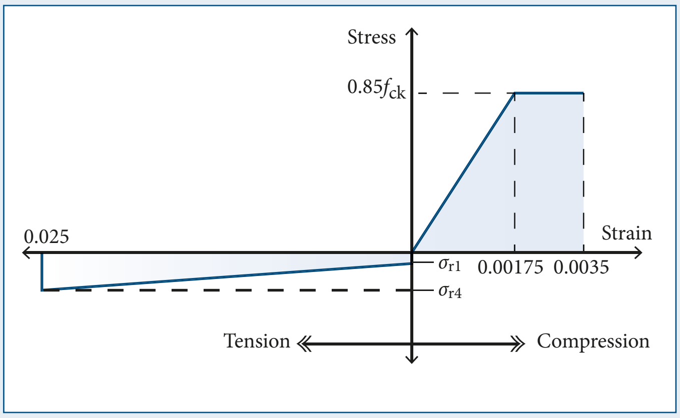


Figure C1.2: Simplified stress–strain relationship for strain hardening.

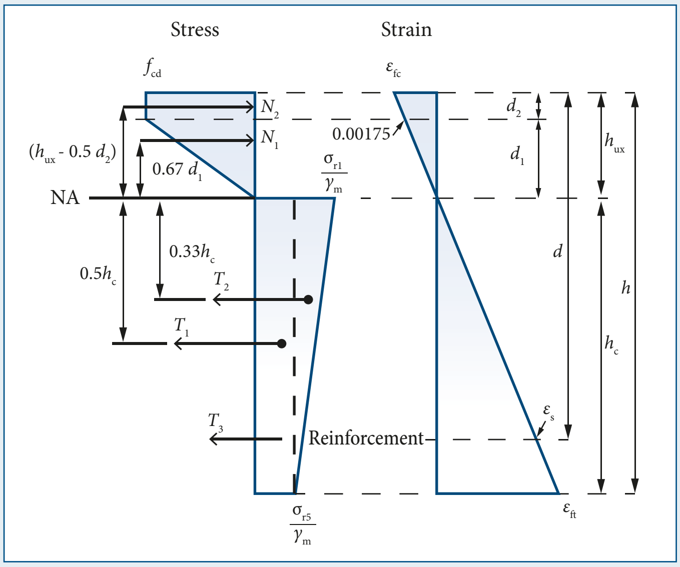


Figure C2.1: Stress and strain diagram for bi-linear stress block for strain softening.

$$\varepsilon_{ft} = \frac{3.5}{h_c} \leq 0.025 \qquad \text{(Equation C1a)}$$

$$\varepsilon_{fc} = \varepsilon_{ft}\left(\frac{h_{ux}}{h_c}\right) > 0.00175 \text{ but } \leq 0.0035 \qquad \text{(Equation C2a)}$$

$$f_{cd} = \frac{0.85 f_{ck}}{\gamma_m} \qquad \text{(Equation C3a)}$$

$$\sigma_{r1} = 0.45 f_{r1} \qquad \text{(Equation C4a)}$$

$$\sigma_{r4} = 0.37 f_{r4} \qquad \text{(Equation C5a)}$$

$$\sigma_{r5} = \sigma_{r1} - \left(\frac{\varepsilon_{ft}}{0.025}(\sigma_{r1} - \sigma_{r4})\right) \qquad \text{(Equation C6a)}$$

$$d_1 = h_{ux}\left(\frac{0.00175}{\varepsilon_{fc}}\right) \qquad \text{(Equation C7a)}$$

$$d_2 = h_{ux} - d_1 \qquad \text{(Equation C8a)}$$

$$N_1 = 0.5\, d_1\, b\, f_{cd} \qquad \text{(Equation C9a)}$$

$$N_2 = d_2\, b\, f_{cd} \qquad \text{(Equation C10a)}$$

$$T_1 = b\, h_c\, \sigma_{r5}/\gamma_m \qquad \text{(Equation C11a)}$$

$$T_2 = 0.5b\, h_c\, (\sigma_{r1} - \sigma_{r5})/\gamma_m \qquad \text{(Equation C12a)}$$

$$T_3 = A_s E_s\left(\frac{d - h_{ux}}{h_{ux}}\right)\varepsilon_{fc}/\gamma_s \leq A_s f_{yk}/\gamma_s \qquad \text{(Equation C13a)}$$

where b is unit width, typically = 1

Horizontal forces equilibrium:

$$N_1 + N_2 = T_1 + T_2 + T_3 \qquad \text{(Equation C14a)}$$

Bending moment equilibrium:

$$M_u = N_1\, 0.67 d_1 + N_2(h_{ux} - 0.5 d_2) + 0.5\, h_c T_1 + 0.33 h_c\, T_2 + T_3\,(d - h_{ux})$$

(Equation C15a)

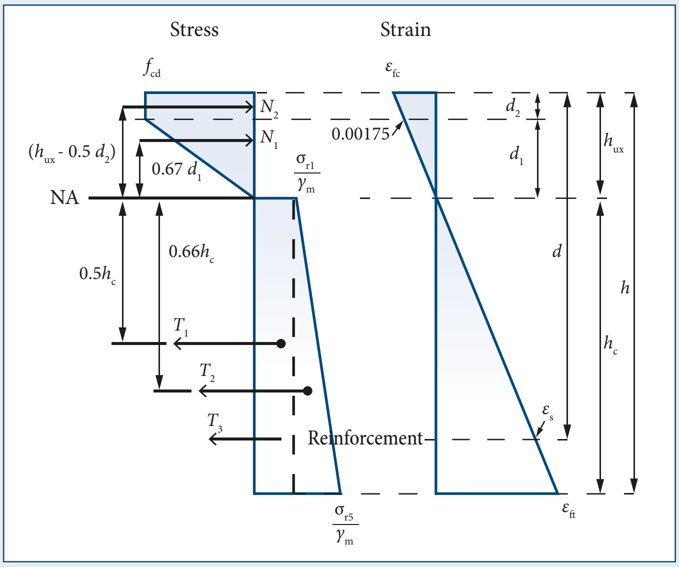


Figure C2.2: Stress and strain diagram for bi-linear stress block for strain hardening.

