CONCRETE GROUND FLOORS & PAVEMENTS

FOR COMMERCIAL AND INDUSTRIAL USE

PART TWO : SPECIFIC DESIGN

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Cement & Concrete Association of New Zealand

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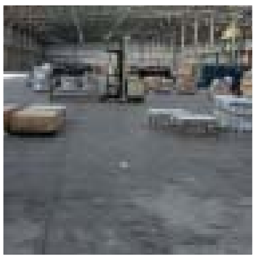

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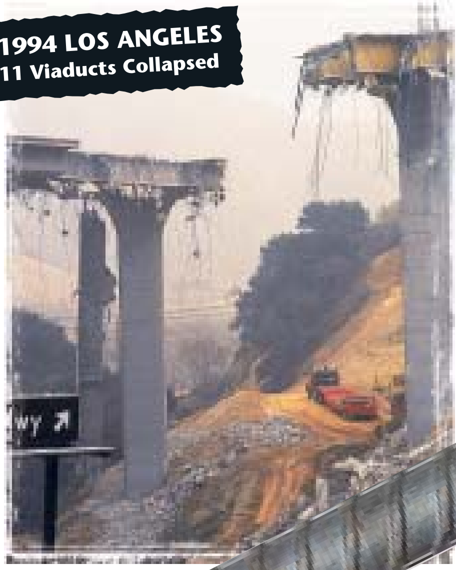
Contents

DEVASTATION.



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Preface

This design guide has been developed as a companion document to two other publications produced by the Cement & Concrete Association of New Zealand. The others being:

- *New Zealand Guide to Concrete Construction*, and
- *Concrete Ground Floors & Pavements for Commercial & Industrial Use, Part 1.*

The Part 1 design guide provides information on the design, construction and specification of industrial concrete floors on ground. A simplified design procedure is presented that limits the applied loading to light or medium loads. Table 1 summarises the definition of these loads.

Table 1 Loading limitations for the Part 1 Design Guide

Loading	Class	Limits of loading
Light	pallet racking	4 levels (one on floor) of 0.76 tonne unit loads, 4.5 tonnes end frame
	mezzanine floor	design load 3.5 kN/m²
	shelving	end frame of 4.0 tonnes
	forklift	capacity 2.0 tonnes
Medium	pallet racking	4 levels (one on floor) of 1.0 tonne unit loads, 6 tonne end frame
	mezzanine floor	design load 5.0 kN/m²
	shelving	end frame of 5.4 tonnes
	forklift	capacity 3.0 tonnes

This publication (Part 2) provides guidance on the determination of the thickness of a concrete floor supported on the ground, for any combination of applied loads. It also looks at the design of post-tensioned, expansive cement, fibre reinforced, and cold store floors. The document provides reference to Part 1, but does not repeat information already contained in Part 1. Designers should refer to Part 1 for issues such as:

- Construction
- Specification
- Effects of Chemicals on Concrete Pavements
- Determination of the Amount of Shrinkage Reinforcement
- Effects of Various Factors on Abrasion Resistance
- Joints and Joint Layout
- Durability Issues Associated with Concrete Properties
- Surface Finishes

In this Part 2 publication, only limited information is provided on concrete both in its plastic and hardened state. This information is contained within *New Zealand Guide to Concrete Construction*.

Acknowledgements

This handbook was published by the Cement & Concrete Association of New Zealand (CCANZ).

The Cement and Concrete Association of New Zealand wishes to acknowledge the varying assistance of the following in the preparation, providing comments, and review of this document.

In alphabetical order: Andreas Baptist, Golden Bay Cement; Darrin Bell, Compusoft Engineering; Ian Brown, Milburn New Zealand; Derek Chisholm, BRANZ; Andrew Dallas, Allied Concrete Ltd; John Evans, Demden Ltd; Sue Freitag, Opus International Consultants Ltd; Jason Ingham, University of Auckland; James Mackechnie, University of Canterbury; Len McSaveney, Golden Bay Cement; Jeff Marchant, Construction Techniques Ltd; Lindsay Mayo, Lesa Systems Ltd; Jubran Naddaf, Stresscrete; Jeff Smith, Ready Mix Concrete Ltd.

1

materials

Chapter 1

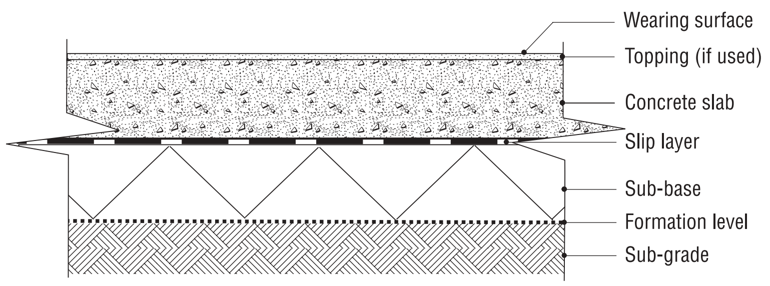
Materials

1.1 INTRODUCTION

The main materials the designer and contractor will need to consider and work with in the successful construction of an industrial floor slab are illustrated in Figure 1.1 and are:

- The sub-grade
- Sub-base
- Slip membrane/vapour barrier
- Concrete
- Reinforcement (refer Chapter 5)

Figure 1.1 Elements of a floor



In this chapter we explore some of these materials. This design guide has been developed as a companion document to two other publications produced by the Cement & Concrete Association of New Zealand. The others being:

- *New Zealand Guide to Concrete Construction*[1]
- *Concrete Ground Floors & Pavements for Commercial & Industrial Use, Part 1*[2]

Information already contained in these two documents is not repeated in this publication, but, when appropriate, supplementary information is provided that is specific to the construction of industrial floors.

1.2 SUB-GRADE

1.2.1 Assessment of sub-grade and soil conditions

A soil investigation of the site should be conducted to determine the properties of the soil below the building. The properties of the soil at depth tend to dictate the settlement characteristics, while the near surface characteristics dictate the likely stresses in the concrete pavement. The soil investigation should identify:

- The presence of layers that might be prone to consolidation, excessive compression or decay.
- The depth of the water table.
- The presence of fill or waste material.

- The elastic properties of the soils.
- The presence of expansive materials.
- The modulus of sub-grade reaction of the sub-grade.

It is important to identify the uniformity of soil properties across the site. The rigidity of a concrete pavement means that the applied loads are spread over a wide area and therefore strong support of the sub-grade is not required. However, it is important that the support provided by the sub-grade is reasonably uniform.

Where sub-grade conditions are not reasonably uniform, this should be corrected by a sub-grade preparation practice such as selective grading, mixing of soil at abrupt transitions, cement or lime stabilisation, and moisture/density control of sub-grade compaction.

1.2.2 Assessment of modulus of sub-grade reaction

Appendix F of Part 1[2] provides some information on the assessment of the sub-grade properties.

A substantial amount of pavement research shows that the stresses in a concrete slab supported on the ground are predicted reasonably well when the modulus of sub-grade reaction (k) is used to model the underlying soil. Although k does not reflect the effect of compressible soil layers at some depth in the sub-grade, its use is appropriate for wheel loads and other concentrated loads because soil pressures under a slab of adequate thickness are not excessive. However, if it is anticipated that heavy distributed loads may be applied to the floor, the amount of settlement should be computed to determine if excessive settlement is expected.

The modulus of sub-grade reaction is measured by a plate loading test on top of the compacted sub-grade or, if a sub-base is used, on top of the sub-base. A detailed description of the load test is given in ASTM D1196, *Non-Repetitive Static Plate load Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements*. However, this test is not specifically orientated to the determination of the modulus of subgrade reaction using a 760mm diameter plate for the test. ACI 360, *Design of slabs on grade*[3], provides details of this test a brief summary of which follows.

Remove loose material from the surface of the grade or subgrade. Place a thin layer of sand or plaster of Paris over this area to assure uniform bearing under the load plates. Then place three 25mm thick steel plates 760, 610, and 460mm in diameter, stacked concentrically pyramid fashion on this surface. Rotate the plates on the bearing surface to assure complete contact with the subgrade.

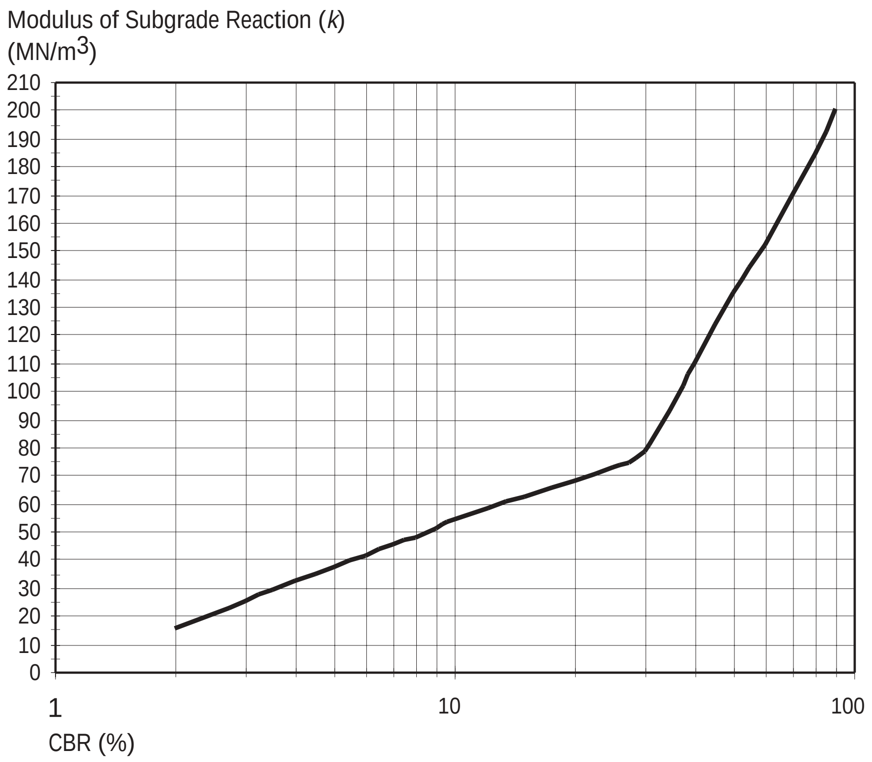
Position three dial gauges on the 760mm diameter plate, 120 degrees apart, to record the plate deflection. Apply a proof load to produce a deflection of approximately 0.25mm. Maintain this load until the settlement is stabilised; then release the load and reset the dial gauges to zero.

After this preparation, the test is performed by applying a series of loads and recording the settlement of the plates. Generally, three load increments are sufficient. The load should be maintained until the rate of settlement is less than 0.025mm per minute. The data is then plotted and the modulus of subgrade reaction calculated at an applied pressure of approximately 70kPa, or a deflection not greater than 1.27mm[24]. k is computed by dividing the unit load by the deflection obtained. The units of k are given in $MN/m^2/m$, kPa/mm or MN/m^3.

A modified plate-bearing test is often used that utilises a 305mm diameter plate. The modified modulus of sub-grade reaction obtained by this test needs to be divided by 2 to determine the equivalent modulus for a 760mm plate test.

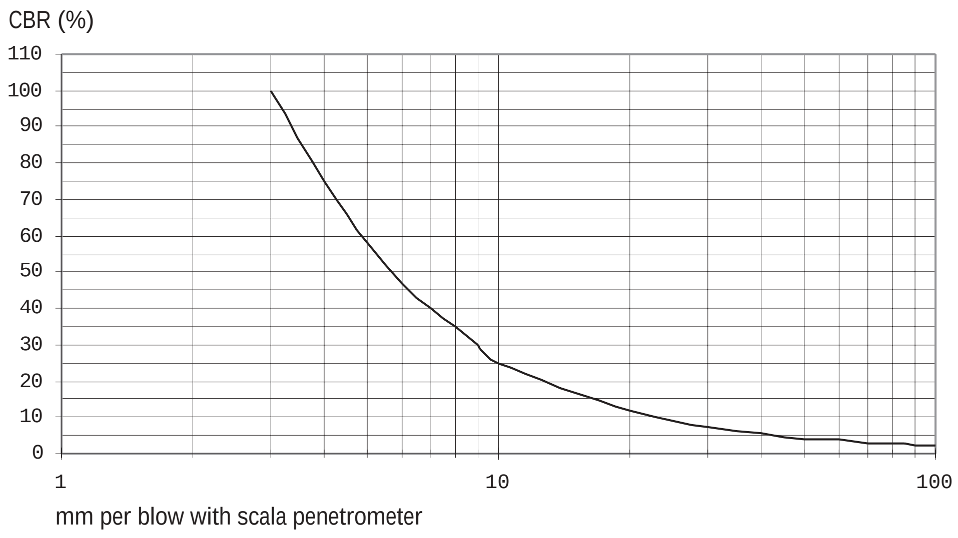
The modulus of sub-grade reaction may also be estimated from correlations with the CBR value, refer Figure 1.2[3], although there is no reliable correlation between CBR and k as they are measurements of different soil characteristics.

Figure 1.2 CBR vs Modulus of Subgrade Reaction (k)



Approximate values for CBR in silts and sands can be obtained using the scala penetrometer test. Correlations suggested by Stockwell[4] are shown in Figure 1.3.

Figure 1.3 Scala penetrometer estimation of CBR



When it is not feasible to perform soil tests, estimates for values for *k*, based on soil descriptions are summarised in Table 1.1[5].

Table 1.1 Typical CBR and *k* values for various soil descriptions

Description	Unit dry weight (kg/m^3)	Field CBR, %	Modulus of sub-grade reaction k (MN/m^3)
Coarse-grained gravelly soils			
Well-graded gravels or gravel-sand mixtures, little or no fines. Excellent foundation, none to very slight frost action potential, excellent drainage characteristics.	2000 - 2240	60 - 80	82 or more
Poorly graded gravels or gravel-sand mixtures, little or no fines. Good to excellent foundations, none to very slight frost action, excellent drainage.	1760 - 2080	25 - 60	82 or more
Silty gravels, gravel-sand-silt mixtures. Good to excellent foundation, slight to medium frost action, fair to poor drainage.	2160 - 2320	40 - 80	82 or more
Silty gravels, gravel-sand-silt mixtures. Good foundation, slight to medium frost action, poor to practically impervious drainage.	1920 - 2240	20 - 40	54 - 82
Clayey gravels, gravel-sand-clay mixtures. Good foundation, slight to medium frost action, poor to practically impervious drainage.	1920 - 2240	20 - 40	54 - 82
Coarse-grained sand and sandy soils			
Well-graded sands or gravelly sands, little or no fines. Good foundation, none to very slight frost action, excellent drainage.	1760 - 2080	20 - 40	54 - 82
Poorly graded sands or gravelly sands, little or no fines. Fair to good foundation, none to very slight frost action, excellent drainage.	1600 - 1920	10 - 25	54 - 82
Silty sands, sand-silt mixtures. Good foundation, slight to high frost action, fair to poor drainage.	1920 - 2160	20 - 40	54 - 82
Clayey sands, sand-clay mixtures. Fair to poor foundation, slight to high frost action, poor to practically impervious drainage.	1680 - 2080	10 - 20	54 - 82
Clayey sands, sand-clay mixtures. Fair to good foundation, slight to high frost action, poor to practically impervious drainage.	1680 - 2080	10 - 20	54 - 82
Fine-grained soils - silts and clay			
Inorganic silts and very fine sands, rock flour, silty or clayey fine sand or clayey silts with slight plasticity. Fair to good foundation, medium to high frost action, fair to poor drainage.	1600 - 2000	5 - 15	27 - 54
Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays. Fair to poor foundation, medium to high frost action, practically impervious.	1600 - 2000	5 - 15	27 - 54
Organic clays to medium to high plasticity, organic silts. Poor foundation, medium to high frost action, poor drainage.	1440 - 1680	4 - 8	27 - 54
Fine-grained soils - silty and clay			
Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. Poor foundation, medium to high frost action, fair to poor drainage.	1280 - 1600	4 - 8	27 - 54
Inorganic clays of high plasticity, organic silts. Poor to very poor foundation, medium frost action, practically impervious.	1440 - 1760	3 - 5	14 - 27
Organic clays of medium to high plasticity, organic silts. Poor to very poor foundation, medium frost action, practically impervious.	1280 - 1680	3 - 5	14 - 27
Peat and other highly organic soils. Not suitable for foundation, slight frost action, fair to poor drainage, very high compressibility.			

1.2.3 Stabilised sub-grade

In some instances considerable benefit can be obtained from stabilising the sub-grade, or using cement bound sub-base. Appendix F of Part 1[2] of this guide provides a summary of these benefits.

1.3 SUB-BASE

A sub-base enhances the local performance of the slab-subgrade system. The modulus of sub-grade reaction (k) used in slab design should reflect this sub-base enhancement where appropriate. Under concentrated point loading, such as from rack legs, the presence of a sub-base modifies the local elastic behaviour of the ground and therefore has an influence on the stresses induced in the slab. For bulk storage with distributed loading conditions, the sub-base offsets the effect of local variations in ground conditions, but the behaviour of the slab is governed more by the overall response of the ground, particularly long term settlement, which is not influenced by the sub-base. For concentrated point loads only, the design values of the modulus of sub-grade reaction can be enhanced as shown in Figure 3.1 when a sub-base is used.

Even where the sub-grade is excellent, a sub-base will generally be required to provide a work surface for construction traffic. A minimum depth of 150mm is generally considered appropriate for this purpose.

The sub-base may also act as a capillary break, preventing moisture rising above the ground water table by capillary action and reaching the underside of the concrete slab. Capillary rise is greater in fine-grained subgrades.

The sub-base should be stable graded granular material of maximum size not exceeding 75mm. It should be fully compacted and blinded with sand or fine crushed material, so that the top surface can be laid and screeded to the specified tolerance. A reasonably smooth surface is necessary to prevent damage to the slip membrane and to minimise friction restraint.

1.4 SLIP LAYER

A slip layer is used to reduce friction between a slab and sub-base. If it is a membrane, it may also serve as a barrier for ground moisture penetrating the building, and prevent loss of moisture and fines from the fresh concrete into the sub-base.

250 micron and 300 micron plastic sheets are most commonly used for slip membranes.

Adjoining sheets of membrane should he lapped by at least 150mm and secured with adhesive tape to prevent displacement after laying; care needs to be taken to avoid wrinkles.

In post-tensioned work two layers of plastic sheet are often used to reduce the coefficient of friction between a slab and its sub-base to a minimum. In conventional slab design contraction joint spacing may be increased, or the reinforcement reduced, by the use of two layers of plastic sheets and by careful preparation of the blinding. 125 micron sheeting may be used for greater economy in double layer applications, but greater care is needed to prevent handling damage and to avoid wrinkles.

An impermeable slip membrane below a slab means that drying can only take place from its top surface, which can increase the risk of curling. Perforated membranes have sometimes been used to reduce this risk, but potential grout loss and increased frictional resistance need to be considered.

1.5 CONCRETE

The *NZ Guide to Concrete Construction*[1] provides an excellent summary of the properties of concrete. It covers the impact of cement, aggregates, water, admixtures, and environmental conditions on the behaviour of concrete.

Part 1[2] of this guide covers issues associated with concrete properties which are specific to industrial floors, such as drying shrinkage, abrasion resistance, corrosion resistance, freeze thaw resistance, and resistance to chemical attack.

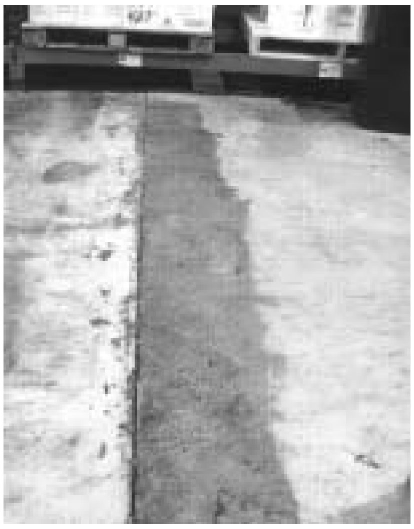
The modulus of rupture is an important parameter in the design of a slab on ground. However, beam test to determine this variable can be expensive, so it is usual to use correlations with compressive strength (f'_c). Usually the f'_c required for durability will be higher than that required for structural purposes. Table 1.2 provides some guidance for selection of the minimum f'_c based on abrasion resistance considerations.

Table 1.2 Minimum concrete strength for abrasion resistance

Member and type of traffic	Minimum characteristic strength, f'_c (MPa)
Floors in commercial areas subject only to pedestrian and/or light trolley traffic	25
Floors subject only to light pneumatic-tyred traffic (vehicles < 3t gross)	25
Floors in warehouses and factories subject to medium or heavy:	
• pneumatic-tyred traffic (> 3t gross)	30
• non-pneumatic-tyred traffic	40
• steel-wheeled traffic	≥40 (to be assessed)

Abrasion in concrete floors can take the form of general loss of the surface and therefore exposure of the aggregate, or breakdown of the surface at cracks and joints. Many industrial warehouses

Joints damaged by solid-tyred reach trucks



Loss of surface by abrasion

utilise solid tyred reach trucks. The small, hard wheels on these vehicles are particularly damaging on unarmoured joints and uncontrolled cracks.

The selection of an appropriate concrete mix design is an important aspect of the construction of a floor slab. However, probably the most common reasons for unsatisfactory end product are shortcomings in the placing, finishing, and early care of slabs. The specification for the project should either provide, or require, a methodology statement and a team meeting prior to the work starting. The methodology statement should define who does what, when, how, where, and define accountability and client expectations. The performance of the mix in terms of plastic cracking, top down setting, and bleed should be discussed at a pre-pour meeting that includes the designer, concrete supplier, contractor and placer.

Early age care should include protection from restrained early age thermal contraction, saw cutting, and curing.

2

loadings

Chapter 2
Loadings

2.1 INTRODUCTION

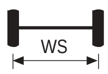
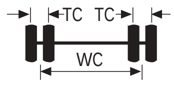
This chapter provides some guidance on typical loads and load cases used in the design of an industrial floor slab supported on the ground. The designer should consult with the building owner/equipment suppliers to determine more accurately the expected loads from equipment.

In determining the design load combinations, some consideration of potential future use of the building should be made. The trend, particularly in warehouses, has been to make greater utilisation of the available floor space, resulting in heavier floor loads.

In this section we consider:

- Dead Loads
- Live loads, including:
    - Wheel loads
    - Post loads, typically from storage racks
    - Distributed loads.
- Seismic loads

Table 2.1 Characteristics of Typical Forklift Trucks

Front Axle Load (tonnes)	Approx. Rated Capacity (tonnes)	Average Wheel Spacing (mm)		
		Single Wheels	Dual Wheels	
				
4	2	750	**WC**	**TC**
6	2.5	800		
8	3.5	850		
10	4.5	900	1450	250
20	9	1050	1750	300
30	13	1150	2000	350
40	17		2200	400
50	22		2350	450
60	26		2450	500
70	30		2550	550
80	35		2600	550
90	40		2600	550

Notes:
(Averages taken from 1983 survey of manufacturer's data)
For pneumatic-tyred vehicles only;
Tyre inflation pressure range = 650 to 750 KPa;
Load contact area assumed per tyre = wheel load divided by inflation pressure;
Maximum front axle load for most forklifts is equal to 2.3 times the rated capacity.
Table 2.2 provides some typical data for reach trucks.

Figure 2.1
Pallet transporter

Figure 2.2
Counterbalance forklift

Figure 2.3
Reach truck

2.2 DEAD LOADS

The self-weight of the slab is typically the main dead load. As the slab's weight is supported directly by the ground, the induced bending moments in the slab are negligible, and can be ignored. This load case becomes more significant when considering uplift under seismic loading, and tensile forces induced from restrained shrinkage and thermal contraction.

2.3 LIVE LOADS

2.3.1 Wheel loads

Wheel loads typically applied to an industrial floor slab derive from trucks or mobile materials handling equipment. Commercial vehicles that use the highway are subject to statutory load limits, and axle configuration. A guide to axle loads for these vehicles is found in the AUSTROADS Pavement Design Guide[6].

2.3.1.1 Mobile materials handling equipment

This section provides a brief introduction of typical materials handling equipment.

Pallet transporters

Refer Figure 2.1. These are predominantly pedestrian or rider pallet trucks. The load capacity is usually limited, with a maximum of approximately 2 tonnes. The load concentration can be high due to the small size of the wheels.

Counterbalance forklift

Refer Figure 2.2. These forklifts are fitted with telescopic masts, with the load always carried ahead of the front wheels. The load capacity can be as high as 22 tonnes, but in an industrial building the forklift capacity is normally below 4.5 tonnes. In warehouses, a typical pallet weight would be 1 tonne, so usually forklifts with a 2.5-3 tonne capacity are used.

The rear wheels are used for steering. These forklifts typically require minimum aisle widths of 4m to allow the forklifts to manoeuvre in front of the loads. Lift heights are limited by stability and do not normally exceed 8.5m.

Reach trucks

Refer Figure 2.3. These trucks have the facility of a moving telescopic mast or pantographic load extender. The trucks front load, but are able to transport the load in a retracted position within the wheelbase. Their manoeuvrability, and ability to "reach out" to pick up loads, allows narrow working aisles averaging 2.3-2.7m.

Reach trucks typically have a load capacity of 2 tonnes, and lift heights that do not normally exceed 8.5m.

Table 2.2 Load data for a typical reach truck

	1.6 t rated capacity	2.0 t rated capacity
Approx weight of reach truck (tonnes)	3.0	4.0
Weight on each front load wheels when carrying load (tonnes)*	1.5	2.0
Weight on each front load wheels when unloaded (tonnes)*	0.6	0.8
Weight on rear drive wheel when carrying load (tonnes)	0.86	1.0
Weight on rear drive wheel when unloaded (tonnes)	0.9	1.3
Weight on rear caster wheels when carrying load (tonnes)*	0.4	0.4
Weight on rear caster wheels when unloaded (tonnes)*	0.5	0.6

** The load refers to the load on each wheel. There are two wheels so the "axle" load is twice the tabled load.*

Figure 2.4
VNA truck

Figure 2.5
Order pickers

Very narrow aisle (VNA) trucks

Refer Figure 2.4. VNA trucks can be used in very narrow aisles that are only marginally wider than the truck (approximately 1.9m). Often a truck guidance system is used, which could be steel rails or a low voltage wire chased into the floor.

These specialist trucks all have the capability of stacking or de-stacking a load at right angles to the direction of travel by utilising a traversing rotating load carriage or a rotating traversing mast. As the VNA trucks are designed to have high longitudinal and lateral stability, basic loads of 2 tonnes can be lifted on telescoping masts to 12m.

Order pickers

Refer Figure 2.5. Order pickers have a load and aisle width specification similar to those of elevating cab VNA lift trucks, except that lift heights are usually limited to 9.5m. Order pickers carry a non-lateral moving empty pallet onto which the operator places items taken from bulk stacks.