$$\varepsilon_{ft} = \frac{3.5}{h_c} \leq 0.025 \qquad \text{(Equation C1b)}$$

$$\varepsilon_{fc} = \varepsilon_{ft}\left(\frac{h_{ux}}{h_c}\right) > 0.00175 \text{ but } \leq 0.0035 \qquad \text{(Equation C2b)}$$

$$f_{cd} = \frac{0.85 f_{ck}}{\gamma_m} \qquad \text{(Equation C3b)}$$

$$\sigma_{r1} = 0.45 f_{r1} \qquad \text{(Equation C4b)}$$

$$\sigma_{r4} = 0.37 f_{r4} \qquad \text{(Equation C5b)}$$

$$\sigma_{r5} = \sigma_{r1} + \left(\frac{\varepsilon_{ft}}{0.025}(\sigma_{r4} - \sigma_{r1})\right) \qquad \text{(Equation C6b)}$$

$$d_1 = h_{ux}\left(\frac{0.00175}{\varepsilon_{fc}}\right) \qquad \text{(Equation C7b)}$$

$$d_2 = h_{ux} - d_1 \qquad \text{(Equation C8b)}$$

$$N_1 = 0.5\, d_1\, b\, f_{cd} \qquad \text{(Equation C9b)}$$

$$N_2 = d_2\, b\, f_{cd} \qquad \text{(Equation C10b)}$$

$$T_1 = b\, h_c\, \sigma_{r1}/\gamma_m \qquad \text{(Equation C11b)}$$

$$T_2 = 0.5b\, h_c\, (\sigma_{r5} - \sigma_{r1})/\gamma_m \qquad \text{(Equation C12b)}$$

$$T_3 = A_s E_s\left(\frac{d - h_{ux}}{h_{ux}}\right)\varepsilon_{fc}/\gamma_s \leq A_s f_{yk}/\gamma_s \qquad \text{(Equation C13b)}$$

where b is unit width, typically = 1

Horizontal forces equilibrium:

$$N_1 + N_2 = T_1 + T_2 + T_3 \qquad \text{(Equation C14b)}$$

Bending moment equilibrium:

$$M_u = N_1\, 0.67 d_1 + N_2(h_{ux} - 0.5 d_2) + 0.5\, h_c T_1 + 0.66 h_c\, T_2 + T_3\,(d - h_{ux})$$

(Equation C15b)

Appendix D: Derivation of dowel load transfer equations

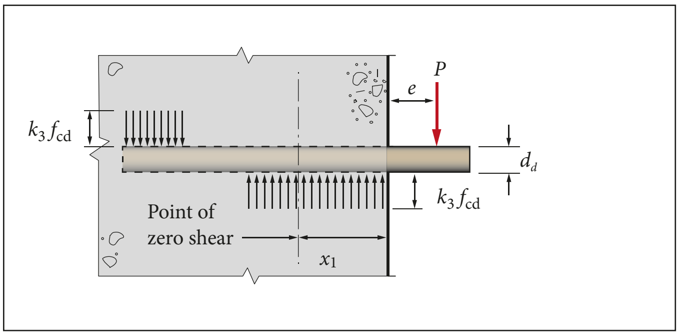


Figure D1: Dowel internal and external forces.

D1. Round dowel bars

Bar diameter = d_d

$f_{cd} = f_{ck}/\gamma_c$

$f_{yd} = f_{yk}/\gamma_s$

Confined compression factor $k_3 = 3$

Maximum moment occurs at point of zero shear

$$x_1 = P/(k_3 f_{cd}\, d_d) \qquad \text{(Equation D1)}$$

Moments to the right of zero shear

$$M = P(e + x_1/2) \qquad \text{(Equation D2)}$$

Dowel capacity in bending

$$M_p = d_d^3 f_{yd}/6 \qquad \text{(Equation D3)}$$

Substituting D1 into D2 and equating to D3:

$$P_{max}\{e + [P_{max}/(k_3 f_{cd}\, d_d 2)]\} = d_d^3 f_{yd}/6 \qquad \text{(Equation D4)}$$

Therefore:

$$P_{max}^2/(k_3 f_{cd}\, d_d 2) + P_{max}\, e\, k_3 f_{cd}\, d_d\, 2 - [k_3 f_{cd} f_{yd}\, d_d^4]/3 = 0 \qquad \text{(Equation D5)}$$

Substituting:

$$\alpha = 3e\,(f_{cd}/f_{yd})^{0.5}/d_d \qquad \text{(Equation D6)}$$

$$P_{max}^2 + [P_{max} 2\, k_3\, d_d^2\, \alpha\, (f_{cd} f_{yd})^{0.5}]/3 - [k_3 f_{cd} f_{yd}\, d_d^4]/3 = 0 \qquad \text{(Equation D7)}$$

Therefore:

$$P_{max} = d_d^2\, (f_{cd} f_{yd})^{0.5}[(k_3/3 + (k_3/3)^2\, \alpha^2)^{0.5} - k_3\, \alpha\, /3] \qquad \text{(Equation D8)}$$

where $k_3 = 3$

$$P_{max\ dowel} = [d_d^2\, (f_{cd} f_{yd})^{0.5}][(1 + \alpha^2)^{0.5} - \alpha] \qquad \text{(Equation D9)}$$

D2. Plate dowels of constant cross-section

Plate width	$= p_b$
Plate thickness	$= t_p$
Plate steel design yield strength	$= p_y$
Confined compression factor k_3	$= 3$

Maximum moment occurs at point of zero shear

$$x_1 = P/(k_3 f_{cd}\, p_b) \qquad \text{(Equation D10)}$$

Moments to the right of zero shear

$$M = P\,(e + x_1/2) \qquad \text{(Equation D11)}$$

Plate capacity in bending

$$M_p = t_p^2 p_b\, p_y/4 \qquad \text{(Equation D12)}$$

Substituting D10 into D11 and equating to D12:

$$P_{max}\{e + [P_{max}/(k_3 f_{cd}\, p_b 2)]\} = t_p^2\, p_b\, p_y/4 \qquad \text{(Equation D13)}$$

Therefore:

$$P_{max}^2 + P_{max}\, e\, k_3 f_{cd}\, p_b 2 - [k_3 f_{cd}\, p_b^2\ t_p^2\ p_y]/2 = 0 \qquad \text{(Equation D14)}$$

Substituting:

$b1 = 2e\, k_3 f_{cd}\, p_b$

$c1 = 2 \times k_3 f_{cd}\, p_b^2\ t_p^2\ p_y$

$k_3 = 3$

$$P_{max\ plate} = 0.5\,[(b_1^2 + c_1)^{0.5} - b_1] \qquad \text{(Equation D15)}$$

Appendix E: Fatigue design check for MHE load repetitions on ground-supported floors

In addition to designing the slab to resist the maximum static loads as described in Section 7, the following check on 'fatigue life' of the slab is recommended where heavy wheel loads from material handling equipment will repeatedly traffic areas of the slab.