Stacker cranes

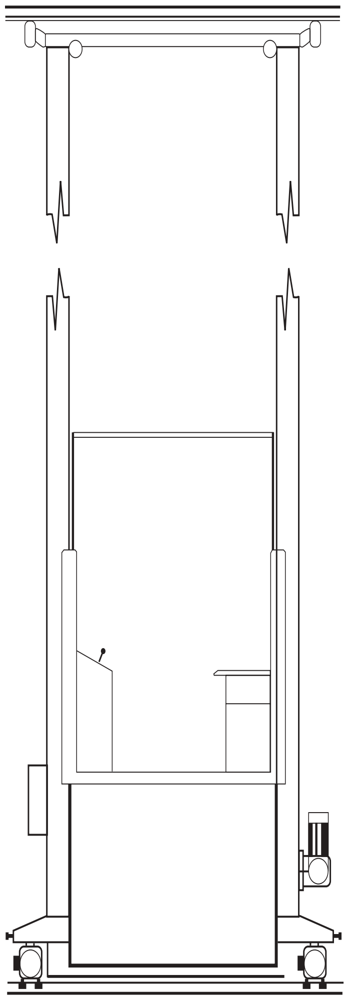
Refer to Figure 2.6. Stacker cranes are restrained between floor and top mounted rails but are capable of transferring by means of special rail links, or bridges, to selected aisles. With lateral rail restraint, stability is not so limited as with other VNA equipment and 2 tonne loads can be lifted to 30m in aisle widths as narrow as 1.2m.

Contact pressures – wheel loads

For pneumatic-tyred forklifts, the contact pressure is equal to the tyre inflation pressure. Typically this is in the range of 650-750 kPa. Table 2.1 summarises characteristic front axle loads taken from a 1983 manufacturers data[7]. Spot checks indicate that this information is still relevant today.

The tyres of reach trucks are typically small and solid. The tyre contact pressure is dependent upon the composition of the material used, and the manufacture of the truck should be consulted. Reference 5 provides some information useful for

Figure 2.6
Stacker crane



preliminary design purposes. Contact pressures of 5.6 MPa are reported for a two tonne reach truck and 9 MPa for polyurethane tyres on a pallet transporters.

2.3.2 Storage System Loads

Numerous storage systems have been developed to cater for various warehouse uses. Figure 2.7 *(see page 20)* illustrates a variety of storage systems promoted by one New Zealand manufacturer.

Pallet racking

Pallet racking systems are designed to accommodate pallets that typically have a maximum weight of 1 tonne. The height of the racking system is usually dictated by either the building height, lifting restrictions of forklifts, or seismic design considerations.

The racking systems are placed in aisles as single racks, or more commonly as back to back racks. Typically the spacing between the legs in the longitudinal direction is approximately 2.7m while in the transverse direction they are typically between 800 and 1200mm. The base plates are small and bolted to the floor.

Where specialist lifting equipment is available, the racks may be utilised in double depth, or with very narrow aisles, refer Figure 2.7.

Other more specialist racking options are illustrated in concept in Figure 2.7.

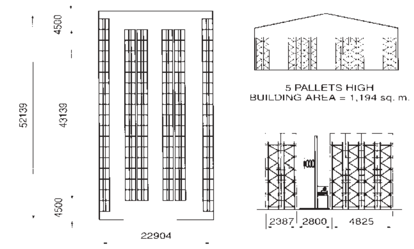
Block stacking

In some facilities material is simply stacked in multiple layers onto the floor.

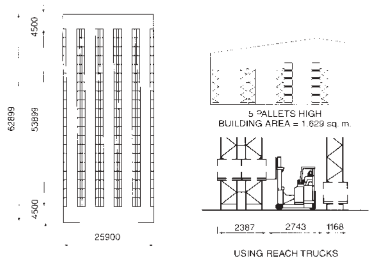
2.4 SEISMIC LOADING

Racking systems are often simply bolted to the floor. During an earthquake uplift forces can develop, which the floor slab must be designed to accommodate. The magnitude of these forces can be calculated using the NZ Loadings Standard NZS 4203[8]. To minimise the uplift forces the slab is required to resist, the racking system may be designed to have yielding base plates. The manufacture of the racking system should be consulted to determine the seismic design philosophy utilised in the proposed racking system.

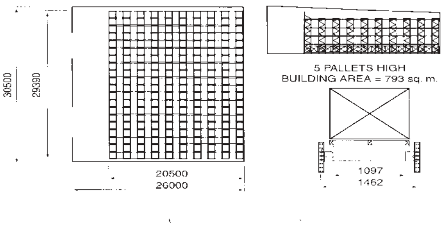
Figure 2.7
Various storage systems



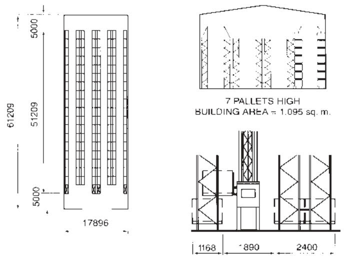
% Average locations used 90% Stock rotation Average
% Immediate accessibility 50% % Floor utilisation 50%



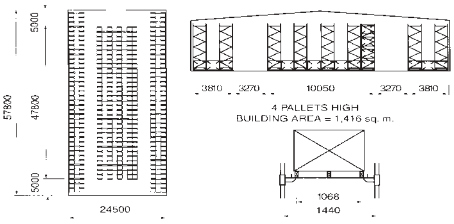
% Average locations used 90% Stock rotation Good
% Immediate accessibility 100% % Floor utilisation 40%



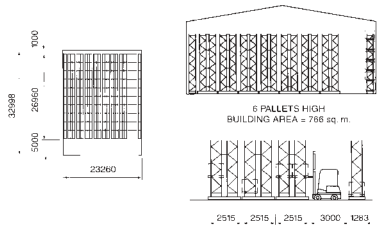
% Average locations used 70% Stock rotation Poor
% Immediate accessibility 5% % Floor utilisation 75%



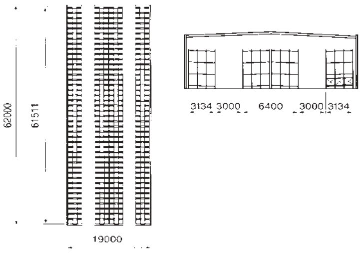
% Average locations used 90% Stock rotation Good
% Immediate accessibility 100% % Floor utilisation 45%



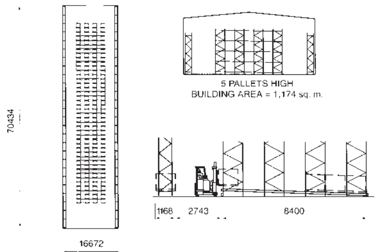
% Average locations used 70% Stock rotation Poor
% Immediate accessibility 25% % Floor utilisation 65%



% Average locations used 90% Stock rotation Good
% Immediate accessibility 100% % Floor utilisation 70%



% Average locations used 90% Stock rotation Poor
% Immediate accessibility 33% % Floor utilisation 65%



% Average locations used 75% Stock rotation Good
% Immediate accessibility 25% % Floor utilisation 55%

3

concrete ground floor thickness determination

Chapter 3

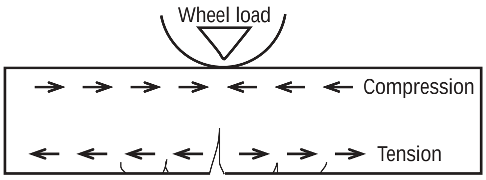
Concrete Ground Floor Thickness Determination

3.1 INTRODUCTION

The design of a concrete slab supported on the ground requires decisions on many issues including:

- Joint spacing
- Joint detailing
- Surface finish
- Durability
- Reinforcement
- Pavement thickness

This chapter focuses on the determination of floor thickness. The other issues are covered in subsequent chapters. The normal sequence for the design procedure for concrete industrial pavements is illustrated in Table 1.1 of Part 1[2]. Prior to the calculation of the floor thickness, the designer would have considered the loading, subgrade conditions, required concrete strength, and surface finish.



The main variables in determining the thickness of a slab on ground are-

- Concrete strength, in particular the tensile strength. As the tensile strength increases, the required thickness reduces.
- Soil properties – generally the softer the soil, the thicker the required pavement.
- Applied loading – as applied load and contact pressure increase, the required pavement thickness increases.
- The combination of different loads.
- Number of load repetitions. Above a threshold stress, as the number of repetitions increases, the pavement thickness needs to increase to prevent fatigue failure.
- The position of the applied load relative to the edge of the slab.
- The presence or absence of load transfer between adjacent slabs.
- The frictional resistance on the base of the slab.
- The reinforcement method, i.e. bars, steel fibres, or post-tensioning.

It is important to note that the presence of reinforcing in the form of mesh or conventional reinforcing bars does not influence the required thickness. Reinforcement is typically provided to control drying shrinkage cracking, refer Chapter 5.

3.2 METHODS OF ANALYSIS –GENERAL COMMENTS

In this section the analysis of conventionally reinforced slabs on ground is discussed. The modifications of this analysis technique when considering shrinkage compensating concretes, post-tensioned, and fibre reinforced floors are described in sections 3.10, 3.11, and 3.12, respectively.

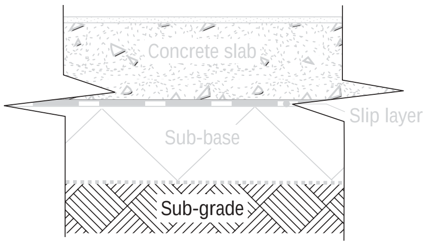
Unlike suspended concrete floors, a conventionally reinforced slab on ground is designed on the basis of limiting the induced flexural stresses in the concrete to a factored level below the ultimate flexural tensile strength. No use is made of bar or mesh reinforcement to resist flexural loads. The reinforcement is provided to control cracking and shrinkage stresses only.

Most analytical methods for slab on ground design are based on the assumption that the soil is an elastic medium. Most methods assume that the soil can be modelled as a series of springs, with the force in the spring being proportional to the vertical deformation. For these models the soil is defined in terms of the modulus of subgrade reaction (refer to section 1.2.2).

A more accurate representation of the soil can be made by assuming that the soil is an elastic half space. This is the approach taken in the Cement & Concrete Association of Australia's publication *Industrial Floor Slabs and Pavements*[9]. Although the assumption of an elastic continuum more accurately models reality, comparative analysis[9] showed that there is relatively little difference between the spring and elastic continuum models. Computations using the two soil models produced maximum bending moments within the slab that are within 1% of each other.

In this design guide, the analysis method presented is based on the soil being represented as a series of springs.

3.3 SUB-GRADE



The sub-grade is the material that ultimately supports the load. A soil investigation should determine the strength and settlement properties of the sub-grade from which the adequacy of a slab on ground can be ascertained.

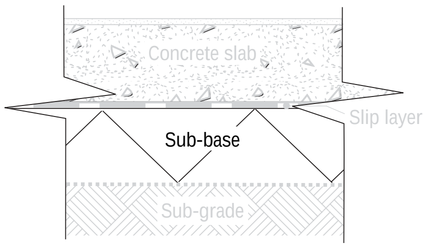
The design method based on sub-grade reaction caters essentially for the safety against rapidly applied stresses near the surface. It does not take account of settlements due to soil consolidation under average live plus dead loading. To estimate the magnitude of such settlements, reference should be made to soils experts with respect to appropriate site investigation, soil testing and interpretation.

Long term settlement can be as much as 20 to 40 times greater than the elastic deflections which are the basis of the slab design. The measure of the elastic sub-grade compressibility is termed the modulus of sub-grade reaction (k) and usually involves elastic compressions of not greater than 1 to 2 mm. k is a measure of the pressure required to deform the sub-grade a unit distance.

Some estimation of the value of k is required to input into the design. Comparatively large variations in k are found to have only a minor effect on the performance of the ground slab, and therefore a precise estimate of its value is not necessary. Indeed, because the formation is frequently just below the existing ground level, and therefore often in variable materials, a uniform reliable value of k may not be obtainable from site tests. Table 1.1 gives an estimate of the value of k which enables assessments to be made against common descriptions and simple investigation measures.

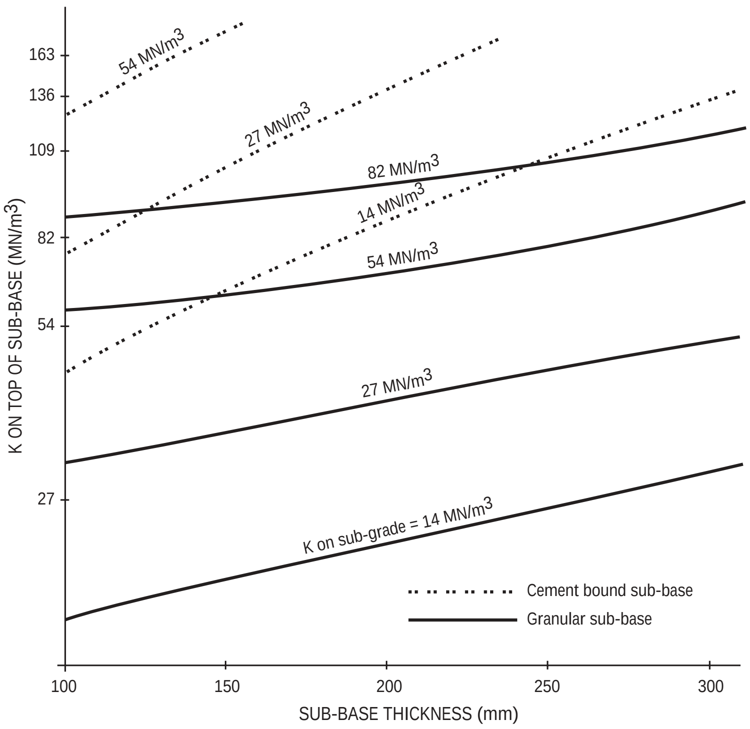
It is emphasised that the value of k does not reflect long term or differential settlements which have no reliable correlation with the elastic properties of the sub-grade.

3.4 SUB-BASE

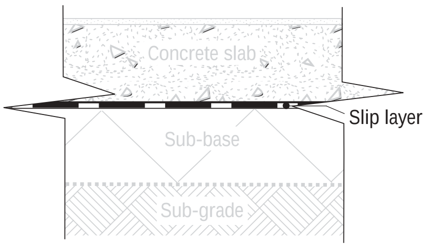


While the presence of a sub-base layer is needed to provide a level and firm working platform on which to construct the slab, advantage can be taken of its structural properties which increase the apparent value of k for use in the design of slabs for concentrated point loads. In the absence of test results, Figure 3.1 gives an approximate assessment of the effect of sub-base on the apparent value of k, refer also to section 1.3.

Figure 3.1 The effect of granular sub-base thickness on modulus of sub-grade reaction, k (after Packard24)



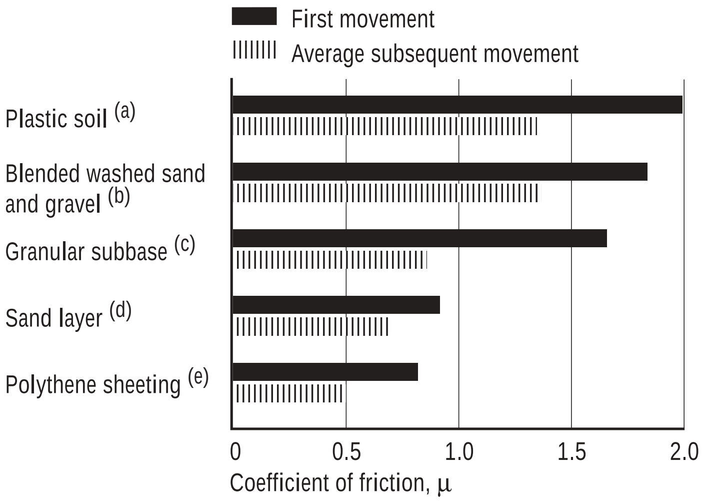
3.5 SLIP LAYER



A slip layer allows the concrete slab to respond to shrinkage, as well as temperature variations, such that excessive tensile stresses are not induced in the concrete. Various surface preparations are used which give diverse frictional restraint, and some of these are illustrated in Figure 3.2. More information on the five selected sub-bases is provided in appendix D of Part 1[2].

Figure 3.2 Typical values for coefficient of friction

Values of the coefficient friction for a 215mm-thick slab on different bases and sub-bases. More information on the five selected sub-bases (a to e) is in Appendix D of Part 1.



Recent testing[25] revealed that the coefficient of friction increases with increased contact pressure. Base sliding tests were conducted on a rig comprising of a 135mm slab on polymer damp proof membrane (DPC) over 25 mm of sand on hardfill. The sand was carefully placed to ensure it was smooth, and one and two layers of DPC were tested. The increase in coefficient of friction with increasing contact pressure was believed to be caused by the indentation of sand grains into the soft material of the membrane. Table 3.1 summarises the results of this testing. These should be considered as lower bound values which can be achieved in laboratory conditions.

Table 3.1 Base sliding friction for slabs on the ground

FoundationType	Contact Pressure kPa	Peak coefficient of friction	Mean coefficient of friction
Single layer of DPC	3.0	0.42	0.38
	6.6	0.53	0.49
Two layers of DPC	3.1	0.21	0.18
	6.2	0.45	0.40

3.6 LOAD FACTORS

To ensure consistency with the New Zealand Concrete Design Standard (NZS 3101)[13], ultimate limit state design requires the use of load factors and strength reduction factors. In this guideline it is recommended that the load factor of 1.5, recommended in the C&CA publication TR550[10], be used. Consistent with TR550, a strength reduction factor (ϕ) of 1.0 is recommended for flexural strength. The factors of safety achieved by using these assumptions are consistent with international design procedures for industrial floors on ground.

When checking punching shear or bearing strength, this guide recommends the use of the relevant provisions of NZS 3101. For consistency, it is recommended that the strength reduction factors defined in NZS 3101 are used for these design considerations.

In this guideline the issue of fatigue has been separated from that of load factors by introducing a reduction factor to the modulus of rupture, which is dependent upon the expected number of repetitions. In many references, 5 and 12, fatigue is addressed by increasing the load factor, or by introducing a 'dynamic load factor'.

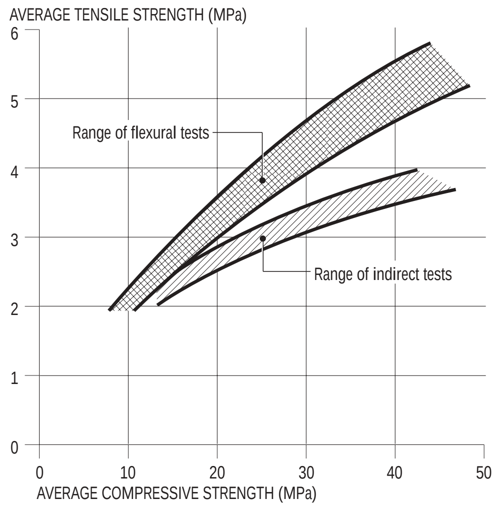
3.7 MODULUS OF RUPTURE OF CONCRETE

The design of pavements is based on the flexural strength of concrete. The method of determining the modulus of rupture for a particular concrete mix is described in Chapter 9. The size of the beam specimen is dependent upon the aggregate size, but is typically 100 x100 x 400mm long. The specimen is bent with third point loading to determine the maximum extreme fibre stress at failure.

For concrete, the flexural test gives a considerably higher value of tensile strength than the splitting test (refer Chapter 9), and there is not a direct relationship between them.

Neither is there a fixed relationship between compressive and tensile strength. This has been widely investigated and a number of authorities have proposed bands within which such a relationship maybe expected to fall. One such band is illustrated in Figure 3.3

Figure 3.3 The relationship between compressive and tensile strengths



Although the design of pavements is based on the modulus of rupture, it is usual practice to determine the relationship between the flexural and compressive strength for the given concrete and to control the quality of the concrete on the project in terms of compressive strength.

The modulus of rupture of concrete can be estimated from Equation 3.1, which provides a relationship between the compressive (standard cylinder) strength and modulus of rupture. This equation is that recommended in reference 5, after conversion from cube to cylinder test specimen compressive strength. Within the typical range of compressive strengths specified for floors (25-45 MPa) the value obtained from Equation 3.1 is within 5% of that using the value of $0.8\sqrt{f'_c}$ specified in NZS 3101. However, the use of Equation 3.1 is recommended as it has been suggested[11] that the 0.5 power exponent used in NZS 3101 is too low and underestimates the modulus of rupture at high compressive strengths.

$$f_r = 0.456 k_1 k_2 (f'_c)^{0.66} \quad \text{MPa} \qquad \text{[Equation 3.1]}$$

Where:

f'_c = the specified 28 day cylinder compressive strength (MPa)

k_1 = time to first application of maximum load,
= 1.0 for 28 days, and
= 1.1 for greater than 90 days

k_2 = load repetition factor, =1.5(0.73-0.0846(log(N)-3) for N= 8,000 to 400,000 refer also to Table 3.2, but should not be less than 0.75 or greater than 1.0

Table 3.2 Load repetition factor k_2

Load repetitions (N)	Load repetition factor k_2
Unlimited	0.75
400,000	0.77
300,000	0.78
200,000	0.81
100,000	0.84
50,000	0.89
30,000	0.90
10,000	0.96
<8,000	1.0

3.8 SELECTION OF CONCRETE PROPERTIES

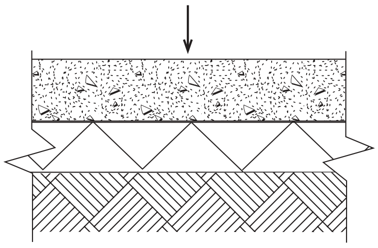
The designer needs to specify the required concrete strength. This is normally specified in terms of the compressive strength, although it is recognised that there may not be a strong correlation between this variable and the attribute the designer is trying to achieve. Compressive strength is specified as it is relatively simple to test.

The major durability consideration for an industrial pavement is normally abrasion resistance and joint protection. However, depending on the environment of the pavement, corrosion of the reinforcement, freeze thaw, chemical resistance and permeability may need to be considered. All these tend to be controlled by specifying an appropriate compressive strength. The strength required for durability will often be higher than that required for structural purposes and would therefore govern design.

Recommendation on the selection of concrete properties for abrasion, corrosion, freeze thaw, and chemical resistance are provided in Part 1[2] of this design guide. Review of appendix E of Part 1[2] will illustrate the importance of finishing technique and curing method on the abrasion resistance of concrete floors. The importance of these variables should be reflected in the specification for the project.

3.9 CONVENTIONALLY REINFORCED SLABS: CALCULATION OF STRESSES DUE TO WHEEL OR RACK LOADING

3.9.1 Point loads – interior of a slab



The stress in the slab beneath a point load located away from the edge of the slab can be approximated using Westergaard's equation (Equation 3.2). Alternatively the thickness required to limit the concrete stress to a specified value under a given load can be determined by trial and iteration.

$$\sigma_i = 2.70(1+\mu)\frac{P}{h^2}\left[4\log\left(\frac{l}{b}\right)+1.069\right]x10^6 \quad \text{kN/m}^2$$ [Equation 3.2]

Where

- μ = Poisson ratio of the slab, typically 0.15
- P = the applied load, tonnes
- h = the thickness of the slab, mm
- l = radius of relative stiffness, refer eqn 3.3, mm
- b = equivalent radius of loaded area, refer Equation 3.4, mm

$$l = \left[\frac{Eh^3x10^3}{12\left(1-\mu^2\right)k}\right]^{0.25} \quad \text{mm}$$ [Equation 3.3]

Where:

- E = Modulus of elasticity of the concrete slab, refer NZS 3101, MPa
- k = modulus of sub-grade reaction, refer section 3.3, MN/m^3

$$b = (1.6r^2 + h^2)^{0.5} - 0.675h \quad \text{for } r < 1.72h$$
$$\text{or } b = r \quad \text{for } r \geq 1.72h$$ [Equation 3.4]

Where:

- r = radius of loaded area, mm, refer Figure 3.4

Figure 3.4 illustrates the definition of the loaded radius. When the centre line distance between two contact areas is less than twice the slab thickness, an effective contact area as shown in Figure 3.4 is assumed[5].

Figure 3.4 Definition of the loaded radius (r) for various load positions and for multiple contact areas

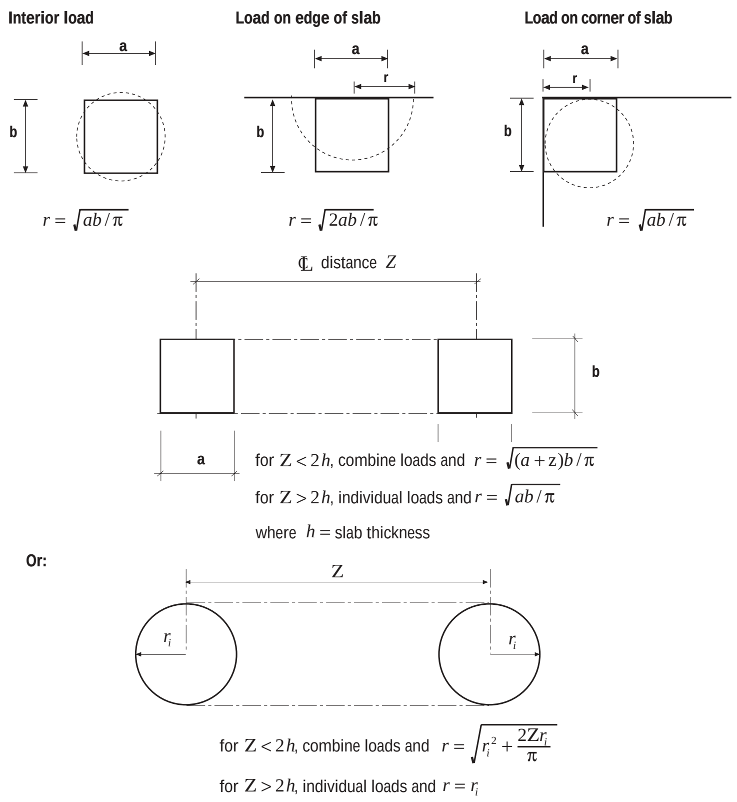
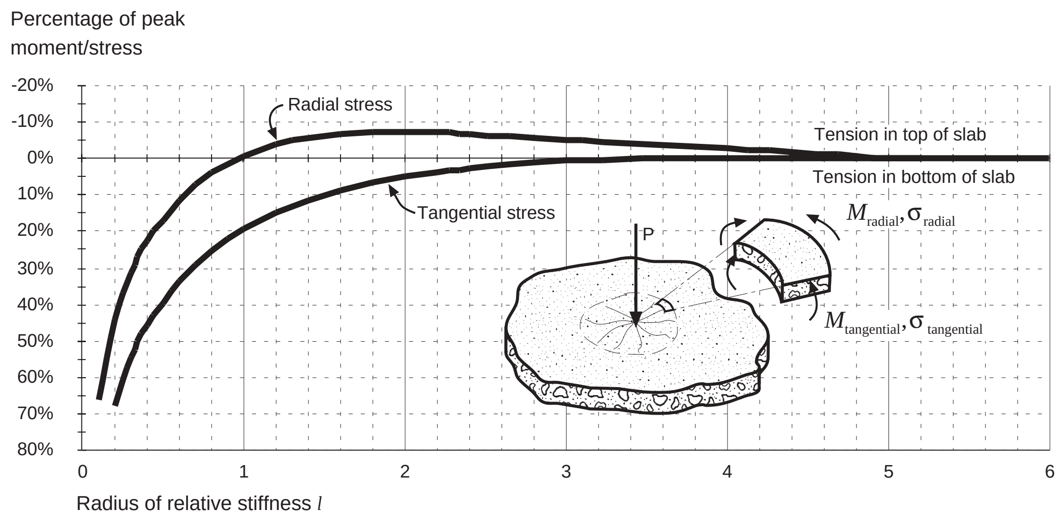


Figure 3.5 Moment/Stress diagram for interior load case



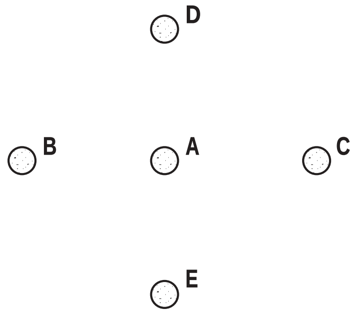
The reduction of stress away from the load application point is illustrated in Figure 3.5. This has been derived from Westergaards original paper[15] and strictly relates to a concentrated point load. The definition for radial and tangential stress is that used by Westergaard and is illustrated in

Figure 3.5. The tangential stress results in cracking that radiates out from the load application point with the cracks initiating on the bottom of the slab (for gravity loads). The radial stress results in circular crack patterns. Figure 3.5 show that the sign of the radial stress reverses 1.0*l* from the load application point.