This is an empirical approach derived from TRRL Research Report 87 *Thickness design of concrete roads*[78], and is the basis of the thickness design for external hardstanding slabs in Concrete Society Technical Report 66 *External in-situ concrete paving*[79]. The following limitations apply to this method:

- The maximum static axle load should not exceed 25,000kg.
- The 'equivalent foundation modulus' of the subgrade and sub-base/capping is at least 100MPa. Table E1 indicates the thickness of capping/sub-base required to achieve this foundation modulus for various subgrade California Bearing Ratios.
- Effective load transfer is required at all joints traversed by the MHE (see Section 7.9).
- The centre-to-centre dimension of wheels on the same axle is greater than the 'radius of relative stiffness' (see Section 7.5). If less, the axle load requires to be enhanced to account for the interaction of wheel loads in accordance with Figure E1. This will need to be checked after the required slab thickness has been calculated, and if necessary the calculation repeated using the enhanced axle load.

Table E1: Sub-base/capping thickness required for equivalent foundation modulus of 100MPa.

	Option 1 – Sub-base-only construction	Option 2 – Sub-base plus capping construction
Subgrade CBR: %	**Thickness of Type 1 granular sub-base: mm**	**Thickness of Type 1 sub-base + capping: mm**
2.5	450	350 + 250
3	410	320 + 240
5	320	240 + 205
10	240	180 + 170
15	200	150 + 150
Table based on Interim Advice Note 73/06 rev 1[80]		

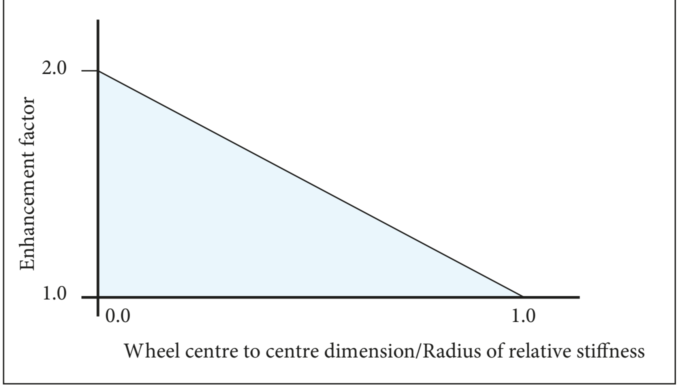


Figure E1: Enhancement factor to account for interaction of wheel loads.

Step 1

Assess the traffic loading resulting from the combination of axle load and number of repetitions of the load over any point on the slab as follows:

- Obtain the static axle load on the front and rear axle of the loaded MHE. Note that the empirical method is based on the damage caused by pneumatic tyres with tyre pressures less than 1.0N/mm^2. Vehicles with solid tyres or very high pressure pneumatic tyres cause higher bending stresses in the slab and this results in a lower fatigue life. The static axle load of MHE with solid tyres (or with pneumatic tyres with a pressure significantly greater than 1.0N/mm^2) should be multiplied by 1.3 to account for this effect.
- Assess the damaging effect of each axle relative to that of a standard axle load of 8160kg as follows:

  Number of equivalent standard axles

  $$= \{[\text{MHE axle load (kg)}] / 8160\}^4 \qquad \text{(Equation E1)}$$

- Carry out the calculation for both the front and rear axles (or each axle if more than two) and sum the results.
- Assess how many times during the design life of the slab the MHE will travel over the same point on the slab. Judgement is required, as in free-movement areas vehicles can wander over the slab and are unlikely to traffic the same point every time they pass, whereas in aisles, narrow corridors or the approach to a loading dock, vehicles will be constrained to drive over the same point in the slab every time they pass.
- The traffic loading from the cumulative traffic is the product of the number of equivalent standard axles for each type of MHE and the number of times the MHE traffics over a point on the slab, expressed as the number of 'million standard axles' (msa). If more than one type of MHE traffics the slab, the calculation is undertaken for each vehicle type and the results summed to provide a total traffic loading.

Step 2

Assess the mean compressive strength of the concrete:

Mean compressive strength (f_{cm})

$$= \text{Characteristic compressive strength } (f_{ck}) + \text{margin (N/mm}^2)$$

Take margin as 7N/mm^2 unless information available from the supplier. From an unaccredited supply a higher margin may be necessary.

Step 3

Determine the required slab thickness as follows:

For fabric-reinforced slabs with A_s < 300mm^2/m:

$$h = 1935\ (L^{0.196} / f_{cm}^{\ 0.68}) \qquad \text{(Equation E2)}$$

where

L = traffic loading, in number of million standard axles (Step 1)
f_{cm} = mean compressive strength in N/mm^2 (Step 2).

For fabric-reinforced slabs with A_s > 300mm^2/m (refer to Note 1):

$$h = 9141\ \left[L^{0.209} / (f_{cm}^{\ 0.663} \times A_s^{\ 0.296})\right] \qquad \text{(Equation E3)}$$

where

L = traffic loading, in number of million standard axles (Step 1)
F = mean compressive strength in N/mm^2 (Step 2)
A_s = area of reinforcement in mm^2 per m width.

Note 1: Where the reinforcement in each orthogonal direction differs, use the lesser value in the calculation.

Note 2: The empirical data in TRRL Research Report 87[78] relates to plain concrete slabs and fabric-reinforced concrete slabs. No data are available for fibre-reinforced concrete slabs, thus this design method should not be used for fibre-reinforced concrete slabs.

Note 3: Where less than 300mm^2/m of reinforcement is provided and the design is based on Equation E2, the reinforcement should be located in the bottom of the slab with continuity of reinforcement across the sawn joints to provide load transfer.

Note 4: Where more than 300mm^2/m of reinforcement is provided and the design is based on Equation E3, the reinforcement should be located in the top third of the slab. As the reinforcement will be cut at sawn joints, additional reinforcement should be located in the bottom of the slab with continuity of reinforcement across the sawn joints to provide load transfer.

Note 5: The 'partial load factor' applied to the static check on MHE axle loads (Section 7.2) is not applied to the axle load used in the 'fatigue' design check described above. The only exception to this is where the layout and/or operation of the building is such that all vehicle movements are constrained to pass over one point in the slab and that the majority of vehicles at this point are considered likely to turn sharply or brake hard. The fatigue design check on this local area of slab should incorporate a factor of 1.4 on the axle load to account for these dynamic effects.

Appendix F: Derivation of punching shear load reduction equation (by ground support)

Based on the simplifying assumption that the bearing pressure increases linearly from zero at some distance from the load to a peak directly under the load (see Figure F1), the proportion of the punching shear load that is carried directly to the soil is represented by the volume contained by the critical punching perimeter. The peak pressure under the load is determined from the Westergaard[24, 25] expression, multiplied by modulus of subgrade reaction *k*.

F1. To calculate radius *b*

For internal load

Volume of 'cone' of bearing pressure

= ⅓ × base area × height

$$P = 0.333\,(\pi b^2)\left(\frac{0.125P}{l^2}\right) \qquad \text{(Equation F1)}$$

where

$b = \sqrt{7.6l^2}$

$= 2.75l$

l = radius of relative stiffness (see Equation 20).

For edge load

Volume of half 'cone' of bearing pressure

= ½ × ⅓ × base area × height

$$P = 0.167\,(\pi b^2)\left(\frac{0.442P}{l^2}\right) \qquad \text{(Equation F2)}$$

where

$b = \sqrt{4.3l^2}$

$= 2.07l$

l = radius of relative stiffness (see Equation 20).

F2. To calculate ground pressure within critical perimeter

For internal load

Figure F1 shows the ground pressure within the critical perimeter for an internal load.

To simplify the calculations it is conservatively assumed that the bearing pressure at the critical perimeter is 85% of the peak bearing pressure. This is a good approximation as indicated below.

For slab thickness h = 150mm and modulus of subgrade reaction k = 0.028, the peak bearing pressure/bearing pressure ratio at the critical perimeter = 89%.

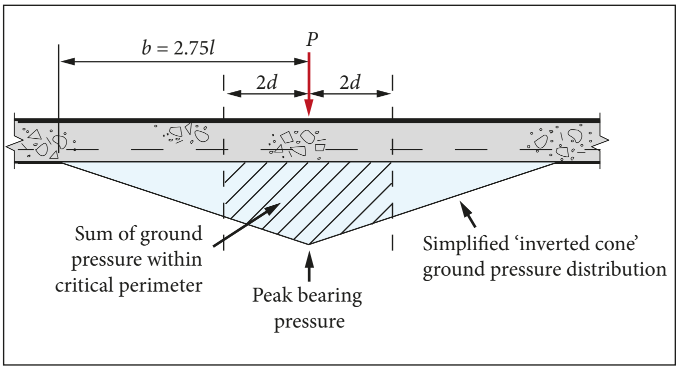


Figure F1: Pressure within the critical perimeter for an internal load.

Similarly, for:

h = 150mm and k = 0.062, the ratio is 86%
h = 300mm and k = 0.028, the ratio is 85%
h = 300mm and k = 0.062, the ratio is 82%

Proportion of P that is transferred directly in to the ground:

Peak bearing pressure $= \dfrac{0.125P}{l^2}$

85% peak Pressure $= \dfrac{0.106P}{l^2}$

Sum of ground pressure within critical perimeter: R_{cp}

$$R_{cp} = \frac{0.106P}{l^2}(2d)^2\pi + 0.333(2d)^2\pi(0.125 - 0.106)\frac{P}{l^2}$$

$$R_{cp} = 1.4\left(\frac{d}{l}\right)^2 P \qquad \text{(Equation F3)}$$

For edge load

Figure F2 shows the ground pressure within the critical perimeter for an edge load.

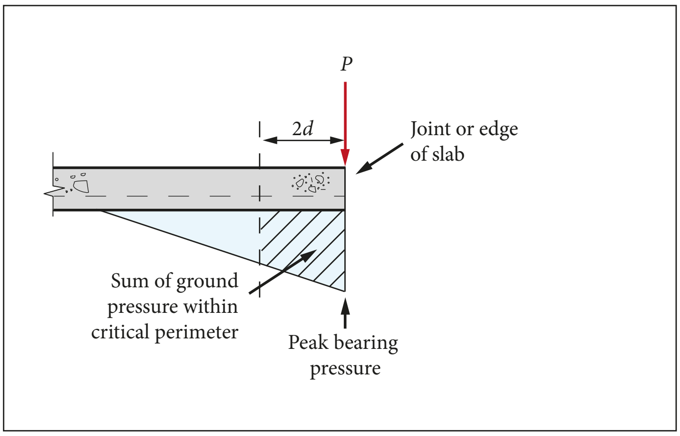


Figure F2: Pressure within the critical perimeter for an edge load.

For edge loads, the ratio of 'peak bearing pressure' to 'bearing pressure at the critical perimeter' is slightly lower at 80%.

Sum of ground pressure within critical perimeter: R_{cp}

$$R_{cp} = 0.5\left[0.8\left(\frac{0.442P}{l^2}\right)(2d^2)\pi + 0.333(2d^2)\pi(0.442-0.345)\frac{P}{l^2}\right]$$

$$R_{cp} = 2.4\left(\frac{d}{l}\right)^2 P \qquad \text{(Equation F4)}$$

F3. Additional reduction if load applied through a stiff bearing

Application of the point load through a stiff bearing increases the length of the critical perimeter and changes the shape of the ground pressure distribution. Rigorous analysis of the resulting increase in ground reaction is complex, but provided the effective radius of the baseplate (a) is small compared to the radius of relative stiffness (l), such that a/l is not greater than 0.2, a simplified analysis, as indicated in the diagrams below, provides acceptable results.

Note that to negate the potentially unconservative assumption that the peak pressure at the perimeter of the stiff bearing is the same as the peak pressure under the point load, the ground pressure directly under the bearing plate is ignored.

Figures F3 and F4 show the conditions for an internal load and edge load respectively.

For internal load

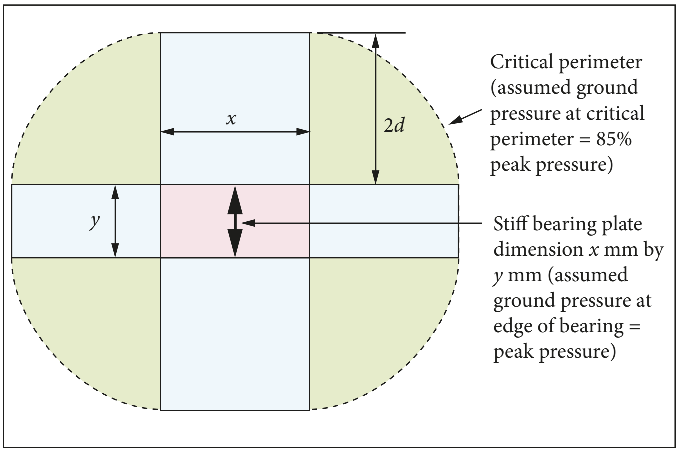


Figure F3: Stiff bearing – internal load.