Figure 3.6 shows how to combine stresses from various load points.

Figure 3.6 Combination of stress from various load points

INTERIOR LOADS



$$\Sigma\sigma_A = \sigma_A + \sigma_{A,Br} + \sigma_{A,Cr} + \sigma_{A,Dt} + \sigma_{A,Et}$$

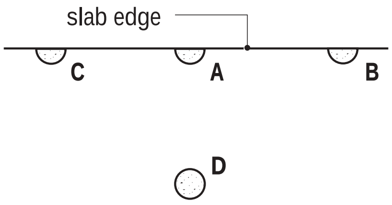
or $$\Sigma\sigma_A = \sigma_A + \sigma_{A,Bt} + \sigma_{A,Ct} + \sigma_{A,Dr} + \sigma_{A,Er}$$

where

$\sigma_{A,Ct}$ = stress at A due to **tangential** stress from a load at C

$\sigma_{A,Cr}$ = stress at A due to **radial** stress from a load at C

EDGE LOADS

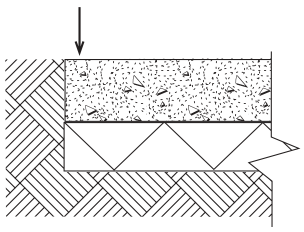


$$\Sigma\sigma_A = \sigma_A + \sigma_{A,Br} + \sigma_{A,Cr} + \sigma_{A,Dt}$$

where the distribution of radial stresses along the edge are given by figure 3.7

The design thickness of a slab subject to loads which are distant from the edge of the slab is appropriate when the superposition of stresses calculated using Equation 3.2 and Figure 3.5, multiplied by a load factor of 1.5, give calculated stresses less than the modulus of rupture (Equation 3.1).

3.9.2 Point loads – edge of a slab



The stressed induced beneath a point load located on the edge of the slab can be approximated using Kelly's equation (Equation 3.5).

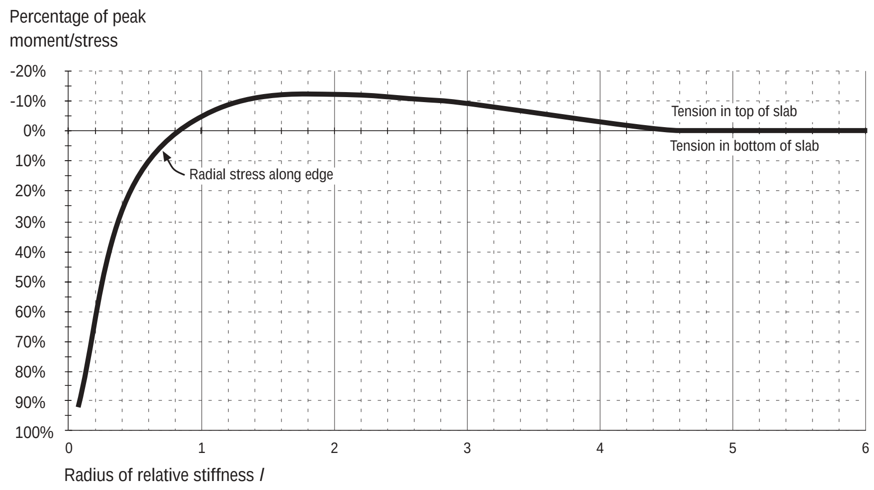
$$\sigma_e = 5.19(1+0.54\mu)\frac{P}{h^2}\left[4\log\left(\frac{l}{b}\right)+\log\left(\frac{b}{25.4}\right)\right]x10^6 \quad \text{kN/m}^2 \qquad \text{[Equation 3.5]}$$

Where:

- μ = Poisson ratio of the slab, typically 0.15
- P = the applied load, tonnes
- h = the thickness of the slab, mm
- l = radius of relative stiffness, refer Equation 3.3, mm
- b = equivalent radius of loaded area, refer Equation 3.4, mm

The reduction of stress away from the load application point is illustrated in Figure 3.7.

Figure 3.7 Moment/Stress diagram for edge load case

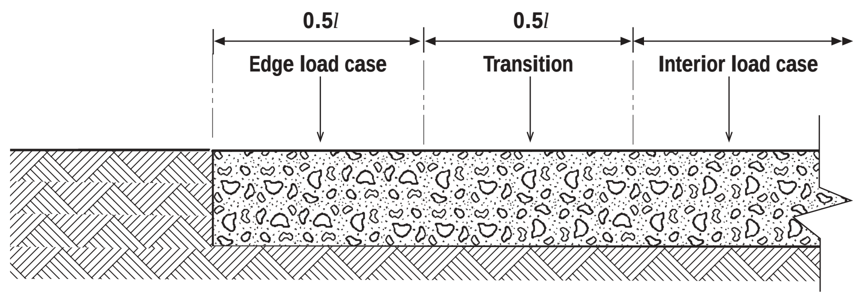


Equation 3.5 assumes that there is no load transfer between adjacent slabs along the shared edge.

Load transfer along the shared edge can occur if dowels are provided or aggregate interlock occurs, ie, a saw cut joint that is expected to open up less than 1mm. When load transfer occurs, the stress calculated by Equation 3.5 should be multiplied by 0.85.

Westergaards original papers provide little guidance to designers on how near to the edge a load point needs to be before it is classed as an edge load. It is suggested that when the centre of the load point is greater than *l* away from the edge, the load be classed as an interior load (Equation 3.2). When the load is closer than 0.5*l* it should be classed as an edge load (Equation 3.5), and for load points between these a linear transition is assumed. Refer Figure 3.8. For a 35 MPa, 150mm thick slab supported on ground with a CBR of 10, and 150mm of hardfill, *l* = 606mm, implying that if the load is closer than 300mm to the edge, Equation 3.5 should be used.

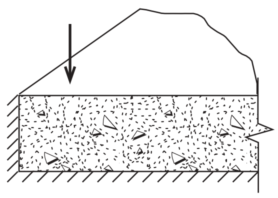
Figure 3.8 Definition of edge and interior load cases



l = radius of relative stiffness [Equation 3.3]

The design thickness of a slab subject to loads which are on the edge of the slab is appropriate when the superposition of stresses calculated using Equations 3.5 (edge loads) and 3.2 (interior loads), and Figures 3.5 and 3.7, multiplied by a load factor of 1.5, give calculated stresses less than the modulus of rupture (Equation 3.1).

3.9.3 Point loads – corner of a slab



For the interior and edge load application positions, the maximum tensile stress occurs at the bottom of the slab below the load application point. When the load is applied at a corner, the maximum tensile stress occurs in the top of the slab a distance $2\sqrt{(\sqrt{2})r\ell}$ from the corner. The slab's behaviour is analogous to that of a cantilevered beam.

The maximum tensile stress in the slab when the corner is loaded can be approximated using Picket's equation (Equation 3.6).

$$\sigma_c = 41.2\frac{P}{h^2}\left[1-\frac{(r/l)^{0.5}}{0.925+0.22(r/l)}\right] \text{x } 10^6 \quad \text{kN/m}^2 \qquad \text{[Equation 3.6]}$$

Equation 3.6 assumes that there is no load transfer along the edge. Childs and Kapernick[17] found that if load transfer was provided by means of dowel bars, for example, the corner stress in curled slabs was reduced by approximately 30% and the edge stress by 15%, making the edge load case more critical in some instances. When edge load transfer occurs at the corner of a slab, the stress calculated by Equation 3.6 should be multiplied by 0.7.

The design thickness of a slab subject to loads which are on the corner of the slab is appropriate when the superposition of stresses calculated using Equations 3.6 (corner loads), 3.5 (edge loads) and 3.2 (interior loads), and Figures 3.5 and 3.7, multiplied by a load factor of 1.5, give calculated stresses less than the modulus of rupture (Equation 3.1).

3.9.4 Punching shear resistance

Punching shear failure is possible when the pavement is subjected to concentrated loads. It is rarely significant to slab design, but can become the critical consideration when heavy loads with small base plates are applied to thin slabs. Posts located near to isolation joints or where two posts are side by side are more critical.

An approach to the assessment of punching shear may be carried out using an analogy of a column supporting a suspended slab. The design equations for this are given in NZS 3101[13]. For consistency, it is recommended that the strength reduction factor (ϕ) as defined in NZS 3101 be used in this calculation. This apparent conservatism is justifiable as punching shear rarely governs the design. Using this model the first approach would be to assume the post load is evenly transferred to the sub-grade. If the calculations indicate that the pavement thickness is insufficient, it is suggested that the designer make a second assessment whereby a proportion of the post load is directly transferred to the sub-grade. This proportion is a function of the pavement thickness and the sub-grade stiffness[14].

3.9.5 Concrete bearing pressure under point loads

The ultimate bearing strength of concrete is defined in NZS 3101[13]. For consistency, it is recommended that the strength reduction factor (ϕ) as defined in NZS 3101 is used in this calculation; however bearing capacity rarely governs the design.

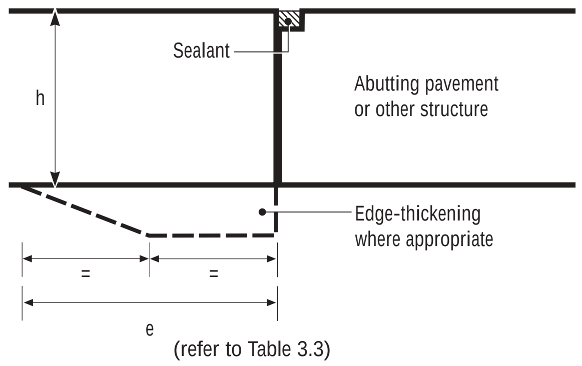
3.9.6 Edge thickening

For wheel loading and post loading, the slab thickness required for edge loading may be greater than that required for interior loading. A guide for the required edge thickening width is given in Table 3.3. However, to minimise restraint and the development of shrinkage cracking, it is preferable if a level bottom surface of the base is provided.

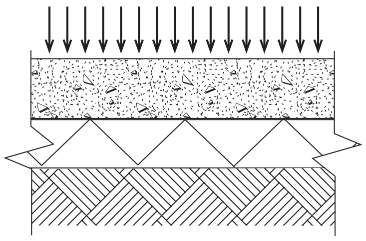
Table 3.3 Distance e* from edge of base at which base thickening should commence

General description of supporting soil	Edge distance e*
Very weak	20h
Weak	15h
Medium	10h
Stiff	8h
Very Stiff	6h
* Refer Figure 3.9	h = thickness required for interior loading

Figure 3.9 Edge thickening at joints where required.



3.9.7 Uniformly distributed loads

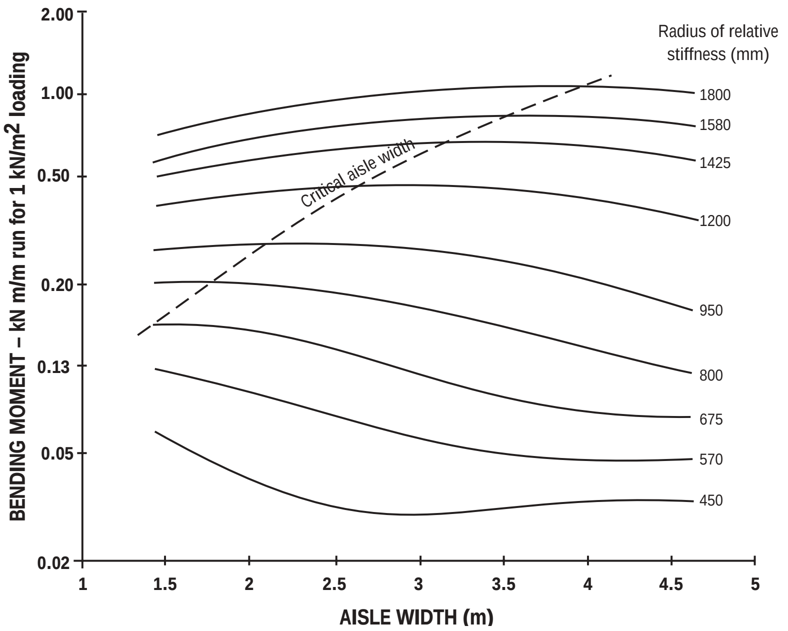


Stress due to uniform loading

The critical bending moment due to uniform loading is determined by a discrete-element slab analysis, as used by Panak and Rauhut[20]. This technique is well documented[21-23] and gives results for this type of problem as accurate as those obtained by the more expensive and time-consuming finite element techniques.

The bending moment in the slab is determined for a given aisle width and radius of relative stiffness (*l*), which is a measure of the ratio of the stiffness of the slab to the stiffness of the sub-grade. Figure 3.10 indicates that, for a given radius of relative stiffness (*l*) there is a maximum bending moment at a certain aisle width.

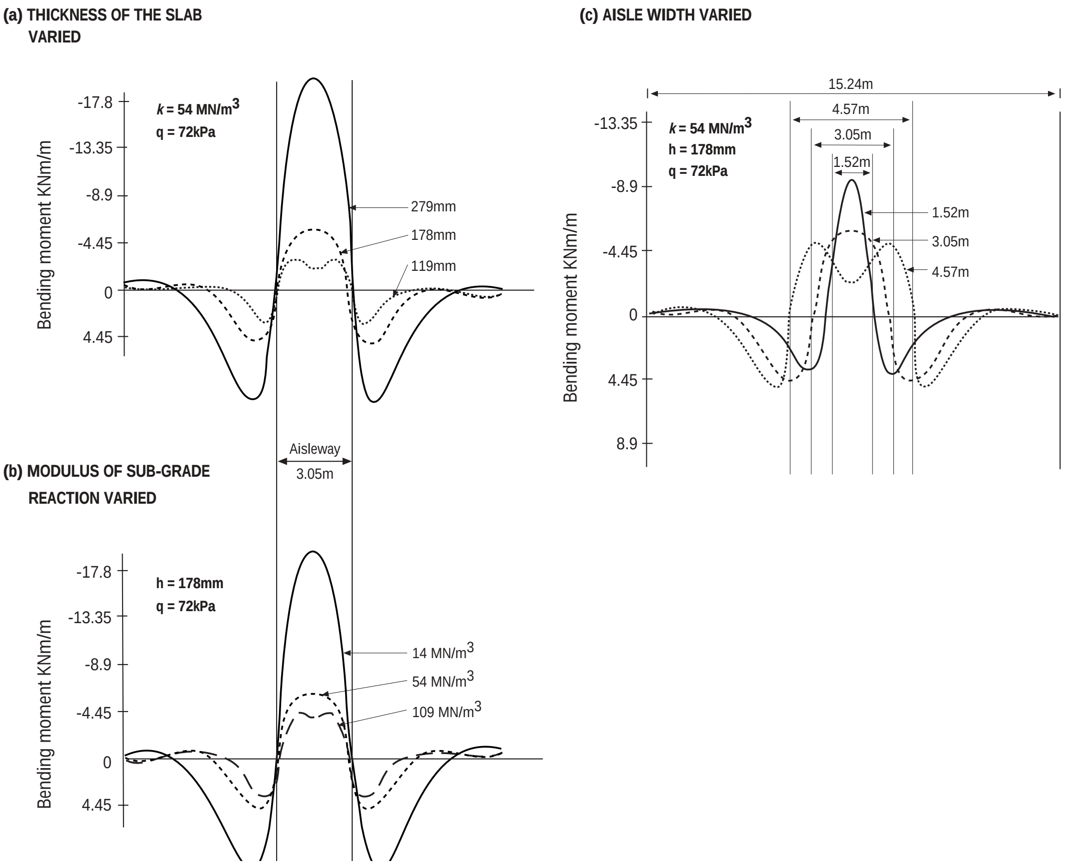
Figure 3.10 Critical bending moment for a loading of 1 kN/m2



This aisle width is known as the critical aisle width and the bending moment as the critical bending moment; any deviation from this critical aisle width would produce a smaller bending moment. The critical bending moment is usually the negative moment causing tension in the top of the slab at mid-aisle, but in the case of large aisle widths may be the positive moment. The critical bending moment is shown in Figure 3.10 for a 1 kN/m^2 uniform loading. This Figure assumes that the aisle is loaded on both sides. If the aisle is loaded on only one side, the bending moment from Figure 3.10 should be halved. The design moment is the bending moment from Figure 3.10 multiplied by the loading in kN/m^2 and by the load safety factor. The required tensile strength of the concrete is the design moment divided by the elastic section modulus. The elastic section modulus for unit width is $h^2/6$.

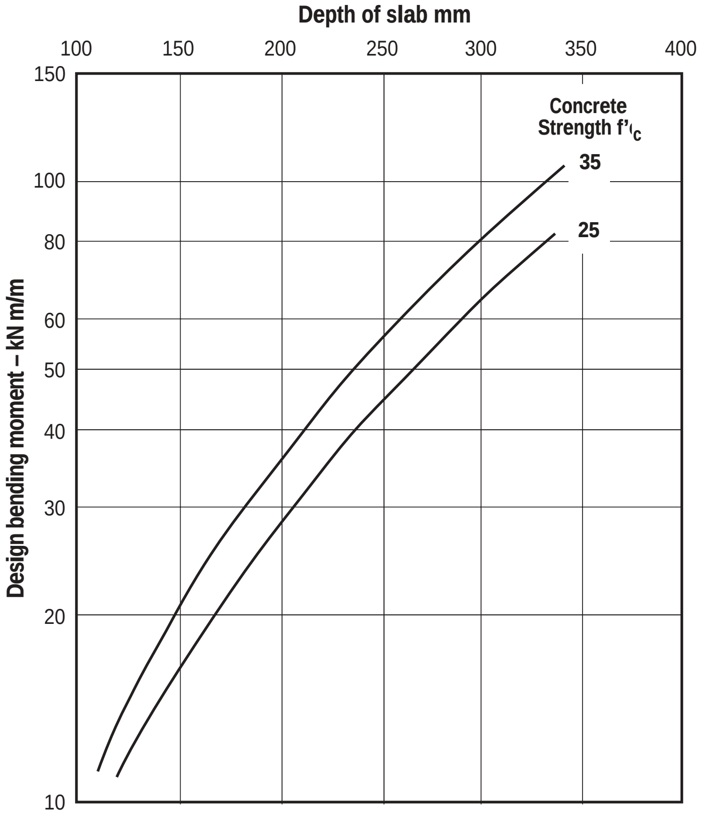
Figure 3.11 illustrates the shape of the bending moment diagram for various combinations of slab thickness, soil modulus, and aisle width as evaluated by Panak and Rauhut[20].

Figure 3.11 Transverse bending moments for a uniformly loaded slab



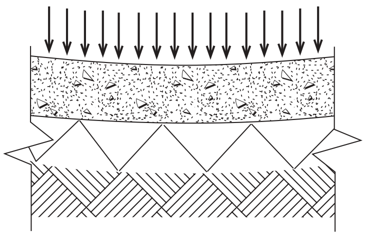
The grade of concrete required to withstand a given design bending moment for a given depth of slab may be found from Figure 3.12.

Figure 3.12 The compressive strength of concrete required to withstand the design bending moments for different depths of slab



For uniform loading, the slab and sub-base have virtually no load-spreading ability. To avoid shear failure, the loading on the slab may have to be limited to the bearing capacity of the underlying soils. Loads may also have to be limited to avoid unacceptable settlement or consolidation.

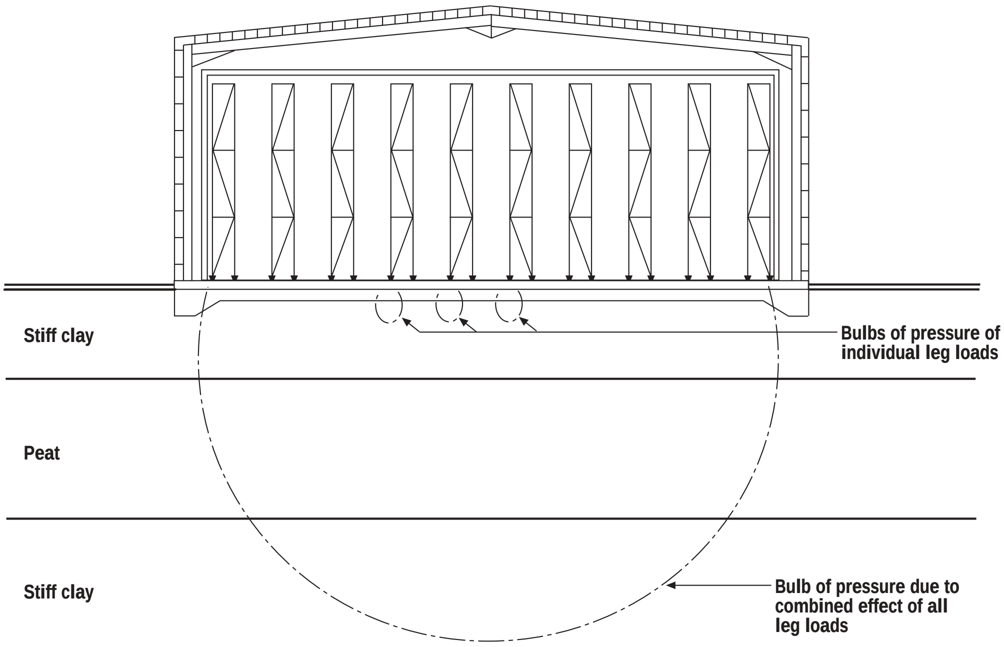
3.9.8 Deflection under uniformly distributed loads



Methods based on the modulus of sub-grade reaction have been proven to be effective and accurate design tools for the evaluation of the stresses in a floor supported on the ground. However, they are not suitable for the estimation of settlements of the slab when the loaded area is large.

The depth of influence of a load applied to a soil is a function of the size of the loaded area. Figure 3.13 illustrates the sub-surface pressure bulbs of individual racking system leg loads, and the combined effect of all the legs. In this example the combined bulb penetrates a soft peat layer, and in this situation the sub-grade may perform satisfactorily for localised loads, but the softer soil at depth may compress under the combined influences of all the localised loads. Settlement of floors due to compression of bands of weak material at depth may be considerably greater and non uniform, than the elastic settlement under individual localised loads. Careful consideration of settlements will be required when mechanical handling equipment requires tight surface flatness tolerances.

Figure 3.13 Diagram showing the increase in depth of influence due to spread of load



The deflection of the base at the centre of the uniformly distributed load can be estimated from Equation 3.7[9]:

$$s = \frac{pW(1-\upsilon_s^{\,2})w_s}{E_s} \qquad \text{[Equation 3.7]}$$

Where:

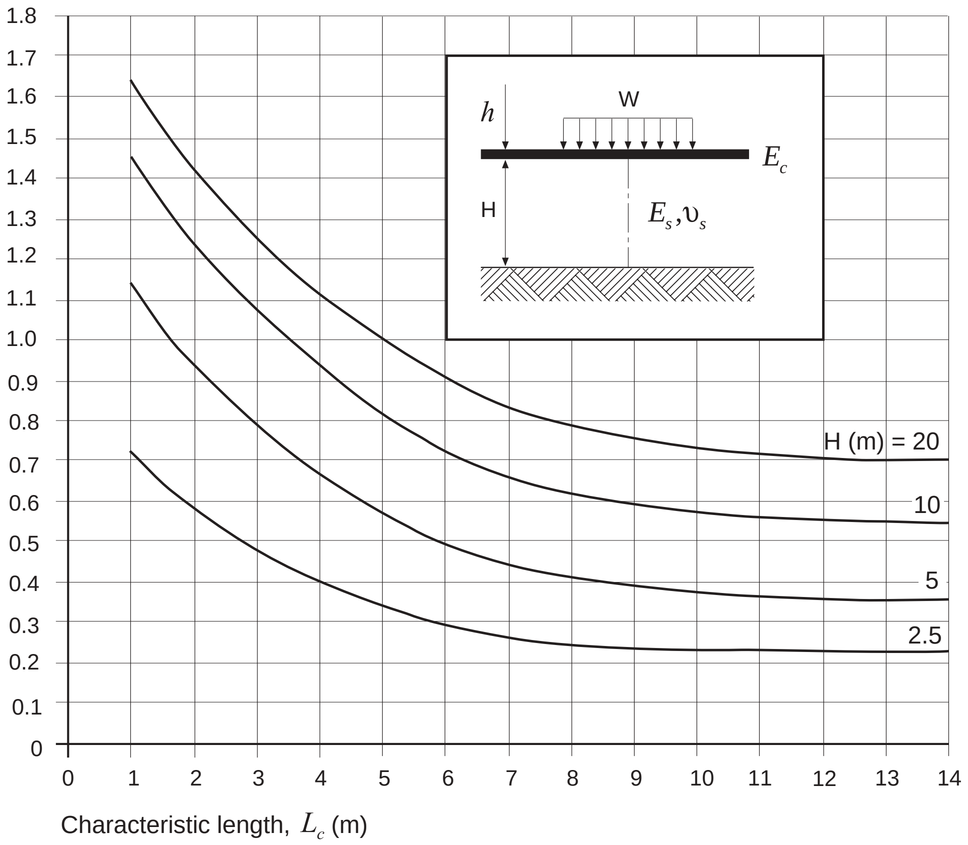
p = magnitude of uniformly distributed load (MPa)

W = width of loaded area, m

ε_s υ_s = long term Young's modulus and Poisson's ratio of the soil

w_s = dimensionless deflection factor calculated from Figure 3.14

Figure 3.14 Central deflection below distributed loading on interior of slab



The characteristic length L_c is calculated from

$$L_c = h\left[\frac{E_c}{E_s(1-\upsilon_s^{\ 2})}\right]^{0.33}$$ [Equation 3.8]

Where:

h = slab thickness, m

E_c = Young's modulus of the concrete, MPa

H = depth of equivalent uniform layer of soil, refer Figure 3.14 and reference 9.

For more information refer to reference 9.

3.9.9 Uplift Loads

Under the influence of seismic actions, it is possible for the feet of racks to exert uplift forces onto the slab. Many industrial racking systems are designed with yielding base plates that limit the uplift force on the slab. Commonly the uplift is limited to 10kN, though the seismic design philosophy may vary with different manufacturers. The slab designer should consult with potential racking system suppliers to determine an appropriate philosophy for the project.