$$\text{Additional } R_{cp} = 0.93[(2y2d)+(2x2d)]\frac{0.125P}{l^2}$$

$$= 0.47(x+y)\frac{dP}{l^2} \qquad \text{(Equation F5)}$$

For edge load

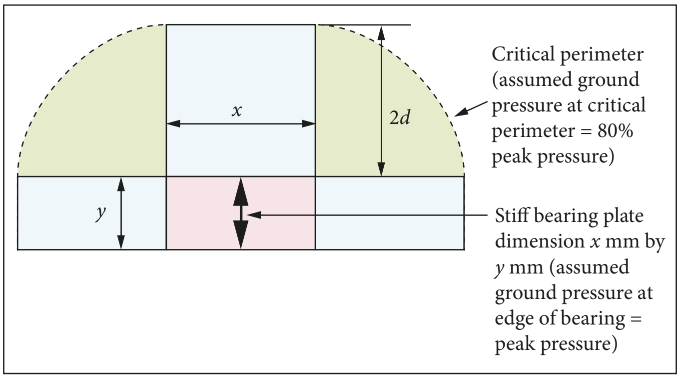


Figure F4: Stiff bearing – edge load.

$$\text{Additional } R_{cp} = 0.9[(2y2d)+(x2d)]\frac{0.442P}{l^2}$$

$$= 0.8(x+2y)\frac{dP}{l^2} \qquad \text{(Equation F6)}$$

Appendix G: Derivation of serviceability limit state equation for h_{min} in pile-supported slabs

Consider an area of slab remote from the perimeter, at the interface between a loaded and unloaded area, see Figure G1.

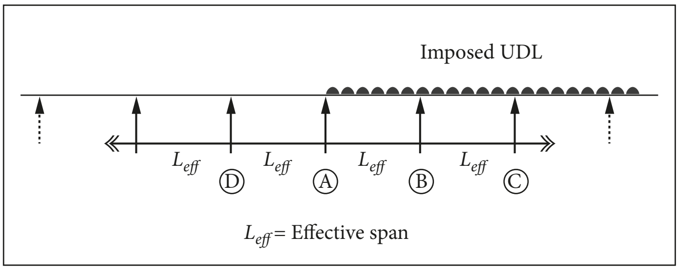


Figure G1: Interface of loaded and unloaded area, remote from the slab perimeter.

A continuous beam analysis has been undertaken to determine the maximum hogging moment.

It is recognised that the hogging moment varies across the width of the slab strip, peaking over the supporting pile. An average, rather than peak, value is used in this derivation. The true peak value is difficult to assess, particularly where the support width is significant in relation to the span dimension, and basing the equation on the theoretical peak moment would result in very thick slabs being required to ensure that the capacity of the plain concrete section is not exceeded locally over the pile.

The use of an average moment is justified on the basis that it is accepted that limited cracking may occur over the piles at serviceability limit state (SLS).

For large area slabs with a low span-to-depth ratio, research [81] indicates that compressive membrane action significantly increases the load at which cracking occurs, provided the slab is adequately laterally restrained by adjacent areas of slab.

A number of variables have been incorporated in the analysis to check the sensitivity of the moment. The results are summarised in Table G1.

Varying the concrete modulus (E_{cm}) between long-term and short-term values has only a small effect on the moments in the 'spring-supported' slabs.

Introducing a spring support reduces the maximum hogging moment. The more flexible the spring, the greater the reduction is for any given span. The spring stiffness modelled results in pile settlement and elastic shortening of only 3–3.5mm under full working load.

Based on this analysis, a bending moment coefficient of $wL/10.5$ appears reasonable.

End spans are not accounted for in this analysis, on the basis that:

- This report recommends end spans be reduced to 75% of internal spans, in which case they will not be critical for SLS hogging moment.
- End spans experience less shrinkage restraint than internal spans and are thus less susceptible to cracking.

The SLS equation is derived as follows:

Maximum negative bending moment (averaged over panel width) ≤ moment capacity of plain concrete.

$$\frac{wL_{eff}}{10.5} = \frac{bh^2_{min}}{6} F \qquad \text{(Equation G1)}$$

where

$$w = qL$$
$$b = 1000$$
$$F = f_{ctd,fl} \frac{\gamma_{m(uls)}}{\gamma_{m(sls)}}$$

The value for $\gamma_{m(uls)}$ is 1.5. The recommended value for $\gamma_{m(sls)}$ in Eurocode 2[27] is 1.0. However, as limiting cracking in a warehouse floor slab is a primary design objective, a higher value of $\gamma_{m(sls)} = 1.15$ is used.

Therefore:

$$F = 1.3 f_{ctd,fl}$$

$$h_{min} = 21L_{eff}\left(\frac{q}{f_{ctd,fl}}\right)^{0.5} \qquad \text{(Equation G2)}$$

Table G1: Analysis of maximum hogging moments in a continuous beam.

Effective span × panel width (m)	Pile support stiffness	E_{cm} (kN/m²)	Bending moment coefficient	Position of maximum negative moment	Slab lifts off support D?
2.5 × 2.5 200mm thick	Infinite	33	$wL/9.96$	B	Yes
		16	$wL/9.96$	B	Yes
	100,000kN/m	33	$wL/10.65$	C	No
		16	$wL/10.81$	C	No
3.5 × 3.5 300mm thick	Infinite	33	$wL/9.44$	B	No
		16	$wL/9.44$	B	No
	200,000kN/m	33	$wL/10.30$	C	No
		16	$wL/10.45$	C	No

This equation has been calibrated against the ultimate limit state (ULS) design equations for typical 'steel fibre only' slabs with a variety of spans and load conditions. Typically, the ULS design will govern the slab thickness, although for slabs supporting high upright loads, the SLS equation may require a marginally thicker slab to be provided.

For slabs where supplementary reinforcement or high-performance steel fibre is incorporated, the SLS equation may govern the slab thickness. This is considered an appropriate result, as the aim is to prevent overly thin slabs being specified, which are capable of supporting the design loads at ULS but may crack excessively at working loads.

Appendix H: Optimised Pile Layouts for Pile Supported Floors

Experience gained since the introduction of design guidance for pile supported floors in The Fourth Edition of TR34 in 2013 suggests that there is considerable scope for improvements in the economy of construction by better coordination of floor joints and pile layouts at the earliest possible stage in any project design.

In the UK and elsewhere, piled slabs are routinely constructed with steel fibre reinforced concrete using mechanised methods, that is to say by "Laserscreed". This method of construction provides considerable benefits in terms of speed of construction and in the resulting well-controlled surface tolerances of floors. However, the construction method is not well suited to the use of conventional reinforcement where reinforcement is needed in the top of the slab, as is usually the case with pile-supported floors. A desirable design objective is therefore to eliminate wherever possible the reliance on supplementary conventional reinforcement where the primary form of reinforcement is steel fibre.

This Appendix gives guidance on optimised pile layouts such that the floor section design remains consistent throughout the floor with no or limited variation in the reinforcement requirements.

By following simple rules notably by reducing perimeter pile spans, the need for additional conventional reinforcement can be avoided. These rules are illustrated in figures H1 to H8. However, in all cases, specific calculations should be carried out for the loads, load locations and spans.

Building perimeters

End spans at the perimeter of the floor where a ground beam or other continuous support is provided should be reduced to 75% of the main spans.

Where the perimeter of the floor is supported on piles only and the floor oversails the pile, commonly to the sheeting purlin as a cantilever, then the cantilever should be limited to 25% of the main span length. The first inner span should be limited to 75% of the main span.

All bearings, such as at edge beams or at pile caps for the main building columns, should be a minimum of 170mm to provide a minimum bearing of 150mm after allowing for 20mm of shrinkage movement.

Floor joints

It is generally the case that building owners want complete flexibility on the location of loads across the floor. Specifications commonly require that adjustable pallet racking (APR) uprights can be located to within a given distance of any joint. Common dimensions are 150mm or 300mm. This is taken to mean that the closest point to a joint will be 150mm or 300mm from the centre line of the upright to the joint. For ground-supported floors this is not usually onerous in design terms for the obvious reason that the floor transfers the load to the ground. This is not the case for piled floors and the load capacity of the floor close to joints is limited by the strength of the dowel systems used in the joints.

There are significant limitations on the strength of the dowel systems, primarily because of the need to allow for free movement at joints. Most jointing systems in common use are at this limit and there are no practical means of increasing this strength.

In practical terms, this means that rack uprights need to be kept in the order of at least 600 mm away from joints. This applies to the largest loads associated with the rows of four uprights, which are usually placed within about two metres of each other running orthogonal to the aisles.

To provide flexibility in the location of rack uprights, the simplest method is to locate the joints over the supporting piles as shown in Figure 8.9. At the same time, the pile spacing on either side of the joint should be reduced to 75% of the main spans so as to eliminate the need for supplementary reinforcement. The spans either side of the joint should be designed as "end spans".

In principle, single uprights in the down aisle direction spaced at typically about three metres can be closer to joints and the joints running down aisle could be located as suggested in Figure 8.10. However, scheme designers should take into account the possibility that racking could be turned through 90 degrees, in which case, the requirement for placing the four uprights over two metres could apply.

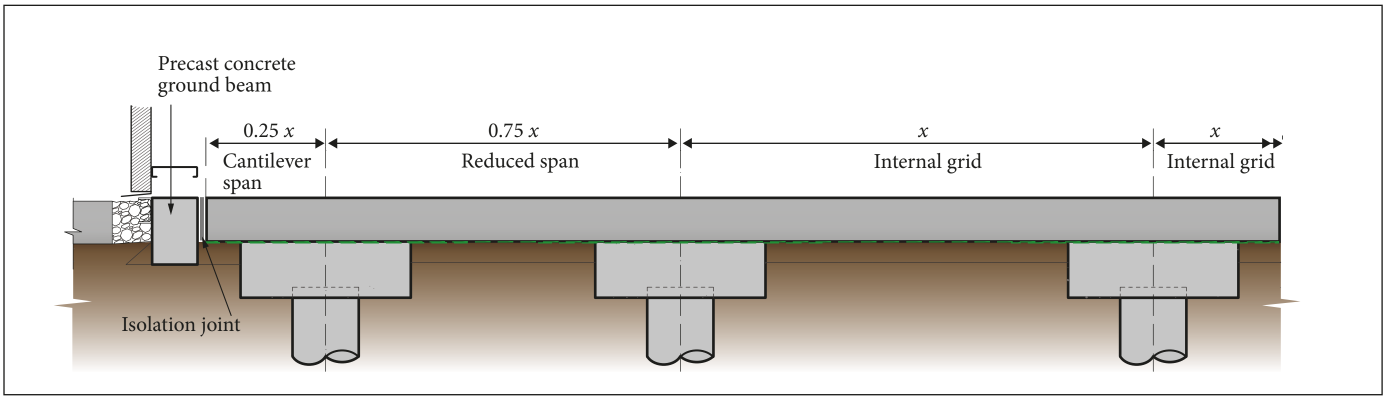


Figure H1: Pile setting out with edge span cantilever.

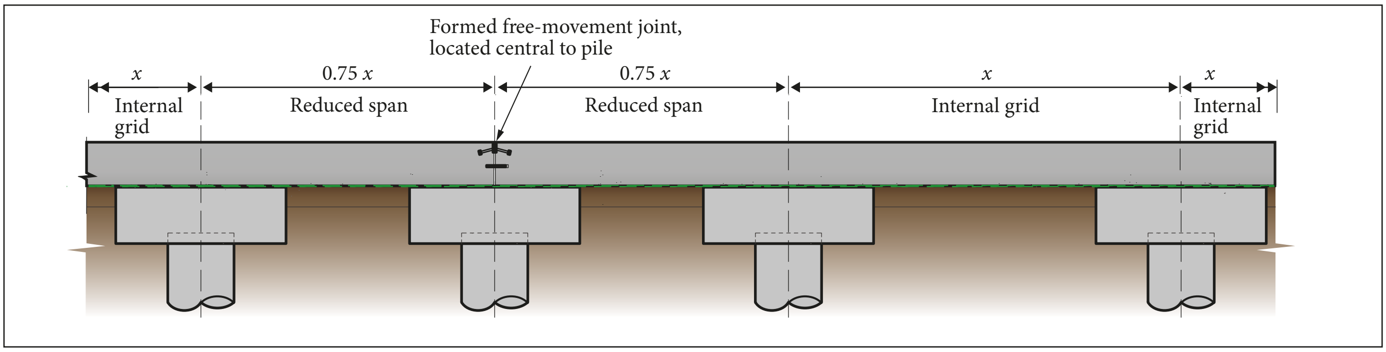


Figure H2: Pile setting out with joint central to pile.