The formulae developed by Westergaard remain valid while the slab is in contact with the ground. If uplift occurs, Westergaard's analysis is no longer applicable and the required thickness of the slab will need to be determined by computer or rational analysis. Uplift for a single load point occurs when the displacement under the load point exceeds the self-weight settlement of the slab. The following tables have been derived to determine the load at which uplift occurs for various combinations of modulus of sub-grade reaction, concrete compressive strength, and slab thickness, when the load is applied a reasonable distance from the slab edge. The information is provided as guidance for determining when Westergaard's analysis is still an appropriate analysis tool.

Table 3.4 Single point load uplift force where displacement equals self weight settlement. Loaded radius =70mm, f'c=25MPa

f'_c=25MPa	Critical Uplift force, kN Soil modulus of sub-grade reaction, MN/m^3			
Thickness, mm	15	37	54	68
100	7.07	4.18	3.37	3.79
125	12.33	7.82	7.04	5.28
150	19.42	12.74	10.78	8.79
175	28.53	17.63	15.51	13.06
200	39.81	25.44	21.29	18.59
250	69.5	43.5	36.32	33.36
300	109.57	68.18	56.38	53.55

Table 3.5 Single point load uplift force where displacement equals self weight settlement. Loaded radius =70mm, f'_c=35MPa

f'_c=35MPa	Critical Uplift force, kN Soil modulus of sub-grade reaction, MN/m^3			
Thickness, mm	15	37	54	68
100	7.51	4.44	4.02	3.58
125	13.1	8.25	7.48	6.12
150	20.63	13.53	11.45	9.72
175	30.31	18.73	16.47	14.29
200	42.3	27.03	22.62	19.92
250	73.85	46.22	38.59	34.74
300	116.4	72.45	59.91	54.78

Table 3.6 Single point load uplift force where displacement equals self weight settlement. Loaded radius =70mm, f'_c=45MPa

f'_c=45MPa	Critical Uplift force, kN Soil modulus of sub-grade reaction, MN/m^3			
Thickness, mm	15	37	54	68
100	7.87	5.03	4.16	3.72
125	13.72	8.76	7.27	6.36
150	21.63	13.8	11.44	10.19
175	31.78	20.26	16.79	14.98
200	44.34	28.27	23.42	20.88
250	77.41	49.35	40.86	36.41
300	122.06	77.78	64.39	57.42

3.9.10 TABLES OF CALCULATED SLAB THICKNESS FOR INTERNAL LOADING CONDITION

The following tables provide the calculated slab on ground thickness for various rack and wheel loading geometries, and soil/concrete strengths. Refer to Chapter 6 for examples using these tables. The following assumptions have been made in determining the tables:

Assumptions-

- The slab is founded on 150mm of granular hard fill above a sub-grade with the tabled CBR
- Fatigue is not a consideration for rack loading, ie. the expected number of load repetitions is less than 8,000.
- The relationship between CBR and modulus of sub-grade reaction, modified for the presence of the hardfill for the rack loads only, is as defined in section 3.4
- The loaded area from the base plate of racks A = 15,400mm^2 giving an equivalent radius of loaded area for the interior load case r = 70mm. $r = \sqrt{A/\pi}$
- For back-to-back racks the combined radius of the loaded area for the two adjacent load points is 100mm.
- The longitudinal spacing of the racking system legs is 2.7m.
- For forklift loads, single wheel axles are assumed, with wheel spacing as defined in Table 2.1, and tyre pressures of 700kPa.
- The load factor is 1.5. Note the load P in the following tables is the unfactored or 'working load', to which a load factor of 1.5 has been applied to determine the slab thickness.
- Tensile stresses due to restrained shrinkage are negligible.
- Load applied after 90 days.
- The tables show the calculated thickness rounded to the nearest 5mm. This is sufficient accuracy to show the influence of different load combinations and concrete strengths. However, for construction purposes it is normal to specify the floor thickness in increments of 25mm.
- The tables give the required thickness for f'_c of 25, 35, and 45MPa. The selection of the appropriate design compressive strength is in many instances determined from durability requirements. With the minimum required f'_c selected, the tables can be used to determine the required pavement thickness.
- For the edge and corner load cases, the terms 'dowels' and 'no dowels' are used. Dowels should be interpreted to mean that load transfer occurs across the joint by either mechanical devises such as dowels, or by aggregate interlock. However aggregate interlock will not occur if the joint opens up by more than 1mm. No dowels should be interpreted to mean that no load transfer occurs between adjacent slabs.

Notes on the above assumptions-

- When the load application area is small, the presence of the stiff granular hardfill increases the apparent modulus of sub-grade reaction of the foundation material. However, as the loaded area increases, the significance of the stiffening enhancement diminishes. In the following tables, the 150 mm layer of compacted granular material was only considered for the racking system load case. For wheel loads the presence of the hardfill has been ignored when determining an appropriate modulus of sub-grade reaction.
- Tensile stresses in the slab caused by restrained drying shrinkage are assumed to be negligible. This is a common assumption but is strictly only true when joints are provided at regular centres, and are not constrained. Drying shrinkage tensile stresses can be evaluated by estimating the magnitude of the frictional forces that develop along the soil/slab interface. If the calculated stresses indicate that these forces are not insignificant, then either the thickness can be calculated by superimposing the drying shrinkages stresses, or the stresses can be reduced by reducing the coefficient of friction or increasing the number of joints.

Table 3.7 Slab thickness for interior loading, single racks, with 150mm of hardfill

Sub-grade CBR %	*k* modified for hardfill MN/m^3	Load P (kN)	Distance x (mm)	Slab thickness (mm) for f'_{c28}=		
				25 MPa	35 MPa	45 MPa
2	20.5	40	800	160	145	130
			1200	155	140	125
		60	800	200	175	160
			1200	190	170	155
		80	800	230	205	190
			1200	220	195	180
5	46	40	800	155	140	125
			1200	150	135	120
		60	800	190	170	160
			1200	185	165	150
		80	800	220	200	180
			1200	210	190	175
10	65	40	800	150	135	125
			1200	150	130	120
		60	800	190	170	155
			1200	180	165	150
		80	800	220	195	180
			1200	210	190	175
20	80	40	800	150	130	120
			1200	145	130	120
		60	800	185	165	155
			1200	180	160	150
		80	800	215	195	180
			1200	210	190	170
80	160	40	800	140	125	115
			1200	135	120	110
		60	800	180	160	145
			1200	175	155	140
		80	800	210	185	170
			1200	200	180	165

Refer start of this section for assumptions and notes

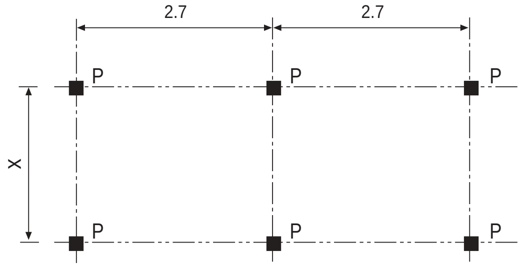


Table 3.8 Slab thickness for interior loading, back to back racks, with 150mm of hardfill

Sub-grade CBR %	*k* modified for hardfill MN/m^3	Load P (kN)	Distance x (mm)	Slab thickness (mm) for f'$_{c28}$= 25 MPa	35 MPa	45 MPa
2	20.5	40	800	225	200	185
			1200	210	190	175
		60	800	280	250	230
			1200	270	240	220
		80	800	330	295	270
			1200	315	280	260
5	46	40	800	215	190	175
			1200	205	185	170
		60	800	265	240	220
			1200	250	225	210
		80	800	310	280	255
			1200	295	265	240
10	65	40	800	210	190	170
			1200	200	180	165
		60	800	260	235	215
			1200	250	225	205
		80	800	300	270	250
			1200	290	260	240
20	80	40	800	210	185	170
			1200	200	180	165
		60	800	260	230	215
			1200	250	220	205
		80	800	300	270	250
			1200	285	260	240
80	160	40	800	200	180	160
			1200	195	170	155
		60	800	250	225	205
			1200	240	215	200
		80	800	290	260	240
			1200	280	250	230

Refer start of this section for assumptions and notes.

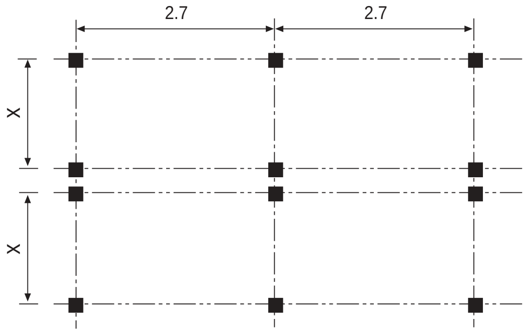


Table 3.9 Slab thickness for interior wheel loads for various forklift capacities & load repetitions with CBR=2 %						
CBR %	*k* unmodified for hardfill MN/m^3	Rated capacity (Tonnes)	Load Repetitions	Slab thickness (mm) for f'_{c28}=		
				25 MPa	35 MPa	45 MPa
2	15	2.5	>400,000	160	140	125
			200,000	150	130	120
			100,000	150	130	115
			50,000	140	125	115
			<8,000	130	115	105
		3.5	>400,000	180	160	145
			200,000	175	150	140
			100,000	170	150	135
			50,000	165	145	130
			<8,000	150	135	120
		4.5	>400,000	205	180	160
			200,000	195	170	155
			100,000	190	165	150
			50,000	180	160	145
			<8,000	170	150	135
		9	>400,000	285	250	230
			200,000	270	240	215
			100,000	265	235	210
			50,000	260	225	205
			<8,000	240	210	190
		13	>400,000	350	310	280
			200,000	335	290	265
			100,000	325	285	260
			50,000	315	275	250
			<8,000	290	255	230

Refer start of this section for assumptions and notes.
The above table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.

Table 3.10 Slab thickness for interior wheel loads for various forklift capacities and load repetitions with CBR=5 %

CBR %	*k* unmodified for hardfill MN/m^3	Rated capacity (Tonnes)	Load Repetitions	Slab thickness (mm) for f'_{c28}= 25 MPa	35 MPa	45 MPa
5	37	2.5	>400,000	145	125	115
			200,000	140	120	110
			100,000	135	120	105
			50,000	130	115	105
			<8,000	120	105	95
		3.5	>400,000	170	145	130
			200,000	160	140	125
			100,000	155	135	120
			50,000	150	130	120
			<8,000	140	120	110
		4.5	>400,000	190	165	150
			200,000	180	155	140
			100,000	175	150	135
			50,000	170	145	130
			<8,000	155	135	120
		9	>400,000	260	230	210
			200,000	250	220	195
			100,000	245	215	190
			50,000	235	205	185
			<8,000	220	190	170
		13	>400,000	320	280	255
			200,000	305	265	240
			100,000	295	260	235
			50,000	285	250	225
			<8,000	265	230	205

Refer start of this section for assumptions and notes.
The above table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.

Table 3.11 Slab thickness for interior wheel loads for various forklift capacities and load repetitions with CBR=10 %						
CBR %	*k* unmodified for hardfill MN/m^3	Rated capacity (Tonnes)	Load Repetitions	Slab thickness (mm) for f'$_{c28}$=		
				25 MPa	35 MPa	45 MPa
10	54	2.5	>400,000	140	120	110
			200,000	135	115	105
			100,000	130	115	105
			50,000	125	110	100
			<8,000	115	100	90
		3.5	>400,000	160	140	125
			200,000	155	135	120
			100,000	150	130	120
			50,000	145	125	115
			<8,000	135	115	105
		4.5	>400,000	180	155	140
			200,000	170	150	135
			100,000	165	145	130
			50,000	160	140	125
			<8,000	150	130	115
		9	>400,000	250	220	200
			200,000	240	210	190
			100,000	235	205	185
			50,000	225	195	175
			<8,000	210	180	160
		13	>400,000	310	270	240
			200,000	290	255	230
			100,000	285	250	225
			50,000	275	240	215
			<8,000	255	220	200

Refer start of this section for assumptions and notes.
The above table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.

Table 3.12 Slab thickness for interior wheel loads for various forklift capacities and load repetitions with CBR=20 %						
CBR %	*k* unmodified for hardfill MN/m^3	Rated capacity (Tonnes)	Load Repetitions	Slab thickness (mm) for f'_{c28}=		
				25 MPa	35 MPa	45 MPa
20	68	2.5	>400,000	135	120	110
			200,000	130	115	105
			100,000	125	110	100
			50,000	120	110	95
			<8,000	115	100	90
		3.5	>400,000	155	140	125
			200,000	150	130	120
			100,000	145	130	115
			50,000	140	125	110
			<8,000	130	115	100
		4.5	>400,000	175	155	140
			200,000	165	145	130
			100,000	160	140	130
			50,000	155	135	125
			<8,000	145	125	115
		9	>400,000	245	215	195
			200,000	235	205	180
			100,000	225	195	180
			50,000	220	190	170
			<8,000	200	175	155
		13	>400,000	300	260	235
			200,000	285	245	220
			100,000	275	240	215
			50,000	265	230	210
			<8,000	245	210	190

Refer section 3.9.10 for assumptions and notes.
The above table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.

3.9.11 Tables of calculated slab thickness for edge loading condition

The following tables provide the calculated slab on ground thickness for various rack loading geometries, and soil/concrete strengths where the load is applied at the edge of the slab. The assumptions made in determining the tables are identical to those defined in section 3.9.10.

Table 3.13 Slab thickness for edge loading, single racks, with 150mm of hardfill

Sub-grade CBR % [k,MN/m^3]	Load P [kN]	Distance x [mm]	Slab thickness, mm, for f'_{c28}= 25 MPa No Dowels	25 MPa Dowels	35 MPa No Dowels	35 MPa Dowels	45 MPa No Dowels	45 MPa Dowels
2 [20.5]	40	800	195	180	175	165	160	150
		1200	190	175	170	155	155	145
	60	800	240	220	215	200	200	185
		1200	230	215	210	195	190	180
	80	800	275	255	250	230	230	215
		1200	265	250	240	225	220	205
5 [46]	40	800	190	175	170	160	160	145
		1200	185	170	165	155	155	145
	60	800	235	215	210	195	195	180
		1200	225	210	205	190	190	175
	80	800	270	250	240	225	225	210
		1200	260	240	235	220	215	200
10 [65]	40	800	190	175	170	155	155	145
		1200	185	170	165	150	150	140
	60	800	230	215	210	195	190	180
		1200	225	210	200	190	185	175
	80	800	265	245	240	225	220	205
		1200	260	240	235	215	215	200
20 [80]	40	800	190	170	170	155	155	140
		1200	185	170	165	150	150	140
	60	800	230	215	205	190	190	175
		1200	225	205	200	185	185	175
	80	800	265	245	240	220	220	205
		1200	260	240	230	215	215	200
80 [160]	40	800	180	165	160	145	145	135
		1200	175	160	155	140	140	130
	60	800	225	205	200	185	185	170
		1200	220	200	195	180	180	155
	80	800	260	240	235	215	215	200
		1200	255	235	230	210	210	195

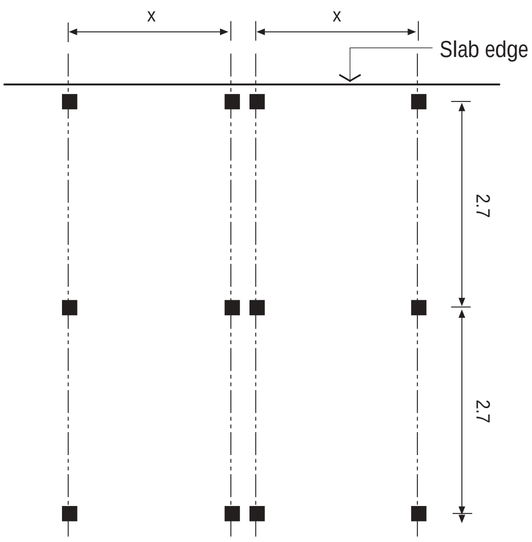
Refer section 3.9.10 for assumptions and notes; refer Table 3.7 for definition of x.

Table 3.14 Slab thickness for edge loading, back to back racks, with 150mm of hardfill

Sub-grade CBR % [k,MN/m^3]	Load P [kN]	Distance x [mm]	Slab thickness, mm, for f'_{c28}=					
			25 MPa		35 MPa		45 MPa	
			No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
2 [20.5]	40	800	285	255	255	230	230	210
		1200	270	240	240	215	220	200
	60	800	365	330	325	300	300	270
		1200	340	310	305	280	280	255
	80	800	430	390	385	355	355	325
		1200	400	365	360	330	330	300
5 [46]	40	800	260	240	235	215	215	195
		1200	250	230	225	205	205	190
	60	800	335	300	300	270	270	245
		1200	315	290	285	260	260	235
	80	800	395	360	355	325	325	295
		1200	370	340	335	305	305	280
10 [65]	40	800	255	230	230	205	210	190
		1200	245	225	220	200	200	185
	60	800	320	290	290	265	265	240
		1200	310	280	275	255	255	230
	80	800	380	345	345	310	315	285
		1200	360	330	325	300	300	270
20 [80]	40	800	250	225	225	205	205	185
		1200	240	220	215	195	200	180
	60	800	315	285	285	260	260	235
		1200	305	275	275	250	250	230
	80	800	370	340	335	305	305	280
		1200	355	325	320	295	295	270
80 [160]	40	800	235	215	210	190	190	175
		1200	230	210	205	190	190	170
	60	800	300	270	265	240	245	220
		1200	290	265	260	235	240	215
	80	800	350	320	315	290	290	260
		1200	340	310	305	280	280	255

Refer to section 3.9.10 for assumptions and the next page for notes.

A:



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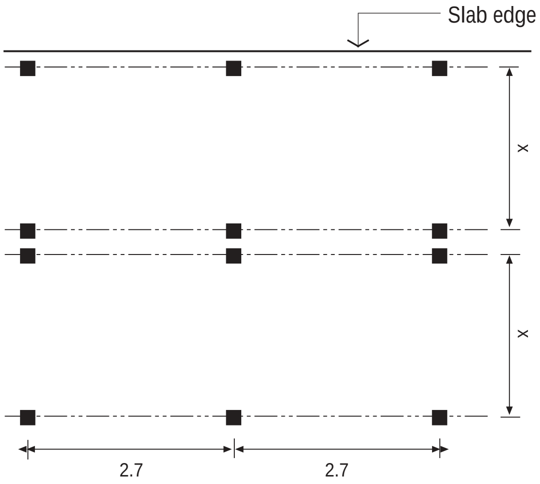

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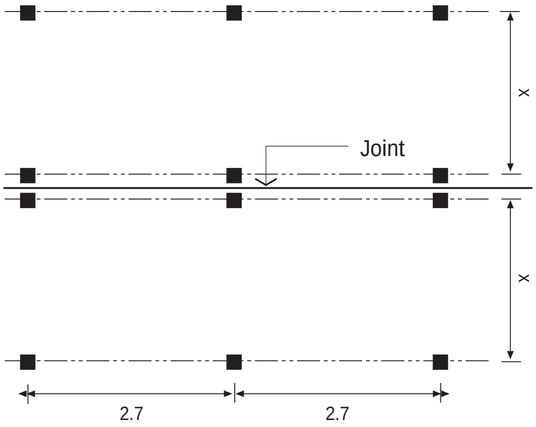


Notes:

- *For the assumption outlined in section 3.9.10, arrangement A is critical and governs the required thickness.*
- *The thickness required for arrangement C can be determined from Table 3.13, single racks for edge loading:*
  - *where no dowels are used, use the 'No Dowel' column of Table 3.13;*
  - *where dowels are used, load transfer between the two slabs will occur. The transfer back and forth cancels any stress reduction so that the no dowel column in Table 3.13 should be used to determine the required thickness.*

Table 3.15 Slab thickness for edge wheel loads for various forklift capacities and load repetitions with CBR=2 %

Sub-grade CBR % [k,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'_{c28}= 25 MPa No Dowels	25 MPa Dowels	35 MPa No Dowels	35 MPa Dowels	45 MPa No Dowels	45 MPa Dowels
2 [15]	2.5	>400,000	195	180	170	155	155	140
		200,000	185	170	165	150	150	135
		100,000	180	165	160	145	145	130
		50,000	175	160	155	140	140	125
		<8,000	165	150	145	130	130	115
	3.5	>400,000	225	210	200	180	180	165
		200,000	215	200	190	175	170	155
		100,000	210	195	185	170	170	155
		50,000	205	185	180	165	160	145
		<8,000	190	170	165	150	150	135
	4.5	>400,000	255	235	225	205	205	185
		200,000	240	220	215	195	195	175
		100,000	235	215	210	190	190	170
		50,000	230	210	200	185	180	165
		<8,000	210	195	185	170	170	155
	9	>400,000	370	335	320	295	290	265
		200,000	345	320	305	280	275	255
		100,000	340	310	295	275	270	245
		50,000	325	300	285	265	260	235
		<8,000	305	275	265	245	240	220
	13	>400,000	450	415	395	365	360	330
		200,000	430	395	430	345	340	310
		100,000	420	385	370	335	335	305
		50,000	405	370	355	325	320	295
		<8,000	375	345	330	300	300	270

Refer to section 3.9.10 for assumptions and notes.
The above table includes no consideration for abrasion resistance. 25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8. The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

Table 3.16 Slab thickness for edge wheel loads for various forklift capacities and load repetitions with CBR=5 %

Sub-grade CBR % [k,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'_{c28}=					
			25 MPa		35 MPa		45 MPa	
			No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
5 [37]	2.5	>400,000	180	165	155	145	145	130
		200,000	170	155	150	135	135	120
		100,000	165	150	145	130	130	120
		50,000	160	145	140	125	125	115
		<8,000	150	135	130	115	115	105
	3.5	>400,000	210	190	185	165	165	150
		200,000	200	180	175	160	155	140
		100,000	195	175	170	155	155	140
		50,000	185	170	165	150	145	135
		<8,000	175	155	150	135	135	120
	4.5	>400,000	235	215	205	185	185	170
		200,000	220	200	195	175	175	160
		100,000	215	200	190	170	170	155
		50,000	210	190	185	165	165	150
		<8,000	195	175	170	155	150	135
	9	>400,000	335	305	295	265	265	240
		200,000	320	290	280	255	250	230
		100,000	310	285	270	245	245	225
		50,000	300	275	260	235	235	215
		<8,000	275	250	240	220	220	200
	13	>400,000	415	380	360	330	330	300
		200,000	395	360	345	315	310	280
		100,000	385	350	335	305	305	275
		50,000	370	340	325	295	290	265
		<8,000	345	310	300	270	270	245

Refer section 3.9.10 for assumptions and notes. The above table includes no consideration for abrasion resistance. 25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8. The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

Table 3.17 Slab thickness for edge wheel loads for various forklift capacities and load repetitions with CBR=10 %

Sub-grade CBR % [k,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'_{c28}= 25 MPa No Dowels	25 MPa Dowels	35 MPa No Dowels	35 MPa Dowels	45 MPa No Dowels	45 MPa Dowels
10 [54]	2.5	>400,000	175	155	150	135	135	125
		200,000	165	150	145	130	130	115
		100,000	160	145	140	125	125	115
		50,000	155	140	135	120	120	110
		<8,000	140	130	125	110	110	100
	3.5	>400,000	200	185	175	160	160	145
		200,000	190	175	165	150	150	135
		100,000	185	170	160	145	145	130
		50,000	180	160	155	140	140	125
		<8,000	165	150	145	130	130	115
	4.5	>400,000	225	205	195	180	180	160
		200,000	215	195	185	170	170	150
		100,000	210	190	180	165	165	150
		50,000	200	180	175	160	160	140
		<8,000	185	170	160	145	145	130
	9	>400,000	325	295	280	255	255	230
		200,000	305	280	265	240	240	220
		100,000	300	270	260	235	235	215
		50,000	285	260	250	225	225	205
		<8,000	265	240	230	210	210	190
	13	>400,000	400	365	350	315	315	285
		200,000	380	345	330	300	300	270
		100,000	370	335	320	290	290	265
		50,000	360	320	310	280	280	250
		<8,000	330	300	285	260	255	230

Refer section 3.9.10 for assumptions and notes. The above table includes no consideration for abrasion resistance. 25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8. The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

Table 3.18 Slab thickness for edge wheel loads for various forklift capacities and load repetitions with CBR=20 %								
Sub-grade CBR % [k,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'$_{c28}$=					
			25 MPa		35 MPa		45 MPa	
			No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
20 [68]	2.5	>400,000	170	155	145	130	130	120
		200,000	160	145	140	125	125	115
		100,000	155	140	135	120	120	110
		50,000	150	135	130	115	115	105
		<8,000	140	125	120	110	110	95
	3.5	>400,000	195	180	170	155	150	140
		200,000	185	170	160	145	145	130
		100,000	180	165	160	140	140	125
		50,000	175	160	150	135	135	120
		<8,000	165	145	140	125	125	110
	4.5	>400,000	220	200	190	175	175	155
		200,000	210	190	180	165	165	145
		100,000	205	185	175	160	160	145
		50,000	195	175	170	155	155	135
		<8,000	180	160	155	140	140	125
	9	>400,000	315	285	275	250	250	225
		200,000	300	270	260	235	235	210
		100,000	290	265	255	230	230	205
		50,000	280	255	245	255	220	195
		<8,000	260	235	225	200	200	180
	13	>400,000	390	355	340	305	305	275
		200,000	370	335	320	290	290	260
		100,000	360	325	315	285	280	255
		50,000	345	310	300	270	270	245
		<8,000	320	290	280	250	250	225

Refer section 3.9.10 for assumptions and notes. The above table includes no consideration for abrasion resistance. 25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8. The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

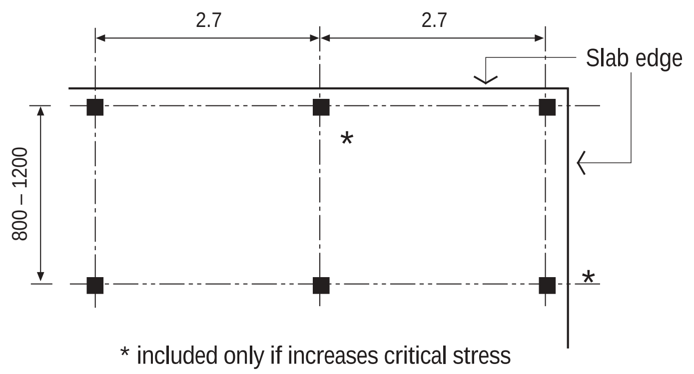
3.9.12 Tables of calculated slab thickness for corner loading condition

The following tables provide the calculated slab on ground thickness for various rack loading geometries, and soil/concrete strengths where the load is applied at the corner of the slab. The assumptions made in determining the tables are identical to those defined in section 3.9.10.

Table 3.19 Slab thickness for corner loading, single racks, with 150mm of hardfill

Sub-grade CBR % [k,MN/m^3]	Load P [kN]	Slab thickness, mm, for f'_{c28}=					
		25 MPa		35 MPa		45 MPa	
		No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
2 [20.5]	40	215	175	190	160	175	145
	60	265	220	240	200	220	180
	80	310	260	280	230	255	210
5 [46]	40	205	170	185	150	170	135
	60	260	215	230	190	210	175
	80	305	250	270	225	250	205
10 [65]	40	205	165	180	145	165	135
	60	255	210	230	185	210	170
	80	310	250	270	220	245	200
20 [80]	40	200	165	180	145	165	135
	60	255	210	225	185	205	170
	80	310	245	265	220	245	200
80 [160]	40	195	155	170	140	155	130
	60	245	200	220	175	200	160
	80	290	235	260	210	235	190

Refer section 3.9.10 for assumptions and notes.



* included only if increases critical stress

Table 3.20 Slab thickness for corner wheel loads for various forklift capacities and load repetitions with CBR=2 %								
Sub-grade CBR % [k,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'_{c28}=					
			25 MPa		35 MPa		45 MPa	
			No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
2 [15]	2.5	>400,000	195	160	170	140	160	130
		200,000	185	150	165	135	150	125
		100,000	180	150	160	130	150	120
		50,000	175	145	155	130	145	115
		<8,000	165	135	145	120	135	110
	3.5	>400,000	225	185	200	160	180	150
		200,000	215	175	190	155	175	140
		100,000	210	170	185	150	170	140
		50,000	205	165	180	145	165	135
		<8,000	190	155	170	135	155	125
	4.5	>400,000	250	205	220	180	200	165
		200,000	240	195	210	170	195	160
		100,000	235	190	205	170	190	155
		50,000	225	185	200	165	185	150
		<8,000	210	170	185	155	170	140
	9	>400,000	345	285	310	250	280	230
		200,000	330	270	295	240	270	220
		100,000	325	265	290	235	265	215
		50,000	315	255	280	225	255	210
		<8,000	295	240	260	210	240	195
	13	>400,000	420	345	375	305	340	280
		200,000	400	330	365	290	325	265
		100,000	395	320	350	285	320	260
		50,000	380	310	340	275	310	255
		<8,000	355	290	315	255	290	235

Refer to section 3.9.10 for assumptions and notes.
This table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.
The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

Table 3.21 Slab thickness for corner wheel loads for various forklift capacities and load repetitions with CBR=5 %

Sub-grade CBR % [*k*,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'_{c28}=					
			25 MPa		35 MPa		45 MPa	
			No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
5 [37]	2.5	>400,000	185	155	165	135	150	125
		200,000	180	145	160	130	145	120
		100,000	175	145	155	125	140	115
		50,000	170	140	150	120	135	110
		<8,000	160	130	140	115	130	105
	3.5	>400,000	215	178	190	155	175	140
		200,000	205	165	180	150	165	135
		100,000	200	165	180	145	160	135
		50,000	195	160	170	140	155	130
		<8,000	180	150	160	130	145	120
	4.5	>400,000	240	195	210	170	195	160
		200,000	230	185	200	165	185	150
		100,000	225	180	200	160	180	150
		50,000	215	175	190	155	175	145
		<8,000	200	165	180	145	165	135
	9	>400,000	330	270	295	240	270	220
		200,000	315	260	280	225	255	210
		100,000	310	250	275	225	250	205
		50,000	300	245	265	215	240	200
		<8,000	280	225	250	200	225	185
	13	>400,000	400	325	355	290	325	265
		200,000	385	310	340	275	310	255
		100,000	375	305	335	270	305	245
		50,000	365	295	320	260	295	240
		<8,000	340	275	300	240	275	220

Refer section 3.9.10 for assumptions and notes.
This table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.
The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

Table 3.22 Slab thickness for corner wheel loads for various forklift capacities and load repetitions with CBR=10 %

Sub-grade CBR % [k,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'$_{c28}$=					
			25 MPa		35 MPa		45 MPa	
			No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
10 [54]	2.5	>400,000	185	150	165	130	150	120
		200,000	175	145	155	125	140	115
		100,000	170	140	150	125	140	115
		50,000	165	135	145	120	135	110
		<8,000	155	125	140	110	125	100
	3.5	>400,000	210	170	185	150	170	140
		200,000	200	165	180	145	165	135
		100,000	195	160	175	140	160	130
		50,000	190	155	170	135	155	125
		<8,000	180	145	160	125	145	115
	4.5	>400,000	235	190	205	170	190	155
		200,000	225	180	200	160	180	150
		100,000	220	180	195	155	175	145
		50,000	210	170	190	150	170	140
		<8,000	200	160	175	140	160	130
	9	>400,000	325	265	285	230	260	215
		200,000	310	250	275	220	250	205
		100,000	305	245	270	215	245	200
		50,000	295	235	260	210	235	190
		<8,000	275	220	240	195	220	180
	13	>400,000	395	320	350	280	315	260
		200,000	375	305	330	265	300	245
		100,000	370	295	325	260	295	240
		50,000	355	285	315	250	285	230
		<8,000	330	265	290	235	265	215

Refer to section 3.9.10 for assumptions and notes.
This table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.
The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

Table 3.23 Slab thickness for corner wheel loads for various forklift capacities and load repetitions with CBR=20 %

Sub-grade CBR % [k,MN/m^3]	Rated capacity [Tonnes]	Load repetitions	Slab thickness, mm, for f'_{c28}=					
			25 MPa		35 MPa		45 MPa	
			No Dowels	Dowels	No Dowels	Dowels	No Dowels	Dowels
20 [68]	2.5	>400,000	180	150	160	130	145	120
		200,000	175	140	155	125	140	115
		100,000	170	140	150	120	135	110
		50,000	165	135	145	120	135	110
		<8,000	155	125	135	110	125	100
	3.5	>400,000	210	170	185	150	170	135
		200,000	200	160	175	140	160	130
		100,000	195	160	170	140	155	130
		50,000	190	155	165	135	150	120
		<8,000	175	140	155	125	140	115
	4.5	>400,000	230	190	205	165	185	150
		200,000	220	180	195	160	180	145
		100,000	215	175	190	155	175	140
		50,000	210	170	185	150	170	135
		<8,000	195	160	175	140	155	125
	9	>400,000	320	260	285	230	260	210
		200,000	305	245	270	220	245	200
		100,000	300	240	265	215	240	195
		50,000	290	235	255	205	235	190
		<8,000	270	215	240	190	215	175
	13	>400,000	390	315	340	275	315	250
		200,000	370	300	325	260	300	240
		100,000	360	290	320	255	290	235
		50,000	350	280	310	245	280	225
		<8,000	335	260	285	230	260	210

Refer section 3.9.10 for assumptions and notes.
This table includes no consideration for abrasion resistance.
25 MPa concrete is unlikely to provide the desired abrasive resistance for the tabled forklifts, refer section 3.8.
The use of dowels, or similar, to achieve load transfer along the edge is recommended for vehicle load cases.

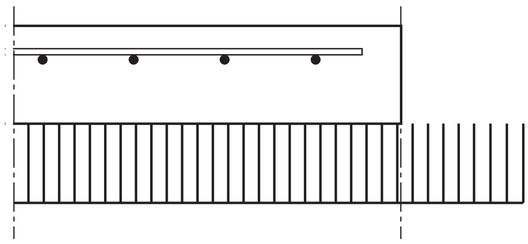
3.10 METHODS OF ANALYSIS – SHRINKAGE COMPENSATING CONCRETE FLOORS

Shrinkage compensating concrete can be produced using expansive cements or expansive components. The expansion occurs after the concrete has hardened, with the aim being that the concrete expands an amount equal to, or slightly greater than the anticipated drying shrinkage. Subsequent long term drying shrinkage will reduce these expansive strains but ideally a residual expansion will remain in the concrete, which can, with appropriate reinforcement, eliminate shrinkage cracking. Figure 3.15 gives a diagrammatic representation showing the sequence of movements of a reinforced slab, cast on ground containing shrinkage compensating admixture. Figure 3.16 illustrates conceptually the expansion/shrinkage characteristics of shrinkage compensating concrete.

Figure 3.15 Diagrammatic representation showing sequence of movements of a reinforced concrete slab, cast on the ground containing shrinkage-compensating admixture

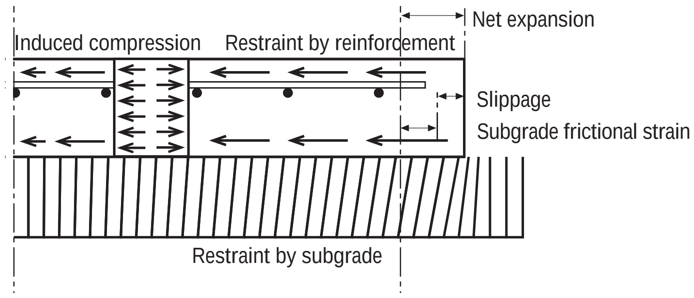
Stage 1 – Immediately after casting

The state of stress before expansion



Stage 2 – After several days' curing

Stresses and movements after expansion has occurred due to the action of the shrinkage compensation admixture



Stage 3 – After several months' drying out

Stresses and movements after shrinkage has occurred

Note the tendancy to counteract shrinkage compensation caused by subgrade friction

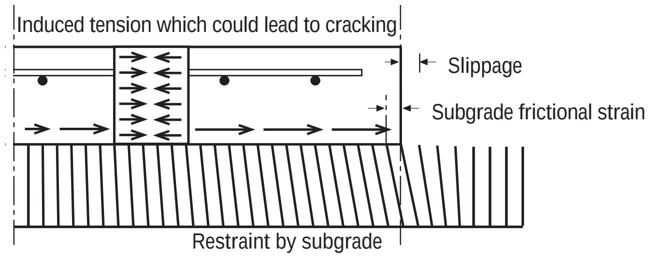
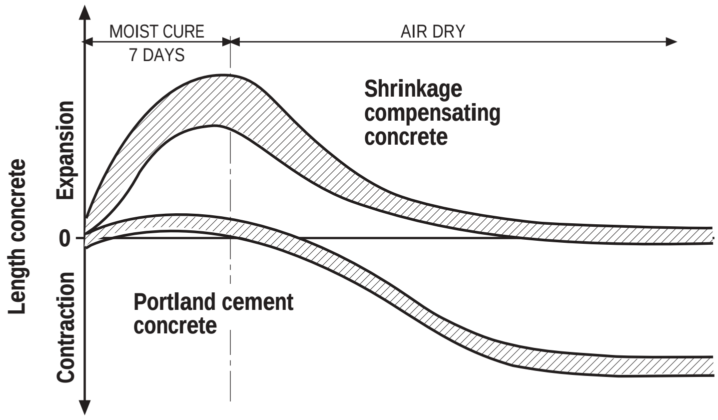


Figure 3.16 Typical length change characteristics of shrinkage-compensating and Portland cement concretes



Shrinkage reducing admixtures, which are also available, do not cause an initial expansion. This section only considers the effects caused by shrinkage compensating admixtures or cements.

An excellent guide for the use of shrinkage compensating concrete is ACI 223-98[26]. This practice guide is limited to shrinkage compensating concrete made using expansive cements.

The determination of the appropriate thickness for a floor slab using shrinkage compensating concrete is identical to that for conventionally reinforced slab outlined in section 3.9. The main advantage in using shrinkage compensating concrete is the reduction or elimination of joints and shrinkage cracking in the interior of the floor slab. The design of floors using these materials focuses on the positioning of joints, detailing to accommodate the expansion, and the required reinforcement.

The expansion of the concrete places the reinforcement into tension. Upon drying and shrinking, there is sometimes a small residual compression left in the concrete. This residual stress is ignored when determining the slab thickness.

Expansion and long-term drying shrinkage characteristics of the available expansive additives vary. The designer needs to ensure that the correct type is used for the given slab size. The larger the slab the more critical it is to chose the correct additive to achieve the desired result.

Success of this system is dependent upon the careful consideration of a few key issues; in brief these are-

It is important to ensure that the base below the floor is as smooth as practical. Two layers of polythene are used to ensure that the coefficient of friction on the base is minimised.

The floor must be isolated from the structure to allow free expansion and shrinkage movement. This is easily achieved with polystyrene between floor slab and columns.

Care should be taken to ensure adequate and consistent mixing of the additive in accordance with the supplier's recommendations. Additional care is necessary if more than one batching plant is used, particularly if there are differences in the source of any of the component materials.

Wet curing is critical with some types of expansive cements to ensure optimum hydration.

It is recommended that the supplier of the expansive additive be consulted during the design phase of the project.

Refer to Chapter 5 for the reinforcement of slabs constructed using shrinkage compensating concrete.

3.11 METHODS OF ANALYSIS – POST-TENSIONED FLOORS

Post-tensioning is the application of a permanent compressive force to the concrete, achieved by casting ducts in the slab and installing steel strands which are tensioned after the concrete has set. This additional compressive force in the concrete can be used to offset both shrinkage stresses and the applied load stress. The magnitude of residual compression is a function of the applied force, friction losses, shrinkage, creep, steel relaxation and anchorage losses. Each tendon typically consists of 3 or 4 strands in a sealed duct. The strand used is typically 12.7 mm diameter with a jacking load of 149 kN (0.8 UTS) per strand, resulting in residual post-tensioning stress of up to 2 MPa after losses. Typical losses are:

- Jacking and anchorage losses 4%
- Concrete Creep 2%
- Concrete Shrinkage 14%
- Steel Relaxation 2.5% (Low relaxation strand)

Friction between the strand and duct causes further losses which are a function of the tendon length, and result in the tensile force varying along the tendon.

Concrete shrinkage commences almost immediately after pouring, so the post-tensioning force must be applied incrementally as the concrete strength increases. The initial post-tensioning force of 15%-20% of the design force is applied within 1-2 days of pouring and must be sufficient to overcome friction between the slab and sub-grade. A low coefficient of sub-grade friction is assured by providing two layers of polythene under the slab.

The full design post-tensioning force cannot be applied until the concrete reaches full strength. The ducts are grouted after stressing is completed.

The thickness determination of a post-tensioned slab uses the method outlined in section 3.9. However, the residual post-tensioning stress significantly enhances the tensile strength of the slab section, with the resulting allowable tensile stress being:

Modulus of Rupture + Residual Post-tensioning Stress

Success of this system is dependent upon the careful consideration of a few key issues. In brief these are:

- It is important to ensure that the base below the floor is as smooth as practical. Two layers of polythene are used to ensure that the coefficient of friction on the base is minimised.
- Additional test cylinders are taken to monitor the early age strength gain of the concrete. It is also important to cure these cylinders on site in conditions that are similar to those encountered by the floor slab.
- The floor must be isolated from the structure to allow free shrinkage movement. This is easily achieved with polystyrene between floor slab and columns.

3.12 METHODS OF ANALYSIS – FIBRE REINFORCED FLOORS

Fibres can be categorised into two main groups, steel fibres and alkali resistant polypropylene fibres. Polypropylene fibres are added to concrete to provide resistance to plastic cracking. They do not provide effective strength for resistance against drying shrinkage or applied load stresses.

Steel fibres provide additional structural strength and robustness to the concrete slab. Steel fibres come in various shapes and aspect ratios, each with its own performance properties.

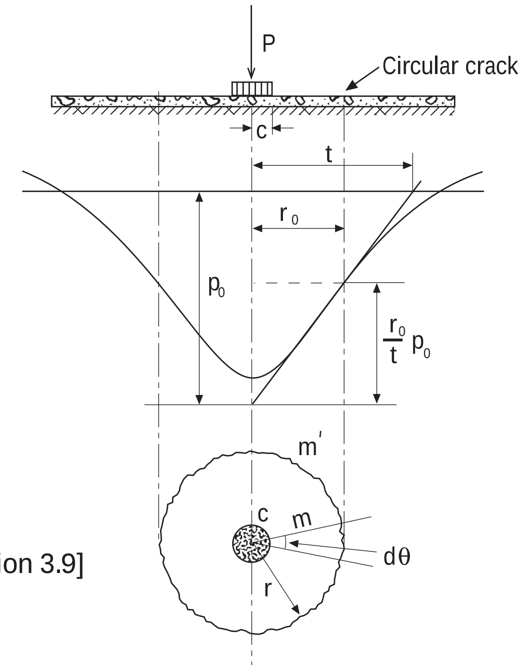
In this section we only consider steel fibres.

The thickness determination of steel fibre reinforced floors typically uses a Meyerhof analysis rather than Westergaard's. Meyerhof's[16] work is based on yield line theory, which is appropriate when the slab has ductility – the ability to maintain moment capacity for rotations beyond the ultimate capacity. Fibre reinforced slabs have this ability while conventionally reinforced slabs do not. In this section, some background information is provided on the analysis of slabs using Meyerhof analysis. In depth discussion is not provided as the industry norm for fibre-reinforced floors is for the fibre or ready mix supplier to provide a dose/thickness advisory service. Excellent software packages exist for the design of fibre reinforced floors.

Figure 3.17 illustrates the yield line pattern for a point load acting in the interior portion of a slab.

Figure 3.17 Failure line diagram for a slab on soil and the assumed soil pressure distribution under the slab

The pressure distribution may be assumed to have a form represented by a straight line between the peak value and the position of the circular crack.



For flexure Meyerhof gives the following equation for the collapse load:

$$P_o = 6\left[1+\left(\frac{2a_r}{L}\right)\right]M_o \qquad \text{[Equation 3.9]}$$

Where:

L = radius of elasticity

a_r = contact radius of load

$$a_r = \sqrt{\frac{P}{\pi\Gamma}} \qquad \text{[Equation 3.10]}$$

Where:

P = wheel load

Γ = tyre pressure

$$M_o = \left[1+\left(\frac{R_{e,3}}{100}\right)\right] x \left[\frac{f_{ct}bh^2}{6}\right] \qquad \text{[Equation 3.11]}$$

Where:

f_{ct} = flexural strength of unreinforced concrete

b = 1m

h = thickness of the slab

$R_{e,3}$= equivalent flexural strength ratio

Design values for $R_{e,3}$ are obtained by beam tests in accordance with the test procedure laid down in TR34[5] for a deflection of up to 3mm on a 450mm span beam. This test method is based on the Japanese standard (JSCE-SF4).

The values for $R_{e,3}$ are specific to the shape, strength, and aspect ratio and dosage of the steel fibre. The manufacturer should be consulted to obtain appropriate values. Some suppliers of fibre concrete have design software packages which will allow the thickness to be determined for various load positions and combinations.

Reduced slab thickness, increased spacing between joint, better crack control and possible economic savings in cost and time of construction of the slab, are all potential benefits of the use of steel fibres in slabs on ground.

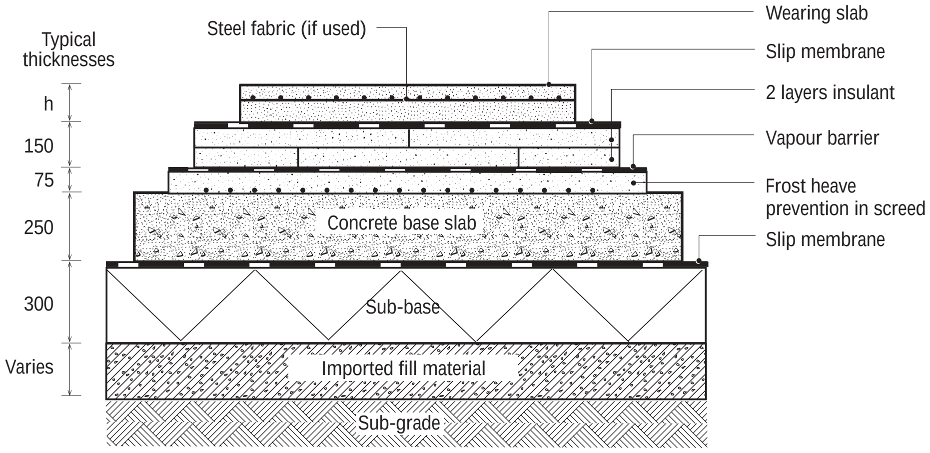
3.13 ANALYSIS OF COLD STORE FLOORS

Cold store floors are normally laid on top of polystyrene, which provides thermal insulation. The thickness determination of these concrete floors uses the approach identified in section 3.9, however the soil modulus requires modification to allow for the polystyrene.

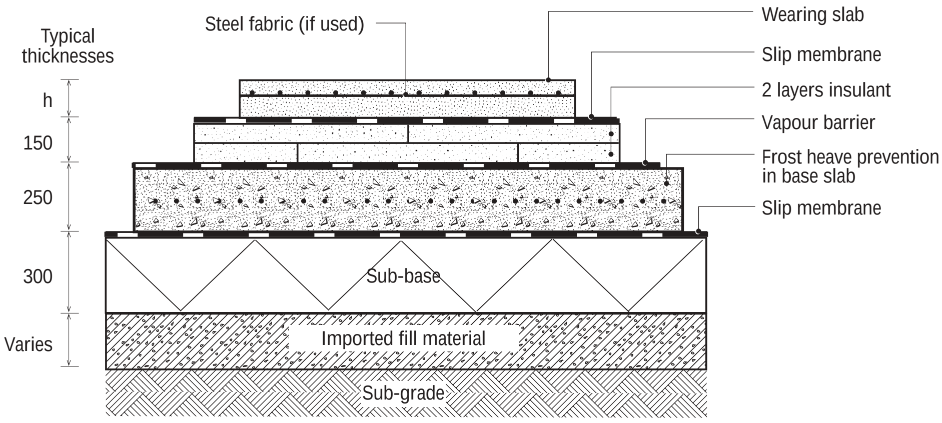
Figure 3.18 illustrates three typical profiles for a cold room floor.

Figure 3.18 Typical cold store floor construction

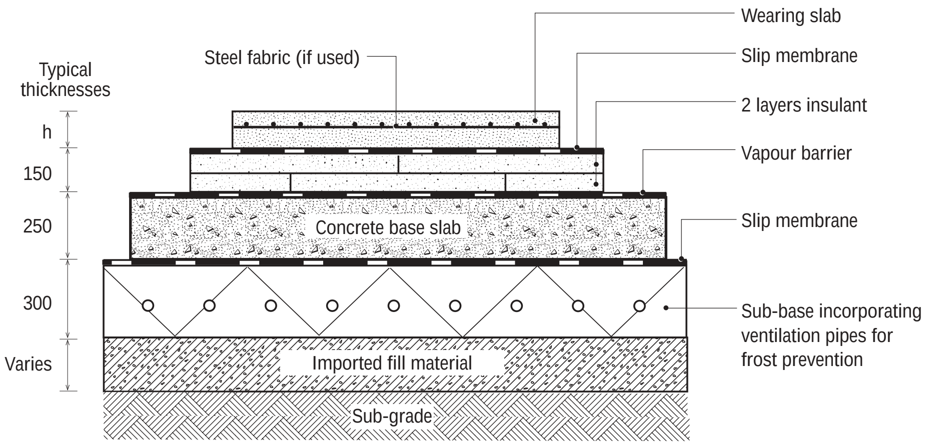
Construction Type 1



Construction Type 2



Construction Type 3



The determination of the thickness of the wearing slab involves:

- Establish the modulus of reaction of the sub-grade.
- Establish the enhancement to the modulus of sub-grade reaction of the fill material, refer Figure 3.1
- Establish the enhancement to the modulus of the base slab and heater mat protection by assuming an equivalent thickness of cement bonded subbase, refer Figure 3.1
- Establish the reduction of modulus of reaction of the heater mat screed due to the insulant layer. Refer Table 3.24
- Determine thickness in accordance with section 3.9

Designers should note that concrete will not gain strength below 0°C. 90 day concrete strengths are commonly used for many industrial complexes, and this assumption is made for the tables in section 3.9, however, this may not be appropriate for a freezer floor. The use of 90 day strengths in the design should be carefully considered in relation to the time at which the cold store is reduced to 0°C.

Table 3.24 Typical properties of various insulating materials

Product name	Equivalent Westergaard Modulus (MN/m3) as a function of insulation thickness, m					Manufacturer	Type of insulation
	0.05	0.1	0.15	0.2	0.25		
Styrofoam IB	120	60	40	30	24	Dow	Extruded polystyrene
Styrofoam SP	264	132	88	66	53	Dow	Extruded polystyrene
Styrofoam HD300	378	189	126	95	76	Dow	Extruded polystyrene

The thickness determination of the wearing slab of a cold room floor is only one aspect of the design of cold room floors. Reference 18 provides some guidance on other design issues.

The concrete used for these types of floors must have freeze thaw resistance. Guidance on appropriate air entrainment rates is provided in Part 1[2].

The concrete in freezer floors is placed at a high temperature relative to its later use. The concrete should be allowed to cure and start to dry before lowering the temperature.

When temperature draw down does occur, it is recommended[19] that the draw down should be gradual to control cracking caused by differential thermal contraction. A typical draw down schedule might be as follows:

- Ambient to 2°C — 5°C per day
- Hold at 2°C — For 2 to 5 days
- 2°C to final — 5°C per day

The low operating temperatures of cold room floors mean that joints will open more than in typical floors. Appropriate detailing will be required to accommodate this.

Sudden changes in stiffness of the material supporting the wearing slab can result in failure. At the transition between insulated and uninsulated slabs, at doors or ribbon insulation, appropriate details are required to ensure a gradual transition from one type of floor construction to the other.

The detailing of insulation and heater mat at doors requires careful detailing. Options are provided in reference 18.

4

joints

Chapter 4
Joints

4.1 INTRODUCTION

Part 1[2] of this guide provides detailed information on:

- The different types of joints
- Joint layout
- Relationship between reinforcement and joint layout
- Joint sealants
- Joint protection
- Saw cuts
- Design examples

Excluded from Part 1 were joints when the continuous pour method was used, and for post-tensioned and shrinkage compensating concrete floors. These are covered in this chapter.

4.2 JOINTS FOR THE CONTINUOUS POUR METHOD

The continuous pour method is illustrated in Figure 4.1. This construction method requires the use of temporary forms or wet screeds to achieve the surface level control. Alternatively specialist laser controlled placing machines are used, refer Figure 4.2

Figure 4.1 Pavement construction methods

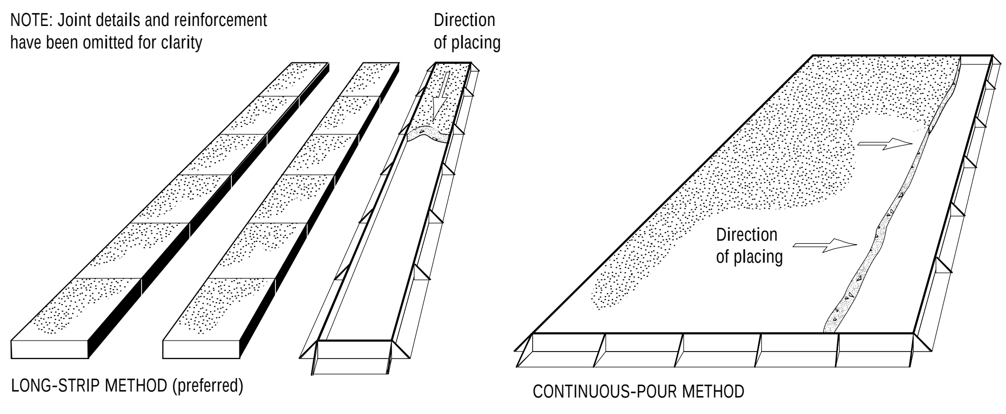


Figure 4.2 A laserscreed machine at work

The spacing requirements for both free movement joints, and tied joints (saw cuts) are as specified in Part 1[2]. The joint spacing required is identical to those for the long strip method. This is understandable as continuous pouring is simply a construction method and does not alter the physical properties of the concrete. The method does however require some thought on how the free movement joints are going to be achieved.