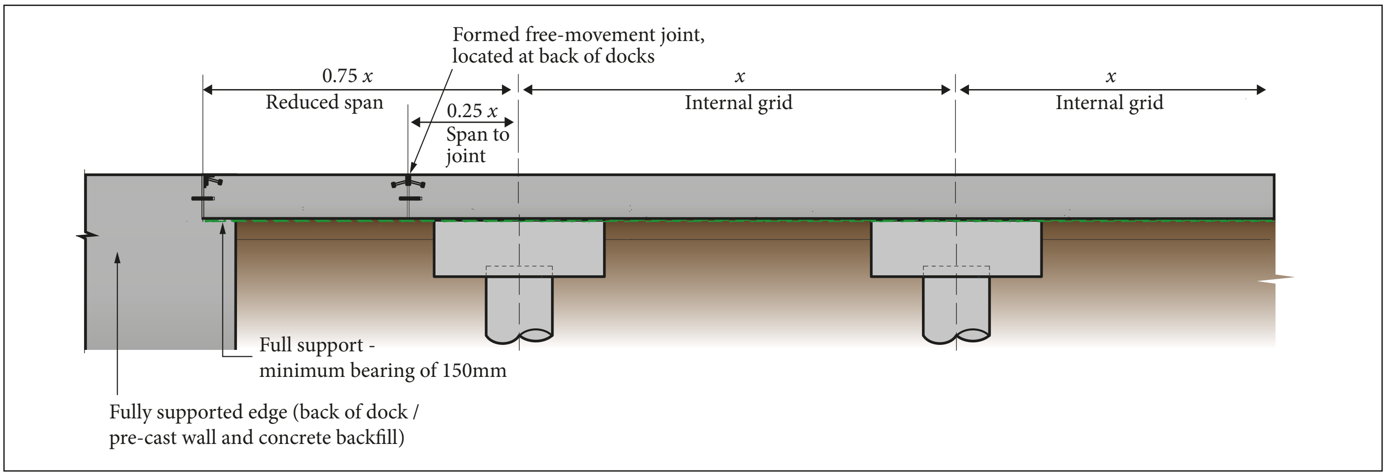


Figure H3: Pile setting out at edge span with docks.

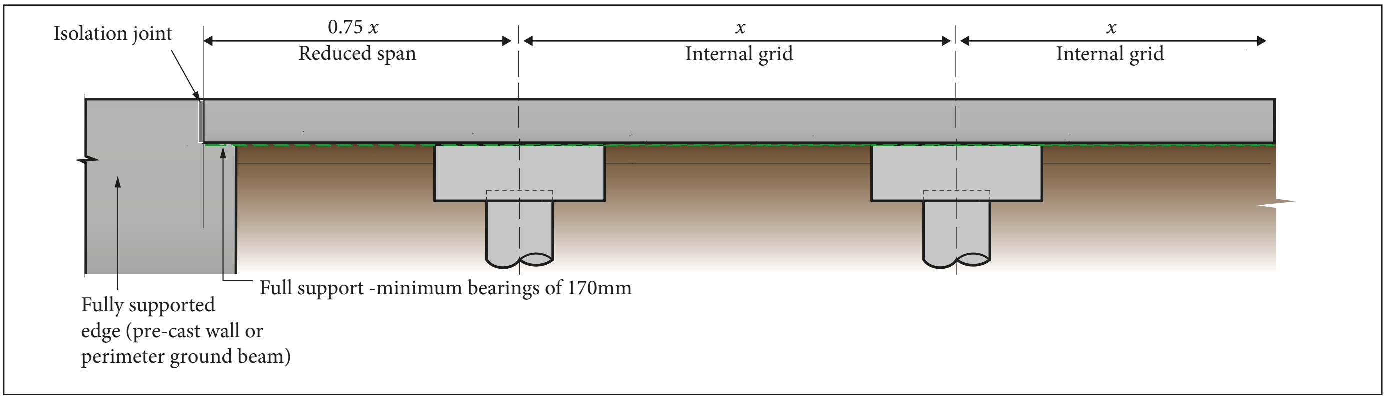


Figure H4: Pile setting out at edge span.

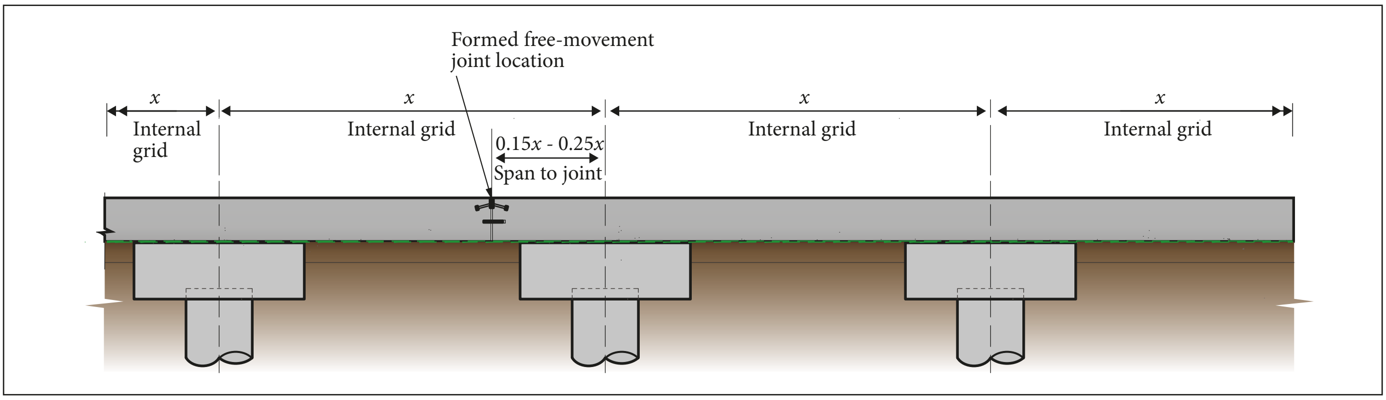


Figure H5: Pile setting out with joint location.

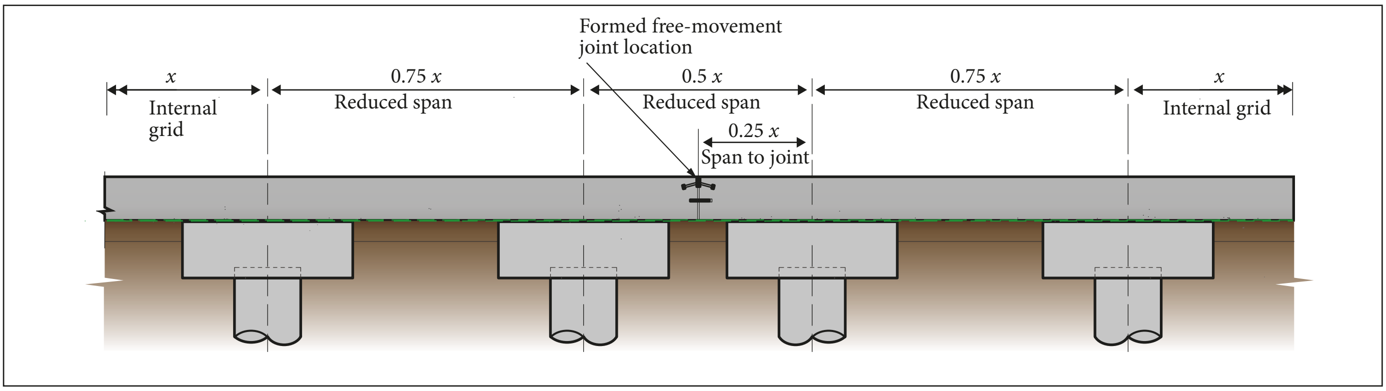


Figure H6: Pile setting out with joint central to pile span.