Several alternatives exist for the construction of the free movement joints. Some discussion of these options follows:

- Crack inducers: These typically are placed at the bottom of the pour and induce a crack in the slab during drying. To prevent movement during construction, they should be stiff. The crack created will resemble a drying shrinkage crack.
- Screed rails: These are more commonly used for the long strip construction method but are also used for the continuous pour construction technique. They can be combined with dowels for increased load transfer across the joint.
- Dowel and saw cut: A free movement joint can be created by providing a break in the reinforcement, providing a saw cut, and using a dowel with one end debonded. The dowel should be placed at mid height of the slab, and securely positioned to prevent movement during concrete placing.
- Armoured edging: Vehicle movement over joints can result in edge break. Joints are particularly vulnerable when solid tyred vehicles are used, and the opening of the joints exceeds 2-3mm. In these circumstances the trafficked edges should be armoured with steel angles or a proprietary system. The armouring may also be utilised as a screed rail to improve level accuracy.

The solution that is appropriate for a particular design problem will depend upon the joint spacing and the traffic load. When a free movement joint is expected to open more than 1mm, the transfer of load by aggregate interlock is lost. In heavily trafficked situations, it is recommended that one end debonded dowels should be used across these joints. This has economic advantages as the thickness of the slab can be reduced (refer Chapter 3), and it also helps to alleviate problems with slab curl. For continuous construction, the dowels are often required to be able to allow both longitudinal and transverse movement across the joint. Square dowels or plate dowels can accommodate this. Often, most of the drying shrinkage movement will occur at the free movement joints even though intermediate tied joint will exist. The edge protection of these joints needs consideration. When solid tyred pallet trucks are used, it is recommended that the trafficked free movement joints be protected with steel.

4.3 JOINTS FOR POST-TENSIONED SLABS

The use of post-tensioning eliminates all movement joints and saw cuts within each slab and ensures that all shrinkage movement is concentrated at the slab perimeter. The combination of post-tensioning and long-term concrete shrinkage will result in a typical 50-metre slab reducing in length by up to 40mm.

Joint-free post-tensioned slabs up to 100 metres square (10,000m^2) can be constructed in a single pour. However, outside of the main centres, the concrete supply and placing resources are often the critical factor limiting slab size. Where multiple slabs are used, the adjacent slabs are usually dowelled and edge armoured. After completion of shrinkage movement the joint may be completed with a compression seal or similar.

If a free joint between slabs is unacceptable the tendon layout may be arranged to provide a compressive load across the slab interface, thereby effectively eliminating the joint. In this case the slabs should be poured within a few days of each other to minimize differential shrinkage movement along the slab interface.

To accommodate slab shrinkage movement without cracking it is essential that the slab is isolated from all fixed structural items such as footings, columns and wall panels which may restrict movement of the slab. Typically this is achieved by providing polystyrene packing to separate the slab from these items.

4.4 JOINTS FOR SHRINKAGE COMPENSATING CONCRETE SLABS

The use of shrinkage compensating concrete allows the movement joints to be placed at greater centres than conventionally reinforced concrete floors supported on the ground. The spacing of the joints is primarily dependent upon the amount and characteristics of the expansion additive, and the amount of reinforcement.

As the design philosophy is for the expansion to equal or exceed the expected drying shrinkage, joints have to be designed to allow for the initial expansion. The joint is often filled with a compressible material. ACI 223 suggests that the width of the joint gap should be twice the expected contraction under expansion, and that the stress in the filler should be less than 172kPa when it has compressed to half its original thickness.

In New Zealand joint free slabs of 55 x 56 m have been constructed using concrete incorporating expansive cement. However, for large pours such as this the capacity of the local concrete supply and placing resources should be checked. A more typical slab size using expansive cements in New Zealand is 40m x 30m.

Saw cut joints between the isolation joints should not be required in a properly designed shrinkage compensating concrete floor, as the concrete may have a small residual compressive stress after drying shrinkage has occurred.

At isolation joints, particularly trafficked joints, the joints should be supported on ground beams, or with dowels that provide for movement in two directions. The dowel selection must consider slab dimensions and expected joint movement.

5

reinforcement

Chapter 5 Reinforcement

5.1 INTRODUCTION

Part 1[2] of this guideline provided information on the design and detailing of reinforcement in jointed concrete pavements. The design philosophy for Part 1 pavements is based on pavements reinforced against sub-base friction. Excluded from Part 1, or not commented on, and provided in this section are recommendations on:

- Unreinforced jointed pavements
- The reinforcement of continuously reinforced pavements
- The reinforcement of floors constructed with expansive cements
- The reinforcement of post-tensioned floors
- Steel fibre reinforced concrete

5.2 REINFORCEMENT GENERAL

It is generally agreed that reinforcement below a certain amount serves no real purpose. Recommendations on the magnitude of this minimum are sparse; however, based on experience and available information, it is recommended that for jointed slabs a minimum of 0.1 % for high-yield steel bars and welded-steel fabric, and 0.2% for deformed mild-steel bars.

5.2.1 Location of Reinforcement

Reinforcement should generally be located 40-50mm down from the surface. However, for post-tensioned slabs, and shrinkage compensating slabs, the reinforcement is generally placed at the mid depth of the slab.

Reinforcement should not cross free movement joints but terminate 75mm from them.

5.3 UNREINFORCED SLABS

This option is not recommended for slabs thinner than 150 mm. Joint spacing should not exceed 4 m. At the perimeter of the construction bay (e.g. long strip, wide strip or large area) joints should preferably be provided with bars for load transfer, similar to as shown as FJ3 in Figure 1.22 of Part 1[2]. At internal induced joints (similar to FJ2 of Figure 1.22, Part 1) load transfer relies upon efficiency of aggregate interlock across the crack, which will reduce if the crack opens excessively. Under heavy point loading, e.g. from racking legs, it may be necessary to improve load transfer by incorporating debonded dowels at the internal joints.

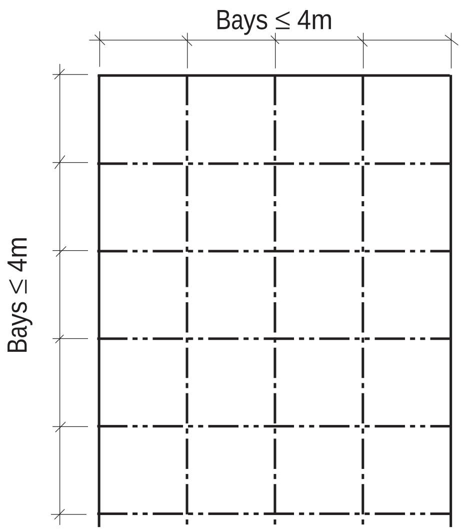
Figure 5.1 illustrates a typical joint layout.

In jointed unreinforced pavements it maybe necessary to reinforce certain panels to control or minimize the effect of cracking. Panels in which reinforcement should be provided include irregularly shaped panels, panels in which the joints cannot be aligned with the joints in adjacent panels, and panels containing pits, footings or block outs. These latter situations (where re-entrant corners are introduced into the panels) will always require trimmer bars, especially across re-entrant corners (this also applies to jointed reinforced slabs), refer Figure 5.2.

By definition, should any random cracking occur for any reason, it will be uncontrolled because of the omission of reinforcement. Careful workmanship and curing are therefore especially necessary.

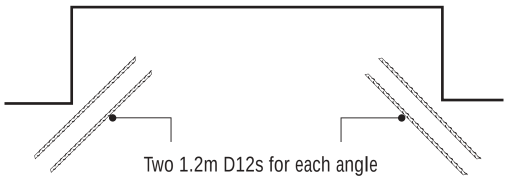
Totally unreinforced floors for commercial and industrial buildings have not been used to any great extent in the NZ context although they are used in America and on the Continent.

Figure 5.1
Unreinforced slabs – possible joint layouts



Movement joints: —-·-—-·- indicates induced contraction joint or FJ2 induced dowel contraction joint

Figure 5.2
Re-entrant angles requiring additional steel



5.4 JOINTLESS CONTINUOUSLY REINFORCED SLABS

The design of jointless continuously reinforced slabs derives from concrete road design. The philosophy is to allow the slab to crack but limit the width of the cracks. Typically the aim is to limit the width of the cracks to 0.2-0.3mm, with cracks occurring at 1-2 m centers. Although an acceptable design philosophy for a road, the suitability of cracks in a warehouse floor must be questioned and discussed with the owner.

The crack widths are mainly dependent upon the bond characteristics of the reinforcement, and the shrinkage characteristics of the concrete. Equation 5.1[32] provides an estimate of the steel area required to limit the crack widths.

$$\rho = \frac{\left(f'_t / f_b\right) d_b (\varepsilon_s + \varepsilon_T)}{2W} \qquad \text{[Equation 5.1]}$$

where:

ρ = Area of steel reinforcement/Gross cross sectional area of the concrete

f'_t / f_b = the ratio of direct tensile strength to the average bond strength between concrete and steel. Typically 1.0 is assumed for plain bars, and 0.5 for deformed bars.

d_b = bar diameter

ε_s = Estimated shrinkage strain allowing for restraint caused by the reinforcement and friction along the base. Where water reducing admixtures, and shrinkage reducing agents are used to achieve a 28 day shrinkage of less than 450 microstrains when tested in accordance with AS 1012 Part 13, this value is likely to be in the range 200 –300 microstrains. In some areas it may not be possible to achieve these low shrinkages strains, in others it may be achievable but at a cost premium above standard concrete mix design. An appropriate value to use in Equation 5.1 should be determined with consultation with the local ready mix supplier.

ε_T = Estimated maximum thermal strain from peak hydration to lowest likely temperature.

W = Maximum allowable crack width

To ensure that yielding of the steel does not occur at the crack, Equation 5.2[33] is used. This equation was developed empirically from experience with continuously reinforced concrete pavements.

$$\rho_{crit} = \frac{f_{ct}(1.3 - 0.2\mu)}{f_{sy} - mf_{ct}} \quad \text{[Equation 5.2]}$$

Where:

ρ_{crit} = the minimum proportion of longitudinal reinforcement to match the design concrete strength

f_{ct} = the concrete tensile strength. A value not exceeding 60% of the 28 day modulus of rupture may be assumed.

μ = the coefficient of friction between the slab and the subbase

f_{sy} = reinforcement yield stress

m = E_s/E_c, typically about 7.5.

In this design the recommended minimum amount of reinforcement for slabs which exceed a length or width of 60 m is 0.6%. For slab lengths below 60m, the required amount of reinforcement may be determined from ground friction formulae. See section 3.3.3.1, p26 of the CCANZ publication *Concrete Ground Floors and Pavements* Part 1[2].

The reinforcement is installed at mid-depth of the slab.

5.5 SHRINKAGE COMPENSATING CONCRETE FLOORS

Reinforcement should be placed in accordance with the expansive cement supplier's recommendations. Generally this requires reinforcing steel to be placed in the centre of the slab in both directions.

The quantity of reinforcing required to achieve adequate prestress varies with the slab dimensions. Suppliers can provide technical support to designers for the selected expansive additive.

A comprehensive commentary on reinforcing in shrinkage compensated floors may be found in ACI 223-98, which covers minimum reinforcement, reinforced slabs on grade and provides for design examples in appendices [reference 26].

5.6 POST-TENSIONED FLOORS

Post-tensioned floors incorporate anchorages, cast into the slab edge, and connected to steel tendons in ducts which traverse the length and width of the floor slab. The tendons are normally tensioned in two stages. The first stage (alternate tendons) should be tensioned as early as possible after the concrete reaches a crushing strength of about 15MPa. A final concrete compression stress of about 2MPa after deferred losses and elastic shortening will usually be sufficient to overcome slab sub-grade friction and reduce or eliminate the formation of shrinkage cracking. All restraints to slab movement should be eliminated prior to post-tensioning. The boundary edges of the slab should be free to move without restraint as should movements past slab penetrations otherwise the pretension compression will not be free to enter the slab.

Anchorage splitting reinforcing may be calculated by rule of thumb as follows. The area of reinforcing to prevent concrete edge tension splitting adjacent to anchorages is calculated as 4% of the anchorage force inducing mild steel stresses at a level of 150 MPa in the confining reinforcing (working stress). Anchorage bursting reinforcement should be placed according to the manufacturers recommendations.

The spacing of the anchorages at the edge of the slab should allow the development of uniform compression stresses in as short a distance from the free slab edge as possible. Where ducts must be curved to clear openings then friction and wobble calculations for curvature will become necessary. This may require additional tendons to offset these curvature losses.

Reinforcing steel should be applied around openings and at any positions where differential strains occur as between compressed and uncompressed concrete so that induced cracking is reduced or eliminated.

5.7 FIBRE-REINFORCED CONCRETE FLOORS

5.7.2 General

Fibre-reinforced concrete is concrete made with hydraulic cements containing fine and coarse aggregates and discontinuous steel fibres. The addition of steel fibres to cement, mortar and concrete provides a means of improving several of their engineering properties, such as fatigue resistance, impact resistance, flexural strength, etc. [27]

5.7.2 Steel-Fibre-Reinforced Concrete (SFRC) Slabs On Grade

The main advantages of SFRC include increased flexural strength, reduced cracking, and reduction in slab thickness.

Steel fibre characteristics

Steel fibres are typically used in amounts up to about 50 kg/m^3. They are produced by cutting or chopping wire or by a melt-extraction process.

Design

Guidance on the design, construction and specification of SFRC floors may be obtained from literature supplied by manufacturers[29,44,45]. Reference 28 recommends that control joints should be sawn to a depth of at least one third of the slab thickness. On the other hand, ACI 544.3R[30] suggests that sawcuts should extend to a depth of one half to two thirds that of the slab.

While the utilization of SFRC slabs permits increases in joint spacing – up to 8m [29] joint spacing is often not increased due to the location of columns, and the risk of joint spalling associated with greater joint opening. (This effect could be minimized or eliminated by adopting dowelled joints.) In some cases SFRC floors have been constructed without joints[30].

Impact resistance

The use of steel fibres improves the impact resistance.

6

design examples

Chapter 6
Design examples

6.1 CONVENTIONALLY REINFORCED SLAB SUBJECT TO FORKLIFT AND UNIFORM LOADS

Design problem

Design a slab on ground for the following loads:

- A forklift with a 9 tonne capacity, with 100,000 load repetitions during the design life
- Uniform loads of 40 kPa acting over widths of 2m and an aisle width of 3m

Geotechnical investigation have revealed competent gravel to great depth with overlying sandy silts with a CBR of 10%

Design analysis

We will assume that the loading and subsurface conditions have been examined and it has been determined that the expected long-term settlements are acceptable. It is also assumed that the bearing capacity of the soil will not impose restrictions on the allowable load on the slab.

The heavy forklift means that durability of the slab surface should be considered. The tyres of the forklift are pneumatic, and using Table 1.2 as a guide, it is decided to use a design 28 day compressive strength of 35 MPa.

The forklift will travel over joints, so it is decided to provide dowels at all joints.

The construction schedule means that the full design load will be applied after 90 days.

We will determine the required thickness using two approaches: using the tables provided in Chapter 3 and by calculation.

Using Tables 3.11, 3.17 and 3.22 for a 9 tonne rated capacity forklift, f'_{c28}=35 MPa, dowelled joints, and 100,000 repetitions provides the following required thicknesses:

- Load on the interior of the slab: 205 mm
- Load on the edge of the slab: 235 mm
- Load on the corner of the slab: 215 mm

Alternatively, 45 MPa concrete could be specified which will give improved durability, and reduced thickness. The tables in Chapter 3 allow the designer to quickly determine the economic implication of these sorts of decisions.

The tables in Chapter 3 assume single wheel axles – refer to assumptions listed in section 3.9.10. After consultation with the client it is determined that the forklift has dual wheels. It is decided to calculate the thickness to determine the benefit that can be achieved through the improved load spread.

Let's assume that the configuration of the wheels is as shown in Table 2.1, and the operating tyre pressure is 700 kPa.

$$\text{Load per tyre} = \frac{\text{axle load}}{4} = \frac{20 \text{ x } 9.81}{4} = 49.05 \text{ kN}$$

$$\text{Effective radius per tyre} = \sqrt{\frac{49.05}{700\pi}} = 0.149 \text{ m}$$

From Table 2.1, TC=300 mm. Therefore the centreline distance between the two wheels is likely to be less than twice the slab thickness, therefore combine as illustrated in Figure 3.4.

$$\text{Effective loaded radius of pair of tyres} = \sqrt{0.149^2 + \frac{2x0.3x0.149}{\pi}} = 0.225 \text{ m}$$

for the interior, and corner load case, and equal to $\sqrt{2}x0.225 = 0.318$ m for edge loading.

The applied load equals $2x49.05 = 98.1$ kN., or approximately 9.81 tonnes.

Use a load factor of 1.5, so the design load = $1.5x98.1 = 147.2$ kN

Modulus of rupture using Equation 3.1 is:

$$f_r = 0.456 k_1 k_2 (f'_c)^{0.66} = 0.456 \text{ x } 1.1 \text{ x } 0.84(35)^{0.66} = 4.4 \text{ MPa}$$

Young's modulus of Concrete from NZS 3101

$$E_c = (3320\sqrt{f'_c} + 6900) = (3320\sqrt{35} + 6900) = 26{,}540 \text{ MPa}$$

Assume modulus of sub-grade reaction, for CBR = 10% from Figure 1.1 = 54 MN/m^3.

Assume, as a first guess, that the thickness is 225 mm, radius of relative stiffness from Equation 3.3 is:

$$\ell = \left[\frac{Eh^3 x10^3}{12(1-\mu^2)k}\right]^{0.25} = \left[\frac{26{,}540x225^3x10^3}{12(1-0.15^2)54}\right]^{0.25} = 831 \text{ mm}$$

distance between two load points WC = 1750 mm from Table 2.1

Consider first the edge load case, as the previous analysis suggested that this will govern the required thickness.

The stress under a point load acting on the edge is given by Equation 3.5

$$\sigma_e = 5.19\,(1+0.54\,\mu)\ \frac{P}{h^2}\left[4\log\left(\frac{\ell}{b}\right) + \log\left(\frac{b}{25.4}\right)\right] \text{ x } 10^6 \text{kPa}$$

$$r < 1.7h \Rightarrow b = (1.6\,r^2 + h^2)^{0.5} - 0.675h$$

$$b = (1.6 \text{ x } 318^2 + 225^2)^{0.5} - 0.675 \text{ x } 225 = 309 \text{ mm}$$

$$\sigma_e = 5.19\,(1+0.54 \text{ x } 0.15)\ \frac{9.81x1.5}{225^2}\left[4\log\left(\frac{831}{309}\right) + \log\left(\frac{309}{25.4}\right)\right] \text{ x } 10^6$$

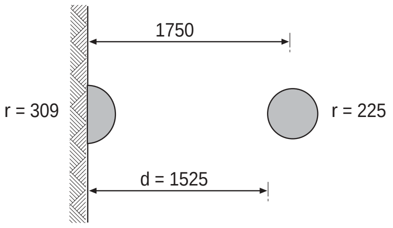
$$= 4.57 \text{ MPa}$$

Because the edge is dowelled, can expect some load transfer so multiply by 0.85, therefore

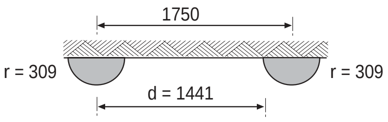
$$\sigma_e = 0.85 \text{ x } 4.57 = 3.89 \text{ MPa}$$

Consider now the stress from the other wheel, there are three cases to consider:

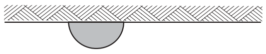
A:



B:



C:



In the case of C the other wheel is off the slab.

Stress below interior load point, from Equation 3.2

$$\sigma_i = 2.7\,(1 + \mu)\,\frac{P}{h^2}\left[4\log\left(\frac{\ell}{b}\right) + 1.069\right] \text{x}\,10^6\ \text{kN/m}^2$$

$$b = (1.6\,r^2 + h^2)^{0.5} - 0.675h = (1.6 \text{ x } 225^2 + 225^2)^{0.5} - 0.675 \text{ x } 225 = 210 \text{ mm}$$

$$\sigma_i = 2.7\,(1 + 0.15) \text{ x } \frac{9.81x1.5}{225^2}\left[4\log\left(\frac{831}{210}\right) + 1.069\right] \text{x}\,10^6 = 3.12 \text{ MPa}$$

For case A, $d/\ell = \frac{1525}{831} = 1.84$ from Figure 3.5, tangential stress from interior load at edge = 6.5% σ_i

$\Rightarrow$ combined stress at edge for Case A $= 3.89 + \frac{6.5}{100}$ x 3.12 = 4.09 MPa < 4.4 MPa

For case B $d/l = \frac{1441}{831} = 1.73$, from figure 3.7, radial stress from other edge load $= -\ 11\%\,\sigma_e$

combined stress at edge for case B $= 3.89 - \frac{11}{100}3.89 = 3.46$ MPa < 4.4 MPa

For case C, $\sigma_e = 3.89$ MPa < 4.4 MPa

Consideration of dual tyres, with increased spacing between the tyres, will allow the thickness to be reduced to 225 mm from a value of 235 obtained from the tables. See if the thickness can be reduced to 200 mm.

$$\ell = \left[\frac{26{,}540 \times 200^3 \times 10^3}{12(1-0.15^2)54}\right]^{0.25} = 760\text{mm}$$

For edge case b = $(1.6 \times 318^2 + 200^2)^{0.5}$ - 0.675 x 200 = 314 mm

For interior case b = $(1.6 \times 225^2 + 200^2)^{0.5}$ - 0.675 x 200 = 213 mm

$$\sigma_e = 5.19\,(1+0.54 \times 0.15) \times \frac{9.81x1.5}{200^2}\left[4\log\left(\frac{760}{314}\right)+\log\left(\frac{314}{25.4}\right)\right] \times 10^6 = 5.4 \text{ MPa}$$

The presence of dowel reduces σ_e to = 5.4 x 0.85 = 4.59 MPa

$$\sigma_i = 2.7\,(1 + 0.15) \times \frac{9.81x1.5}{200^2}\left[4\log\left(\frac{760}{213}\right)+1.069\right] \times 10^6 = 3.75 \text{ MPa}$$

For case A $d/\ell = \frac{1525}{760} = 2.0$ from Figure 3.5, percentage of stress = 4.9%

Combined stress for case A = $4.59 + \frac{4.9}{100} \times 3.75 = 4.76$ MPa > 4.4 MPa

For case B $d/\ell = \frac{1441}{760} = 1.9$, from Figure 3.7 percentage of stress = - 11%

Combined stress for case B = $4.59 - \frac{11}{100} \times 4.59 = 4.09$ MPa < 4.4 OK

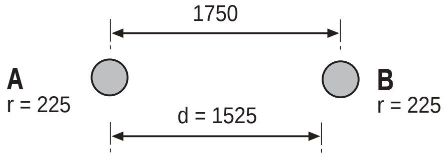
For load case C = 4.59 MPa, > 4.4 MPa, 4% overstress.

The analysis shows that the edge of the slab will be slightly overstressed. The designer may wish to re check the assumptions regarding the load repetitions, which if found to be conservative might allow the use of a 200 mm thick slab. Alternatively use a 225 mm thick slab on the edges.

Check interior load case

Assume a 200 thick slab.

From previously: $\sigma_i = 3.75$ MPa, $\ell = 760$ mm



$d/\ell = \frac{1525}{760} = 2.0$, from Figure 3.5, the percentage of stress from load point B at load point A is:

tangential stress = 4.9%

radial stress = - 7.6%

the tangential stress will govern

combined stress = $3.75 + \frac{4.9}{100} \times 3.75 = 3.93$ MPa < 4.4 MPa

Check to see if thickness can be reduced to 175 mm

$$\ell = \left[\frac{26{,}540 \text{ x } 175^3 \text{ x } 10^3}{12(1-0.15^2)54}\right]^{0.25} = 688 \text{ mm}$$

$$b = (1.6 \text{ x } 225^2 + 175^2)^{0.5} - 0.675 \text{ x } 175 = 216 \text{ mm}$$

$$\sigma_i = 2.7\,(1 + 0.15) \text{ x } \frac{9.81 \text{ x } 1.5}{175^2}\left[4\log\left(\frac{685}{216}\right) + 1.069\right] \text{x } 10^6 = 4.6 \text{ MPa}$$

$$d/\ell = \frac{1525}{688} = 2.2$$

From Figure 3.5, the percentage of stress from load point B at load point A is:

tangential stress = 3.6%

radial stress = - 7.4%

combined stress = $4.60 + \frac{3.6}{100} \text{ x } 4.6 = 4.77 \text{ MPa} > 4.4 \text{ MPa}$

Cannot reduce slab thickness to 175 mm for interior load case, therefore leave as 200 mm thick.

Check corner load case

Assume a 200 thick slab, from Equation 3.6

$$\sigma_c = 41.2\,\frac{P}{h^2}\left[1 - \frac{(r/\ell)^{0.5}}{0.925 + 0.22(r/\ell)}\right] \text{x } 10^6 \text{ kPa}$$

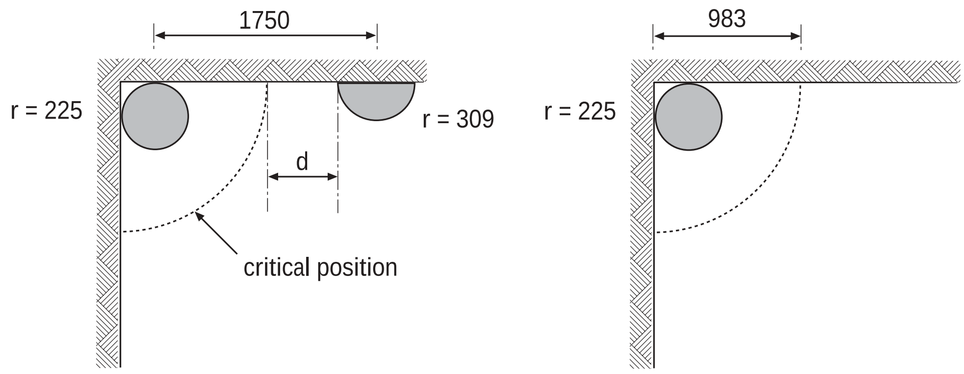
$$\sigma_c = 41.2 \text{ x } \frac{1.5 \text{ x } 9.81}{200^2}\left[1 - \frac{(225/760)^{0.5}}{0.925 + 0.22(225/760)}\right] \text{x } 10^6 = 6.83 \text{ MPa}$$

Because the joints are dowelled, the stress is reduced to:

$$\sigma_c = 0.7 \text{ x } 6.83$$
$$= 4.78 \text{ MPa}$$

Consider two cases:

A: B:



The radius of the critical position $= 2\sqrt{\left(\sqrt{2}\right)r\ell}$

$$= 2\sqrt{\sqrt{2}\,x 225 x 760} = 983 \text{ mm}$$

$$d = 225 + 1750 - 309 - 983 = 683 \text{ mm}$$

$$d/\ell = \frac{683}{760} = 0.9$$

The corner load causes tensile stresses in the top of the slab at the critical position. Figure 3.7 shows that the radial stress from the edge load $0.9\ell_{critical}$ from the load point causes a small tensile stress at the position, therefore case A will govern.

The combined stress at the critical position = $4.78 + \frac{1}{100} 4.59 = 4.83 > 4.4$ MPa

Increasing the slab thickness to 225 mm gives a combined stress at the critical position of 3.95 MPa, which is less than 4.4 MPa and is therefore OK.

Therefore a 35 MPa, 200 mm thick slab is adequate for the interior load case, but slightly overstressed at the edges and corners. At these locations a 225 mm thick slab is required. The designer can look at providing edge thickenings as specified in 3.9.6, or simply use a 225 mm thick slab throughout. A single thickness slab is the preferred option.

Uniform loading

The design uniformly load = 1.5 x 40 = 60 kPa,

ℓ = 760 mm

From figure 3.10, with an aisle width of 3 m, bending moment for unit load = 0.16 kNm

$\Rightarrow$ Design moment = 0.16 x 60 = 9.6 kNm/m

$$\text{stress} = \frac{6M}{h^2} = \frac{6 \times 9.6}{0.2^2} = 1.44 \text{ MPa} < 4.4, \text{OK}$$

200 mm thick slab OK for uniformly distributed load.

Joint Design

Joint location, reinforcing and construction methods are all interrelated. In this example, the designer may choose to locate the joints below the aisles of uniform loading. This minimises the number of joints which will be trafficked. The longitudinal joint can therefore be provided at 5, 10, 15 or 20m. The width will be dependent upon local construction resources. For this example we will assume that vibrating truss screeds are locally available which can accommodate a 10 m wide pour. Therefore free joints are provided at 10 m centres longitudinally. Typical details are provided in Figure 1.22 of Part 1.

Table 1.12 of Part 1 provides guidance on the spacing of free joints. Using 661 mesh a transverse joint spacing of 25 m is assured. Provide tied saw cut joints at 5.0 m centres (refer section 3.3.3.2 of Part 1).

6.2 CONVENTIONALLY REINFORCED SLAB SUBJECT TO PALLET RACKING LOADS

Design Problem

Design a slab on ground for the following loads

- Back to back racks which create a point load 60 kN each leg with a transverse spacing of 800 mm and a longitudinal spacing of 2700 mm between the legs
- 2 tonne capacity solid tyred forklift

Geotechnical conditions are identical to problem 6.1, with 150 mm of hardfill provided below the slab.

Design Analysis

Similar to problem 6.1, we will assume analysis had determined that the long term ground settlements are acceptable.

The solid tyred forklift will place considerable durability demands both on the concrete surface and the joints. Using Table 1.2 as a guide, it is decided to use a design 28 day compressive strength of 35 MPa.

To improve the performance of the joints, and reduce curl, it is decided to dowel all joints.

Again the construction schedule means that full loads will not be applied until after 90 days.

We will determine the required thickness using two approaches, using tables provided in Chapter 3 and by calculation.

Using tables 3.8, 3.14 and 3.19 for 60 kN back to back racks, with dowels, the following thicknesses are required:

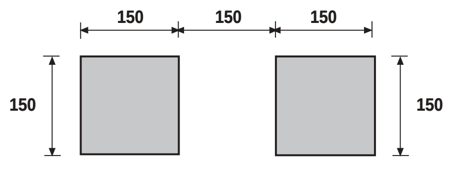
- load on the interior of the slab 235 mm
- load on the edge of the slab 265 mm for Case A or 210 mm for Case C (refer Table 3.14 for summary of case type)
- load on the corner of the slab 185 mm

The tables in Chapter 3 do not provide information for a 2 tonne capacity forklift but the required thickness for a 2.5 tonne capacity forklift with > 400,000 repetitions is:

- load on the interior of the slab 120 mm (Table 3.11)
- load on the edge of the slab 135 mm (Table 3.17)
- load on the corner of the slab 130 mm (Table 3.22)

The tables show that the racks will govern the thickness of the slab. The tables however are based upon an assumption that the plate area is 15,400 mm and that for back to back racks, the base plates are hard up against each other.

After consultation with the racking system supplier, the base plates are specified to be 150 mm square and 150 mm apart.



From Figure 3.4:

The equivalent radius = $\sqrt{\dfrac{150x450}{\pi}}$ = 146 mm compared with 100 mm used in the tables

The equivalent half radius at the edge of a slab = $\sqrt{2}$ x 146 = 207 mm

For the single legs r = $\sqrt{\dfrac{150^2}{\pi}}$ = 84 mm

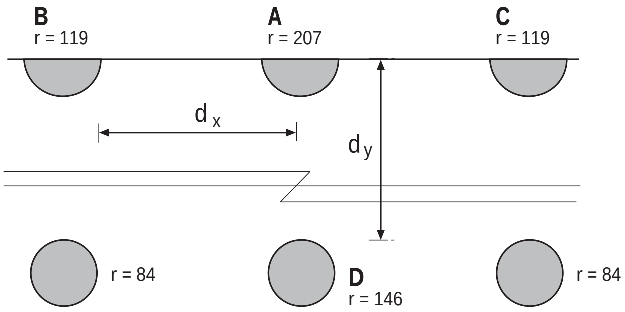
Try a 250 thick slab and assume that Case A (refer Table 3.14) is the case we are checking for.

For a CBR of 10, Figure 1.1 provides a k = 54 MN/m^3 if 150 mm of hardfill is provided the modified value of k from Figure 3.1 is k' = 65 MN/m^3.

From Equation 3.3:

$$\ell = \left[\frac{Eh^3 \text{ x } 10^3}{12(1-\mu^2)k}\right]^{0.25}$$

$$= \left[\frac{26{,}540 \text{ x } 250^3 \text{ x } 10^3}{12(1-0.15^2)65}\right]^{0.25} = 859 \text{ mm}$$



If the longitudinal spacing between legs is 2700 mm and the transverse is 800 mm:

$$\frac{dx}{\ell} = \frac{\text{spacing between legs + half(plate length + space between plates) - radius}}{\ell}$$

$$= \frac{800 - 0.5(150 + 150) - 119}{859} = 0.97$$

$$\frac{dy}{\ell} = \frac{2700 - 146}{859} = 2.97$$

From Equation 3.4

$$\text{b} = (1.6\text{r}^2 + \text{h}^2)^{0.5} - 0.675\text{h}$$

$$\text{b} = (1.6 \text{ x } 207^2 + 250^2)^{0.5} - 0.675 \text{ x } 250 = 193 \text{ mm}$$

With a load factor of 1.5, the edge stress of A is:

$$\sigma_{e,A} = 5.19\,(1 + 0.54\mu)\,\frac{P}{h^2}\left[4\log\left(\frac{\ell}{b}\right) + \log\left(\frac{b}{25.4}\right)\right] \text{x } 10^6$$

$$= 5.19\,(1 + 0.54 \text{ x } 0.15)\,\frac{1.5 \text{ x} 6 \text{x } 2}{250^2}\left[4\log\left(\frac{859}{193}\right) + \log\left(\frac{193}{25.4}\right)\right] \text{x } 10^6$$

$= 5.6$ MPa, with dowels, stress reduced by 15%

$$\Rightarrow \sigma_{e_A} = 0.85 \text{ x } 5.61 = 4.76 \text{ MPa}$$

Stress at point B, C:

$$\text{b} = (1.6 \text{ x } 119^2 + 250^2)^{0.5} - 0.675 \text{ x } 250 = 123 \text{ mm}$$

$$\sigma_{e,B,C} = 5.19\,(1 + 0.54 \text{ x } 0.15) \text{ x } \frac{1.5\text{x}6}{250^2}\left[4\log\left(\frac{859}{123}\right) + \log\left(\frac{123}{25.4}\right)\right] \text{x } 10^6$$

$= 3.28$ MPa, with dowels, stress is reduced by 15%

$$\Rightarrow \sigma_{e_{B,C}} = 0.85 \text{ x } 3.28 = 2.79 \text{ MPa}$$

Stress at point D:

$$b = (1.6 \times 146^2 + 250^2)^{0.5} - 0.675 \times 250 = 142 \text{ mm}$$

From Equation 3.2:

$$\sigma_{i,D} = 2.7\,(1 + \mu)\,\frac{P}{h^2}\left[4\log\left(\frac{\ell}{b}\right) + 1.069\right] \times 10^6$$

$$= 2.7\,(1 + 0.15) \times \frac{1.5x6x2}{250^2}\left[4\log\left(\frac{859}{142}\right) + 1.069\right] \times 10^6$$

$$= 3.75 \text{ MPa}$$

Combine stresses

radial stress at A from B, $\frac{d_x}{\ell} = 0.97$ from page 81

From Figure 3.7:

percentage stress at 0.97ℓ = -4%

tangential stress at A from D, $\frac{d_y}{\ell} = \frac{2700 - 146}{859} = 2.97$

From Figure 3.5:

percentage stress of 2.97ℓ = 1.0%

$\Rightarrow$ stress at A due to loads at A, B, C and D:

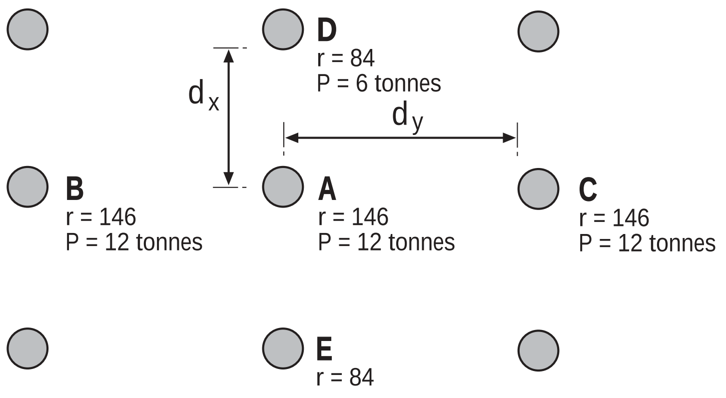
$$\sigma = 4.76 + 2 \times \frac{-4}{100} \times 2.79 + \frac{1.0}{100} 3.75 = 4.57 \text{ MPa}$$

Ultimate strength from Equation 3.1:

$$f_r = 0.456 \times 1.1 \times 1.0 \times (35)^{0.66} = 5.24 \text{ MPa} > 4.57 \text{ MPa} \Rightarrow \text{OK}$$

Therefore by using a larger base plate, and providing some spacing between the legs it was possible to decrease the required edge thickness from 265 to 250 mm. Alternatively, the information obtained from the tables indicates that case A (refer Table 3.8) governs the slab thickness. The designer could provide a joint layout which eradicates the possibility of this joint and racking arrangement.

Consider the load case where the leg loads are all distant from a joint:



Try using a 225 thick slab.

$$b_{A,B,C} = (1.6 \text{ x } 146^2 + 225^2)^{0.5} - 0.675 \text{ x } 225 = 139 \text{ mm}$$

$$b_{D,E} = (1.6 \text{ x } 84^2 + 225^2)^{0.5} - 0.675 \text{ x } 225 = 96 \text{ mm}$$

$$\ell = \left[\frac{26{,}540 \text{ x } 225^3 \text{ x } 10^3}{12(1-0.15^2)65}\right]^{0.25} = 794 \text{ mm}$$

From Equation 3.2:

$$\sigma_{i,A,B,C} = 2.7\,(1 + 0.15) \text{ x } \frac{1.5 \text{ x } 6 \text{ x } 2}{225^2}\left[4\log\left(\frac{794}{139}\right) + 1.069\right] \text{ x } 10^6$$

$$= 4.52 \text{ MPa}$$

$$\sigma_{i,D,E} = 2.7\,(1 + 0.15) \text{ x } \frac{1.5 \text{ x } 6}{225^2}\left[4\log\left(\frac{794}{96}\right) + 1.069\right] \text{ x } 10^6$$

$$= 2.62 \text{ MPa}$$

$$\frac{d_x}{\ell} = \frac{800 + 0.5(150 + 150) - 84}{794} = 1.09$$

$$\frac{d_y}{\ell} = \frac{2700 - 146}{794} = 3.22$$

Combine stresses:

σ_i = stress at A + stress at A due to tangential stress from D and E + stress at A from radial stress from B and C

Stress at A due to tangential stress from D, from figure 3.5, with $1.1\ell = 0.17\,\sigma_{D,E}$

Stress at A due to radial stress from C, from figure 3.5 with $3.22\ell = -0.05\,\sigma_{C\,B}$

$\Rightarrow \sigma_i = 4.52 + 2 \text{ x } 2.62 \text{ x } 0.17 + 2 \text{ x } (-0.05) \text{ x } 4.52 = 4.96 \text{ MPa} < 5.24 \text{ MPa}$ OK

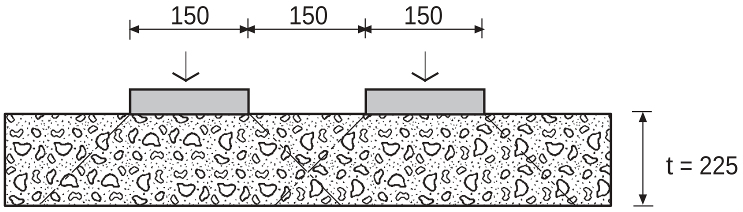
With consideration for the larger base plate size, the thickness could be reduced to 225 mm for the interior load case.

Check bearing capacity under plate using NZS 3101

For an interior load position bearing strength $P_b = 2 \text{ x } 0.85 \text{ x } f'_c A$
$= 2 \text{ x } 0.85 \text{ x } 35 \text{ x } 0.15 \text{ x } 0.15$
$= 1.34$ MN

$\Rightarrow$ LF x load $\leq \phi\, P_b = 0.65 \text{ x } 1.34 = 870$ KN
LF x load = 1.5 x 60 = 90 KN << 870 KN $\Rightarrow$ bearing OK

Check punching shear using NZS 3101 for an interior load position:



The distance between base plates is less than t, therefore consider as one combined load:

$$v_n = \frac{P}{b_o t} = \frac{2 \text{ x } 60 \text{ x } 1.5 \text{ x } 10^3}{(2(150+225)+2(450+225)) \text{ x } 225} = 0.38\text{MPa}$$

The above conservatively assumes no direct transfer of the load to the sub-grade below the load point.

$$v_c = \text{the smaller of } 0.17\,(1 + 2\beta_c)\sqrt{f'_c} \quad \text{or} \quad 0.17\left(1+\frac{\alpha_s d}{2b_o}\right)\sqrt{f'_c} \quad \text{or} \quad 0.33\sqrt{f'_c}$$

where:

β_c = ratio of short side to long side = $\frac{150}{450}$ = 0.33

α_s = 40 for interior loads

d = thickness

$$v_c = 0.17\,(1 + 2 \text{ x } 0.33)\sqrt{35} = 1.67 \text{ MPa} \qquad [A]$$

or $$v_c = 0.17\left(1+\frac{40x225}{2x2100}\right)\sqrt{35} = 3.16 \text{ MPa} \qquad [B]$$

or $$v_c = 0.33\sqrt{35} = 1.95 \text{ MPa} \qquad [C]$$

Equation A governs

v_n must be less than $\phi 1.67$

$v_n = 0.38 < 0.75 \text{ x } 1.67 = 1.25$ MPa OK

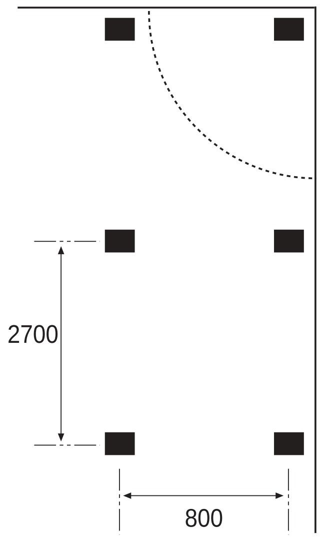
$\Rightarrow$ Punching shear OK even for conservative assumptions. The designer should check the punching shear for single legs and at the slab edge although this example demonstrates that punching shear is rarely critical.

Therefore 225 mm slab is OK for interior load condition.

Return to a consideration of the edge load case. Assume it is possible through careful consideration of the joint positions, to remove load case A shown in Table 3.14. Look at the option of placing the joint between the back to back legs. For this situation, the load case is similar to the single rack edge load case without dowels, regardless of whether dowels are supplied.

From Table 3.13, the required thickness is 210 mm. Therefore a 225 mm thick slab would be OK for interior and edge load cases.

Consider now the corner load case:



Assume the above load arrangement is a situation that requires checking.
Using Equation 3.6:

$$\sigma_c = 41.2 \text{ x } \frac{P}{h^2} \left[1 - \frac{(r/\ell)^{0.5}}{0.925 + 0.22(r/\ell)}\right] \text{x } 10^6 \text{ MPa}$$

$$= 41.2 \text{ x } \frac{6\text{x}1.5}{225^2} \left[1 - \frac{(84/794)^{0.5}}{0.925 + 0.22\left(\frac{84}{794}\right)}\right] \text{x } 10^6 \text{ MPa}$$

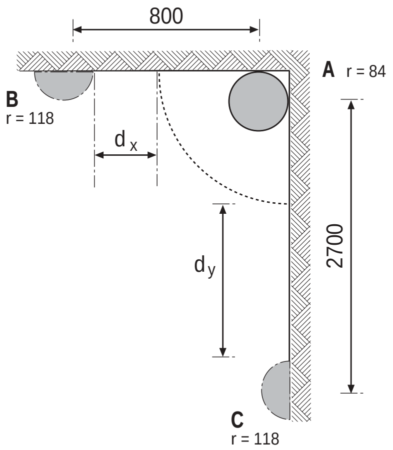
$$= 4.82 \text{ MPa}$$

With dowels multiply by 0.7, therefore:

$$\sigma_c = 0.7 \text{ x } 4.82 = 3.4 \text{ MPa}$$

$$\text{position of critical stress} = 2\sqrt{\left(\sqrt{2}\right)r\ell} = 2\sqrt{\left(\sqrt{2}\right)84 \text{x} 794} = 614 \text{ mm}$$

$$d_x = 800 + 84 - 118 - 614 = 152, \text{ mm}$$



$$d_y = 2700 + 84 - 118 - 614 = 2052 \text{ mm}$$

$$\frac{d_x}{l} = \frac{152}{794} = 0.19$$

$$\frac{d_y}{\ell} = \frac{2052}{794} = 2.58$$

From Figure 3.7:

$$\frac{d_x}{\ell} = 0.19 \text{ produces tension in the bottom} \Rightarrow \text{ not critical}$$

$$\frac{d_y}{\ell} = 2.58 \text{ produces tension in top, } = 10\% \text{ of stress at point C}$$

Edge stress at load point C, from Equation 3.4

$$b = (1.6 \text{ x } 118^2 + 225^2)^{0.5} - 0.675 \text{ x } 225 = 118 \text{ mm}$$

$$\sigma_{e,c} = 5.19\,(1 + 0.54 \text{ x } 0.15) \text{ x } \frac{1.5 \text{ x } 6}{225^2}\left[4\log\left(\frac{794}{118}\right) + \log\left(\frac{118}{25.4}\right)\right] \text{ x } 10^6$$

$$= 3.97 \text{ MPa with dowels multiply by } 0.85$$

$$\sigma_{e,c} = 0.85 \text{ x } 3.97 = 3.37 \text{ MPa}$$

$\Rightarrow$ combined stress at critical location:

$$\sigma_A = 3.4 + \frac{10}{100} \text{ x } 3.37 = 3.73 \text{ MPa} < 5.24 \text{ MPa} \Rightarrow \text{ OK}$$

Consider now the seismic load case.

Assume that the nominated racking system supplier uses a yielding base plate concept to limit the magnitude of the uplift forces during an earthquake. Using the NZ loading standard, NZS4203, the following loads acting a the base of the legs have been determined.

Dead+ live load: 60kN

Dead+Live+Earthquake: 120kN transverse EQ force causing maximum compression

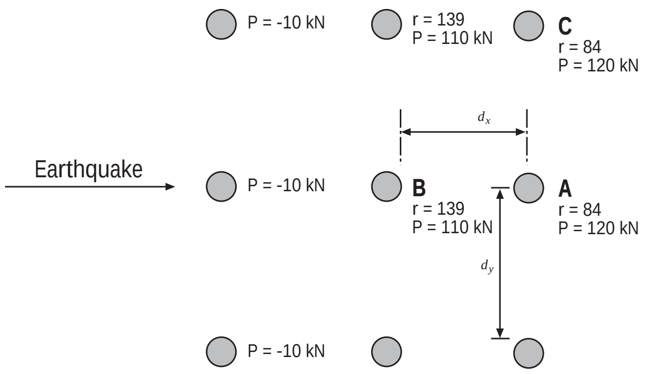
Dead+Live+Earthquake: -10kN transverse EQ force causing minimum compression

Note negative implies uplift.

Consider the loads acting a distance from the slab edge.

Table 3.5, provides guidance on the individual point uplift force required to cause the slab to just lift off the ground. For a 225 mm thick slab with k=68MN/m^3, the uplift force to cause lift off is in excess of 19kN. As the maximum uplift force is less than 10 kN, lift off does not occur and Westergaard analysis is still applicable.

When using earthquake load combinations it is appropriate to use a load factor of 1.0.



From inspection the critical position is under point A.

Stress under A, from Equation 3.2 is:

$$\sigma_{i,A} = 2.7(1+0.15)\,x\,\frac{12}{225^2}\left[4\log\left(\frac{794}{96}\right)+1.069\right] x\ 10^6 = 3.48\ \text{MPa}$$

$$\sigma_{i,B} = 2.7(1+0.15)\,x\,\frac{11}{225^2}\left[4\log\left(\frac{794}{139}\right)+1.069\right] x\ 10^6 = 2.76\ \text{MPa}$$

$$\frac{d_x}{l} = \frac{800 + 0.5(150 + 150) - 139}{794} = 1.02$$

$$\frac{d_y}{l} = \frac{2700 - 84}{794} = 3.29$$

From Figure 3.5:

Stress at A due to the tangential stress from a load at B= 20% of the stress at B

Stress at A due to the radial stress from a load at B= 0% of the stress at B

Stress at A due to the tangential stress from a load at C= 0.6% of the stress at C

Stress at A due to the tangential stress from a load at C= -4.5% of the stress at C

Combined stress at A is as follows:

$$\sigma_{i,A} = 3.48 + \frac{20}{100}\ x\ 2.76 - \frac{4.5}{100}\ x\ 2x\ 3.48 = 3.72\ \text{MPa}$$

or:

$$\sigma_{i,A} = 3.48 + \frac{0}{100}\ x\ 2.76 + \frac{0.6}{100}\ x\ 2\ x\ 3.48 = 3.5\ \text{MPa}$$

In each case the combined stress at A is less than the 5.24 MPa threshold.

Therefore a 225mm thick slab is OK for seismic load combinations where the legs are distant from the edge.

6.3 POST-TENSIONED FLOOR SLAB

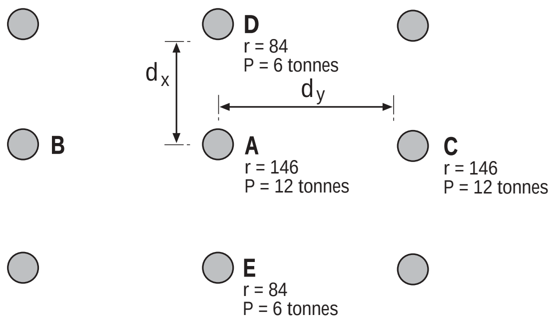
Design Problem

Assume an identical design problem to section 6.2, but evaluate the benefits of using a post-tensioned floor.

Assume that the residual compressive stress in the concrete due to the post-tensioning after all losses is 2.0 MPa.

From previous example f_r = 5.24 MPa, therefore factored loads must cause a stress less than 5.24 + 2 = 7.24 MPa.

Consider the interior load case:



In the conventionally reinforced slab, the required thickness was 225 mm. For the post-tensioned slab try 200 mm:

$$\ell = \left[\frac{Eh^3 \text{ x } 10^3}{12(1-\mu^2)k}\right]^{0.25} = \left[\frac{26{,}540 \text{ x } 200^3 \text{ x } 10^3}{12(1-\mu^2)65}\right]^{0.25} = 726\text{mm}$$

$$b_{A,B,C} = (1.6 \text{ x } 146^2 + 200^2)^{0.5} - 0.675 \text{ x } 200 = 137 \text{ mm}$$

$$b_{D,E} = (1.6 \text{ x } 84^2 + 200^2)^{0.5} - 0.675 \text{ x } 200 = 92 \text{ mm}$$

From Equation 3.2:

$$\sigma_{i\,A,B,C} = 2.7\,(1 + 0.15) \text{ x } \frac{1.5 \text{ x } 6 \text{ x } 2}{200^2}\left[4\log\left(\frac{726}{137}\right) + 1.069\right] \text{ x } 10^6$$

$$= 5.54 \text{ MPa}$$

$$\sigma_{i\,D,E} = 2.7\,(1 + 0.15) \text{ x } \frac{1.5 \text{ x } 6}{200^2}\left[4\log\left(\frac{726}{92}\right) + 1.069\right] \text{ x } 10^6$$

$$= 3.25 \text{ MPa}$$

$$\frac{d_x}{\ell} = \frac{800 + 0.5(150 + 150) - 84}{726} = 1.19$$

$$\frac{d_y}{\ell} = \frac{2700 - 146}{726} = 3.52$$

Combine stresses:

σ_i = stress at A + stress at A due to tangential stress from D & E + stress at A from radial stress from B & C

Stress at A due to tangential from D, from figure 3.5 with $1.19\ell = 0.15\sigma_{i,D,E}$

Stress at A due to radial stress from C, from figure 3.5 with $3.52\ell = -0.04\,\sigma_{i,A,B,C}$

$\sigma_i = 5.54 + 2 \text{ x } 3.25 \text{ x } 0.15 + 2 \text{ x } (-0.04) \text{ x } 5.54 = 6.07 \text{ MPa} < 7.24 \text{ MPa}$ OK

[Note if reduce thickness to 175 mm, $\sigma_i = 7.96 > 7.24$ MPa NG]

Use a 200 thick slab with 2 MPa residual post-tensioning compressive stress.

7

computer design software

Chapter 7

Computer Design Software

7.1 INTRODUCTION

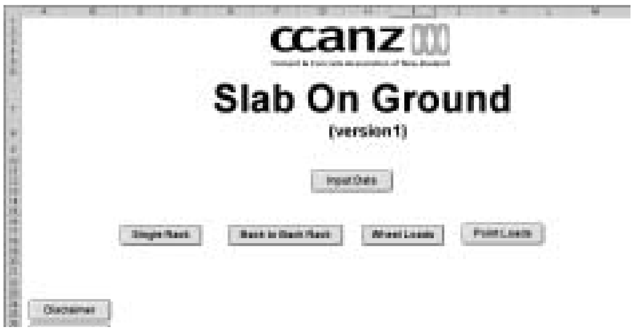
Powerful desktop computers and software allow designers to verify and refine the thickness design of pavements. These programs are design tools and engineering judgment is always required to ensure the model assumptions are valid for the final pavement detail.

Numerous computer programs are available in New Zealand, Australia and overseas that can used to analyse industrial pavements. Some relevant ones are described below.

7.2 CCANZ SLAB ON GROUND

CCANZ Slab On Ground[34] is a slab on ground analysis excel spreadsheet based upon the Westergaard analysis outlined in this design guide. The program allows the required thickness, or slab stresses to be calculated for a limited number of load combinations.

Figure 7.1 CCANZ Slab On Ground program



The load combinations considered are:

•Single racking systems

•Back to back racking systems

•Forklift wheel loads

•Individual point loads.