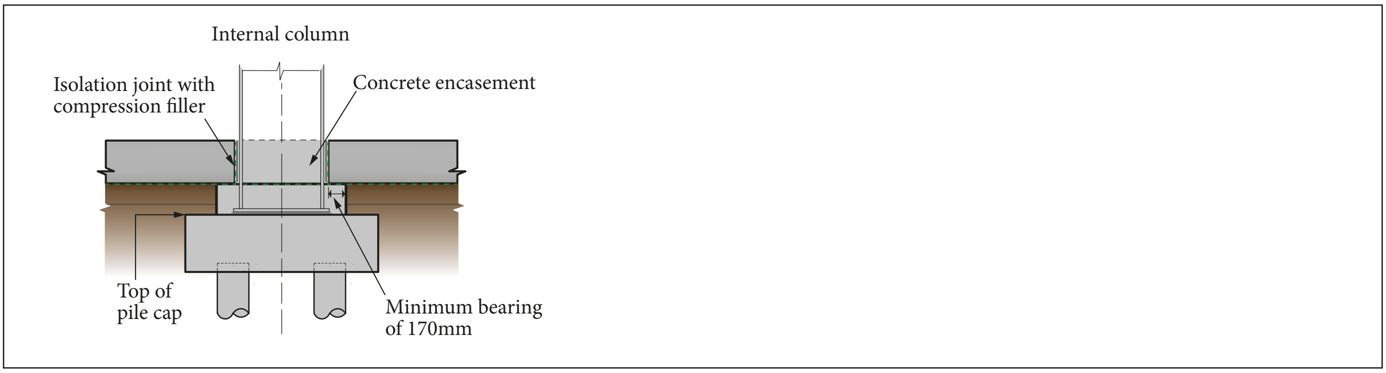


Figure H7: Internal column.

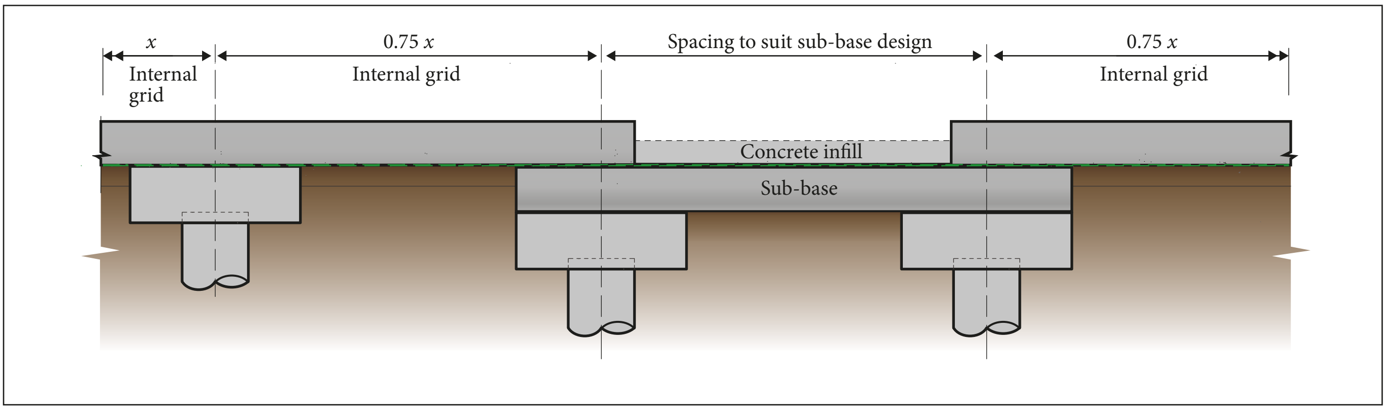


Figure H8: Pile setting out with lift pit / recess.

Appendix I: Example daily work activity check sheet

PART ONE

Project		**Building temp**	**8 am**	°C	**12 pm**	°C	**4 pm**	°C
Pour Ref		**Check by**			**Date**			

Work activity & description	✓	**Comment/detail photos taken**
Sub-base tolerance (+0 –15mm, average = 7.5mm)		
Level check (doors, interfaces & dock levellers)		
Sub-base compaction (by inspection & rerolling operation)		
Polythene (laps, level, min laid out, no damage)		
Mesh (Fabric) cover (50mm btm face, stability of spacers)		
Mesh (Fabric) laps (300mm min, tied at edges)		
Setting out (acc drawings)		
Isolation details (stable, square & taped joints)		
Joints & datum (line & level check)		
Level check against adjacent pour (datum check, line and level)		
Steel day joints (pre-cut @ s/c, rail ends nr s/c, T's & X's)		
Protection (walls, columns, dock levellers etc.)		

PART TWO

Pour times (key construction stages)	**Concreting** start		**Concreting** finish	
Finish times (key construction power-finishing stages)	**Pan/float** start		**Polish/trowel** finish	

Work activity & description	✓	Comment/detail photos taken
Fibre addition (count boxes/bags, mixer size)		
Fibre dosage (wash-out testing, 10-litre containers)		
Fibre mixing (visual check, no fibre balls, distribution)		
Consistency (slump) check (cone tests, visual checks)		
Concrete (mix spec, no balls, well mixed)		
Cubes taken (size, process, curing etc.)		
Beams taken (size, curing, location, fibres?)		
Environment (building enclosed, 3°C & rising)		
Set characteristics (floating, trowelling & finishing operations)		
Laser screed/machine laying (level chk, vibration & operation OK)		
Manual laying (straight-edge, levels, poker vibrator, bay edge)		
Curing (dosage, even cover, poly protection)		
Saw cuts (line, depth, clean cut arris, pre-cut)		
Finish quality (consistent, no marks, edges, agg rash)		

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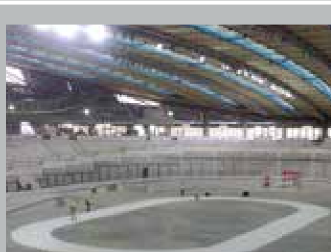

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