7.3 SAFE

SAFE[35] is distributed in New Zealand and is a powerful slab design and analysis tool. SAFE is a finite element program specifically formulated for the design and analysis of on ground and suspended floors.

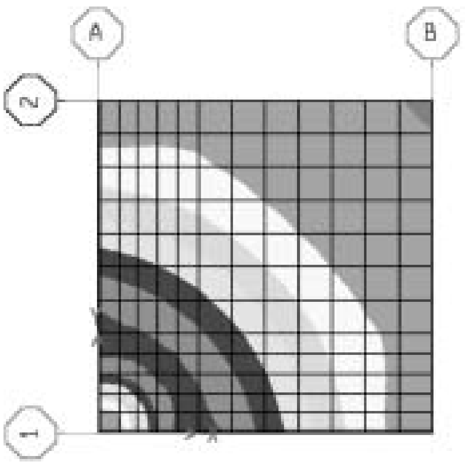
The program models a concrete ground slab as a finite element mesh of 2-D shell elements. Automatic mesh generation is often employed. The shell elements may be isotropic or orthotropic, thin or thick plate bending element. Thin plate elements are typically three to four-node elements based upon the classical linear thin plate bending theory, neglecting the effects of out-of-plane shear deformations. Thick plate elements account for the effects of out-of-plane shear deformations.

Contraction and isolation joints in ground slabs may be modelled by altering the fixity of the appropriate degrees of freedom between shell elements on either side of the joints. Assigning both a moment and a shear release at the joint location effectively models a free edge. This could be used to model isolation joints or induced contraction joints with no steel across the joint.

Assigning a moment release only effectively models a tied joint, where there is full shear transfer (displacement compatibility) across the joint ,but no moment transfer (rotation compatibility). This could be used to model tied joints.

For doweled (debonded) contraction joints, there may be only partial shear transfer across the joint. Theoretically this could be accounted for by modelling a modified shear stiffness across the joint, however there may be uncertainty as to appropriate stiffness values. In lieu of details

Figure 7.2
Slab Moment Diagram (Mmax) for load case with load applied to the corner



on joint stiffness, doweled contraction joints could be modelled with both a moment and a shear release, and peak edge stresses modified as with the conventional analysis method.

Slab loads are defined as point, line or area loads. For concentrated area loads, such as wheel and leg loads, the load should be applied over an effective area that accounts for slab thickness and relative stiffness of the slab and sub-grade.

Soil support is defined by specifying the modulus of sub-grade reaction of the soil for regions of the slab. The program generates equivalent mesh point linear elastic springs based upon the tributary areas of the regions. Additional line supports or beams may be added to account for edge footings or restraint by other structure.

No-tension surface support conditions are required when uplift may occur. This requires iterative non-linear analysis with soil supports modelled as compression only springs.

Results are typically presented in the form of colour contour plots for easy assessment, refer Figure 7.2. Peak stresses and deflections may be compared directly with specified acceptance limits. Slab parameters such as thickness may then be revised and the system re-analysed to achieve compliance or to optimise the design.

7.4 STRAND 6

STRAND 6[36] is an Australian designed finite element analysis software package for PCs. The package is flexible and can carry out various linear type analyses, and more specialised material non-linear, geometric non-linear and natural frequency analyses.

The software can have ASCI or graphical inputs for user flexibility Output is in ASCI and 2-D or 3-D graphic representations.

For pavements and sub-grades, the user can choose from various plate and brick elements to suit the design refinements. Concrete joints may be modelled with various specialised beams to allow shear, tension and compression behaviour.

7.5 PCA-MATS

PCA-Mats[37] is a computer program for the analysis of pavements, foundation mats, combined footings, and pavements The base is modelled as an assemblage of rectangular finite elements. The boundary conditions may be the underlying soil, nodal springs, or translational and rotational nodal restraints.

The model is analysed under static loads that may consist of uniform (surface) and concentrated loads. The resulting deflections, soil pressure (or spring reactions) and bending moments are output. In addition, the program computes the required area of reinforcing steel in the base.

PCA-Mats uses thin plate bending theory and the Finite Element Method (FEM) to model the behaviour of the mat or base. The soil supporting the base is assumed to behave as a set of compression-only springs (Winkler foundation). If, during the analysis, a nodal uplift is detected, the corresponding spring stiffness is updated and the mat is re-analysed.

7.6 RIGID 5

Rigid 5[38] is a spreadsheet-style software package that allows designers to determine the base thickness for concrete pavements in a road configuration. This program is specially designed around the design methodology in the AUSTROADS Pavement Design Guide [39]. The output of the program is the cumulative fatigue and erosion distress represented as a percentage of allowable axle load repetitions.

The program has the same pavement configurations to those noted in the AUSTROADS Pavement Design Guide, ie bases with or without shoulders, plain or continuously reinforced concrete, and transverse joints with or without dowels The program allows for any axle spectrum that has been derived from site data, and includes twin-steer axles.

During and after the input procedure, the program has many features to facilitate variations of the site information or the design parameters. An iterative process, with user inputs, is required for the determination of an appropriate design base thickness. The feature of this program is the ability to make changes to the design and get immediate results, and thus, increasing the productivity of the pavement engineer.

The output to the program consists of:

On-screen results of the cumulative fatigue and erosion distress as a percentage of allowable axle load repetitions. The designer can verify from the output that both estimates of the distress level are below 100%.

A hard copy option presented as a report in one or more of the following formats: Optional Summary; the listing of CULWAY data; the calculations.

7.7 FEAR

FEAR (Finite Element Analysis of Rafts)[40] performs an analysis of rafts constructed on layer soil properties of finite depth. The soil is treated as an elastic continuum- an advantage over the use of use of spring models which do not allow the interaction between one spring and another. The interface between the raft and the soil is assumed to be smooth. The soil can be treated as consisting of horizontal layers of different elastic properties.

Rafts which have a shape in plan that can be made up of rectangular (and which are loaded by either uniform loads or point loads or moments) can be analysised. Bending moments rotations, twists and displacements can be calculated in the raft. The contact stress distribution between the raft and the soil may also be calculated.

Results from the FEAR analysis can be processed by the plotting program FEARP. This allows contour plots of moments and displacements in the raft to be made. Contour plots can be made as either a plan view of the raft or an isometric view of the raft. Isometric views of the deformed raft can be view from various angles.

8 cracks and surface defects

Chapter 8
Cracks and Surface Defects

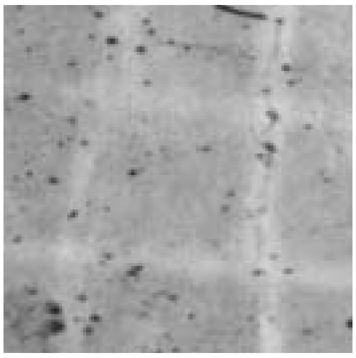
8.1 INTRODUCTION

Although it may at first appear negative, having an understanding of what can go wrong can greatly increase the chance of a successful outcome. With an understanding of the potential problems it is possible to develop appropriate contingency plans. Fortunately most concrete industrial floors are constructed which are fit for purpose and meet the client's expectations. However, when things go wrong with concrete, the remedial solutions are often costly. This chapter provides a brief summary of some cracks and surface defects that may diminish the quality of the end product.

8.2 CRACKS

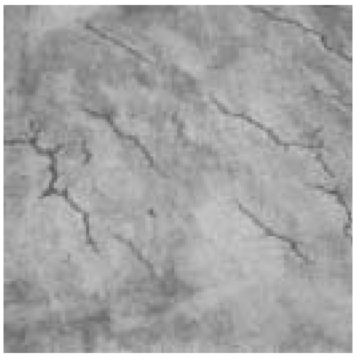
The relatively low tensile strength of concrete means it is prone to cracking. Often cracks are wrongly diagnosed as drying shrinkage, because with time, drying shrinkage tends to open up any pre-existing crack. Probably the most common forms of unwanted cracking in New Zealand are associated with early age cracking due to plastic shrinkage or restrained early thermal contraction. The following is a summary of different types of cracks, when they are likely to appear, the cause, and measures which can be taken to prevent them.

8.2.1 Plastic Settlement



This type of cracking normally occurs over the top of reinforcement, or at changes in depth, such as in waffle slabs. It is more common in deep sections, but can occur in slabs. The cracks are caused by differential settlement of the fresh concrete. Settlement can occur in fresh concrete as the aggregates sink and water comes to the top surface. This water is referred to as the bleed water. Volume changes in the fresh concrete can also be associated with the early hydration and absorption of water by the dry cement and aggregates. Concrete with high bleed volumes, or which is subjected to poor vibrating practice, are the most common causes of this type of cracking. Reduction in bleed capacity, by modification to the mix design, or revibration are preventative measure. These cracks appear early on in the life of the concrete, usually from 10 to 180 minutes from placing.

8.2.2 Plastic Shrinkage



This form of cracking derives its name from the fact that it occurs while the concrete is still plastic. Once concrete is in place, evaporation can only occur from the free surface. In the absence of appropriate precautions and unfavourable drying conditions, the rate of evaporation at the surface can be greater than the rate with which water within the concrete can migrate to the surface to make good the loss. In these situations the surface dries and shrinks, leading to surface cracks.

If these cracks are noted early it may be possible to re-compact the concrete to remove them. However, if they are noticed after the concrete has hardened, re-compaction will not be possible. The photo shows a typical example of plastic cracking. Typical characteristics are:

- The cracks occur either while finishing or within 30 minutes to 6 hours of finishing,

- The cracks are often not straight and have a jagged appearance.
- Cracks may intersect each other forming T junctions or acute angles.
- They are surface related but may extend deeper with subsequent drying of the slab.

When the estimated evaporation rate exceeds 1 litre/m^2 per hour, precautions need to be taken to prevent plastic cracking. If water reducing admixtures are used, or the concrete has a low bleed volume, it is recommended that the precautionary bleed rate be reduced to 0.5 litres/m^2 per hour. Graphs for estimating the evaporation rate are provided in the *NZ Guide to Concrete Construction*[1].

Precautions to prevent plastic cracking are:

- The use of proprietary anti-evaporatant alcohols. It is important to note that these products are not curing agents, and will need to be reapplied if the surface is disturbed.
- Water misting; although this can be difficult to achieve in windy conditions,
- Polypropylene fibres; these are typically added at the batching plant and therefore their use requires planning.

8.2.3 Restrained Early Thermal Contraction.

A large drop in the ambient temperature on the night following the pour can lead to cracking caused by restrained thermal contraction. The tensile strength of the concrete is often at its lowest 6 to 18 hours after pouring and is therefore very vulnerable to cracking caused by tensile stresses induced by restrained thermal contraction. These cracks may be hardly visible at first but often open up as the slab dries and shrinks. They often have a similar appearance to drying shrinkage cracks. Slabs that have random cracks despite the presence of saw cuts at regular intervals have often suffered from restrained early thermal contraction. The probability of these cracks occurring can be reduced by:

- The use of early entry saws. With these saws the slab is cut within 2 hours of finishing
- Using joints which are formed in the fresh concrete
- The use of crack inducers cast into the slab
- Insulation of the slab

The cracks will normally appear 12-24 hours after pouring but maybe difficult to see until drying shrinkage opens them up.

8.2.4 Drying Shrinkage

These cracks are caused by the volume changes associated with the loss of the chemically unbound mix water within the concrete. These volume changes occur in the cement phase, and sometimes the aggregates used in the concrete. If the slab is restrained or develops sufficient frictional forces on the base, the shrinkage will induce tensile strains within the concrete, which can lead to the formation of cracks.

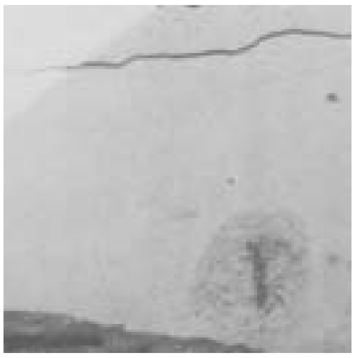
It is not possible to prevent drying shrinkage. The most common design strategy is to provide joints at regular intervals to ensure that the location of shrinkage movement is constrained to defined locations. Alternatively, post-tensioning or expansive cements (with appropriate reinforcement) may be utilised to ensure that shrinkage does not induce tensile stresses in the floor slab.

Long term drying shrinkage usually appears several weeks or months after casting the element.

8.2.5 Construction Movement

This occurs where there is inadequate temporary support of the falsework or formwork, or can sometimes occur in slabs when there is movement of the slip membrane.

8.2.6 Corrosion-induced cracking

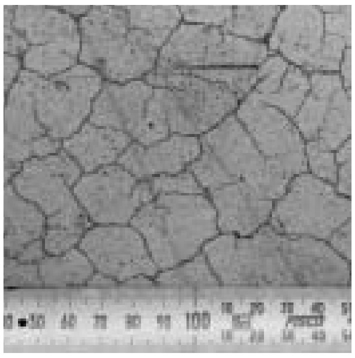


Corrosion-induced cracks occur in structures where the steel reinforcing is exposed to a corrosive environment. These cracks are caused by the expansion of the reinforcing steel as it corrodes and are sometimes associated with rust stains, particularly when the cause is chloride attack.

These cracks are formed directly over the reinforcing steel. The time to first appearance depends on the quality of the concrete construction and how aggressive the environment is.

8.3 SURFACE DEFECTS

8.3.1 Crazed cracking



Crazed cracking consists of fine cracks which appear on the concrete surface as irregular hexagonal shapes. The depth of cracking is 2-3mm at the most. If the depth is greater than this the cracking is not strictly crazed cracking.

The underlying cause is where surface tensile stresses develop due to differential shrinkage of the surface relative to the body of the concrete. This differential shrinkage may be caused by:

- Over-trowelling which can create a thin layer of cement rich paste on the surface, which will shrink more than the concrete below
- Working the bleed water into the surface. If trowelling commences too early then bleed water can be mixed into the surface, creating a surface with a higher water content
- Using cement to mop up the bleed water
- Incorrect use of specialist surface dry shakes
- Intermittent curing allowing the surface to get wet and dry
- Smooth low permeability formwork

It is generally accepted that crazing is a cosmetic problem. There is much anecdotal evidence of industrial floor slabs that exhibit crazed surface cracking that have been in service for many years without deterioration.

Deterioration problems can occur when the cause of the cracking was working the bleed water into the surface. This can result in a surface that dusts and does not have the desired abrasion strength. If the cracking was caused by inappropriate use of a dry shake, the surface may, in extreme cases delaminate. In situations where freeze thaw is a common occurrence, the depth and width of the cracks should be considered. In most instances the cracks are thin and shallow and therefore unaffected by freeze thaw cycles.

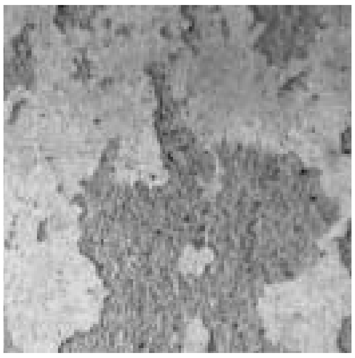
Crazed cracking is difficult to avoid as in many instances the desire to get a good hard surface by steel trowelling can result in a crazed surface. However, cracking can be reduced by:

- Avoiding the use of very wet mixes
- Do not trowel until the bleed water has gone
- Continuously cure
- Do not use 'driers'
- Avoid overworking the surface

8.3.2 Dusting

The strength of concrete is primarily dependent upon the water to cement ratio. The higher this ratio, i.e. greater amount of water, the lower the concrete strength. If additional water is introduced to the concrete surface during finishing, a thin layer of weak paste will be created on the surface of the concrete. Water could be introduced by rain, poor finishing work practises, or commencing finishing before all the bleed water has evaporated. This weak surface is easily abraded, and when subjected to wear, dusts. Eventually the surface can wear away to reveal the larger aggregates below the top surface of the slab.

8.3.3 Delamination



Some research on delamination attribute it to bleed water and/or air entrapped below a hard dense steel trowelled surface. If the trowelling operation is started too early, thereby sealing the surface, rising bleed water can effectively lift the top 1-3mm off the surface causing it to delaminate from the slab. It has also been hypothesised that top down setting is significant.

Top down setting can lead to the surface being finished while the concrete below is still fresh, resulting in bleed water being trapped below the surface. This can lead to surface delamination. Top down setting is more probable when:

- It is windy, therefore drying the surface
- The sub-grade is cold
- Concrete mixes with low bleed water volumes are used
- Air temperatures are rising
- Low relative humidity
- A vapour barrier is placed below the slab

Another probable cause of surface delamination occurs where the placer finds that the surface is hardening faster than the available trowelling resources can finish the surface. In these situations the surface cement paste may be spread over an already hardened surface, and water may be applied. This thin layer often craze cracks and then delaminates.

Other potential causes are frost damage during the early stages of curing.

8.3.4 Surface Deposits

The most common types of surface deposits are efflorescence and exudation.

Efflorescence is a deposit of salts, usually white, which results from the precipitation of calcium carbonate on the surface. Water movement through concrete, either from the natural drying process or from external water pressure, can transport the abundant calcium hydroxide in the concrete to the concrete surface where it reacts with carbon dioxide in the air, forming the white deposit.

Efflorescence is an aesthetic problem; the deposit can be removed by a light acid wash. However, if the source of moisture movement through the concrete is not corrected, efflorescence will return.

Exudation is a solid or gel-like component of material discharged at an opening in the concrete surface

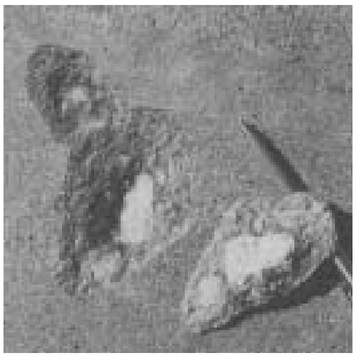
8.3.5 Discolouration

Discolouration of the surface of the concrete can be brought about by many factors. Changing cement types and concrete suppliers, variable finishing techniques, uncontrolled use of admixtures, variable water-cement ratios and curing, all have an effect on the uniformity of the end product. Some aggregates which are high in iron content can cause rust staining of the surface of the concrete.

If plastic sheets are used for curing, wrinkling can result in a mottled surface appearance.

Variable sub-grade conditions can cause surface discolouration when a vapour membrane is not used.

8.3.6 Pop outs



Pop outs are holes in the surface of the concrete which have resulted from the breaking away of the concrete surface due to internal pressure. These are most frequently associated with frost susceptible aggregates or organic contaminants such as lupin seeds in the concrete.

9 concrete testing

Chapter 9
Concrete Testing

9.1 INTRODUCTION

This chapter describes methods used in New Zealand to sample and test concrete in the construction of floors. It is not a complete list of all test methods available. Testing of materials used to make concrete is covered in The NZ Guide to Concrete Construction[1].

9.2 TESTING PRINCIPLES

When designing a test programme it is important to select a property or properties that will give the information required. Compressive strength, for example, will not always be the best indicator of a floor's performance. It is also important to know how the results will be interpreted. Will they be compared against a specified value, or will they be comparative – in which case a control sample will also need to be tested. A concrete technologist will be able to advise on the most appropriate techniques for solving a given problem.

It is rarely practical to design tests that replicate the conditions that will be encountered on a particular site. Instead, most tests are carried out following standard procedures and in controlled environments. As a result, standard concrete tests are often criticized for bearing little relationship to what happens to concrete on site: test specimens are cast and cured differently from site concrete, and tests are carried out at prescribed ages by which time it may be too late to rectify shortcomings easily or too early to accurately represent the ultimate performance. Controlled, artificial conditions are used because the properties of concrete vary with age and curing condition. The controlled conditions allow concretes cast at different times to be compared to each other, and to specified requirements.

9.3 PERSONNEL

Sampling, manufacture of test specimens and testing must always be carried out by qualified personnel who understand the principles behind the test. This will ensure that correct procedures are used wherever possible, i.e. that standardised methods are followed, and if deviations from the standard are made then their effects are understood.

Accreditation by International Accreditation New Zealand (IANZ) indicates that the laboratory's facilities meet the specified requirements, and that its staff have a good understanding of testing principles. However IANZ accreditation is test-specific, and laboratories will usually only be IANZ accredited for tests that they perform regularly. In lieu of IANZ accreditation for a particular test, compliance of the laboratory's quality systems with the requirements of NZS/ISO/IEC17025:1999 (and therefore with ISO 9001 and 9002) as assessed by an independent accreditation body indicates that the laboratory is competent.

9.4 SAMPLING

A "sample" of concrete is a portion of concrete removed from the mix to measure the properties of a particular batch or mix design. A "specimen" is the individual piece of concrete on which the test is carried out, e.g. a cylinder.

Test results must represent the properties of the concrete being tested. Hence the test sample must represent the concrete from which it is taken. NZS 3112: Part 1:1986 sets out procedures for obtaining samples from freshly mixed concrete. Guidance for taking core samples of hardened concrete is given in NZS 3112: Part 2:1986.

NZS 3109:1997 imposes a number of requirements on the sampling of concrete for acceptance testing during construction. When concrete is supplied by a ready mix plant, sampling and testing is usually done at the plant prior to delivery. This reduces the site testing to carrying out a workability test at time of delivery. However, there may be good reasons why strength samples should be taken at the site. For example concrete air contents have been known to increase between the concrete plant and discharge at site.

The sampling location is determined by the information sought from the testing. Three options for sampling and testing for a project are set out in NZS 3109:1997:

Full acceptance of plant testing records (to ascertain consistency of concrete supply),

Use of full sampling from site (to ascertain consistency of concrete delivery),

A partial system of plant records and some specific site testing.

To ensure that samples are representative of the concrete being delivered to the site, they must be collected at random. Selection should never be made on what the concrete looks like as it is being discharged. Samples must be collected by appropriate methods, such as those described in NZS 3112: Part 1 1986, and NZS 3109:1997.

The number of test specimens to be made from a sample will be prescribed by the test method to be used.

9.5 TESTS ON FRESH CONCRETE

9.5.1 WORKABILITY

"Workability" describes how easily fresh concrete can be mixed, placed, consolidated and finished. The ability of the mix to flow is one of the most readily observed properties, and is sometimes referred to as its consistence or consistency. It is most often measured by the slump test. "Consistency" is also used to describe uniformity of supply, i.e. that the quality of successive batches is consistent, which can be confusing because flow properties are most often used to monitor uniformity. In this publication "consistency" is used to describe uniformity, and "workability" and "slump" are used to describe the physical characteristics of the concrete.

Workability characteristics include the ease with which the concrete can be put in motion and kept in motion (its rheological properties), whether it remains cohesive while moving or segregates (fluid concrete) or falls apart (stiff concrete), and whether it holds its shape when unsupported. Changes in concrete mix design and materials may be reflected in some of these characteristics and not in others. Many tests have been devised to measure these properties, but no test measures all parameters and some tests are better suited to certain types of concrete than others. Results from different tests are not always comparable and it is not possible to use results from one test to establish compliance with a specification based on another test.

Workability tests are used to design concrete for a particular application and site conditions. A particular workability is specified to ensure that the concrete can be placed without blocking delivery equipment or segregating, compacted to remove entrapped air and fully encase reinforcement, prestressing and other insertions and finished to the required flatness.

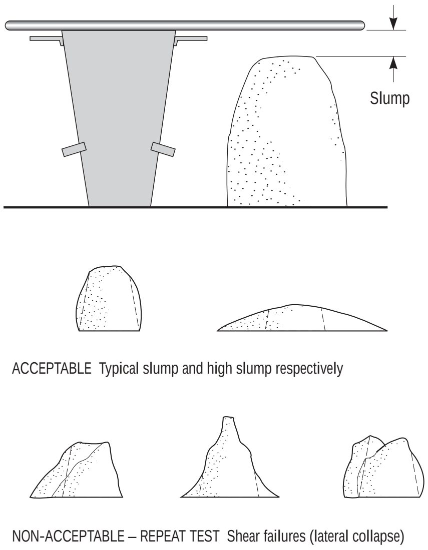
Workability must be measured within a limited time because concrete stiffens as it hydrates, even within the first minutes after mixing. NZS 3112: Part 1:1986 requires that workability tests be completed within 15 minutes of taking the sample.

The **slump** test is quick and simple, the equipment is relatively inexpensive, and the test can be carried out in the laboratory or on site. It is the most widely used workability test, and is often used to monitor uniformity of supply.

The slump test is described in NZS 3112:Part 1:1986 Section 5. The test is carried out by filling a conical mould with concrete and then withdrawing the mould. The amount by which the concrete subsides or 'slumps' is then measured Figure 9.1. After the slump has been measured, the concrete is tapped gently on the side to obtain an indication of the cohesion of the mix.

Figure 9.1 The slump test

Figure 9.2 Slump test results



Mixes which are well proportioned and cohesive tend to subside a little further. Poorly proportioned, harsh mixes tend to fall apart. The slump test must be performed on a flat, level surface free from vibration or the movement of the concrete will be affected.

The test does not work well for concretes with either very high or very low workabilities. Very stiff concretes may not subside at all. Very fluid concretes may simply lose their shape completely by subsiding and flowing. Highly thixotropic mixes, such as those containing mineral admixtures, may in practice be much more "workable" than their slump values indicate. The slump of superplasticised concretes may continue for an extended period after the mould is withdrawn.

Some mixes may lack sufficient cohesion for the test to be carried out properly. The cone of concrete may shear or otherwise collapse as the mould is withdrawn. If this occurs, the test must be repeated with another part of the sample. If the concrete again shears or collapses, the slump is not measured but a shear or lateral collapse is recorded (Figure 9.2). Such concretes may be difficult to handle and place satisfactorily.

The workability of very fluid mixes may be better measured by the **spread** test (NZS 3112: Part 1:1986 Section 11). This test was designed to measure the flow of concrete used to fill the cavities in concrete masonry construction. An inverted slump cone is filled then lifted off its baseplate, and the diameter of the resulting disc of concrete measured. A modified spread test, which measures the rate of spread as well as the ultimate spread has been used to measure the flow of self-compacting concrete. A similar test, measuring the rate and volume of flow through an L-shaped form has also been used for these materials. Neither of these two methods is yet standardized.

Figure 9.3
The spread test

The rheological properties of concrete may be measured with a **workability apparatus**. This method measures the forces required to start concrete moving (the "yield") and keep it moving (the "plastic viscosity"). The apparatus is basically a viscometer designed for concrete. An impeller rotates at a range of speeds in a bowl containing the sample and the torque exerted on the impeller by the concrete each rotation speed is measured. The relationship between the torque and the impeller speed is a straight line, of which the gradient reflects the plastic viscosity and the intercept reflects the yield. The equipment is highly specialized and not portable so the technique is most appropriate for laboratory development of materials and mix designs.

Figure 9.4
The Vebe test

The **Vebe consistometer** test is described in NZS 3112: Part 1:1986 Section 7. The test is carried out by determining the time taken for a cone of concrete to completely subside inside a mould when subjected to vibration. The method works well for concrete with very low workability. Whilst the test is sensitive to changes in materials, early stiffening of concrete and other factors that affect its workability, it is not easy to carry out with consistent results, and requires a power supply and specialist equipment. Its application in the field is therefore limited but it has been used in the laboratory in the past to investigate materials and their impact on workability. It is no longer widely used because its limitations as a site test mean it must be done in addition to the slump test, and it is usually difficult to justify the cost of two different measurements of the same property.

The **remoulding effort** test (NZS 3112:Part 1:1986 Section 6) measures the amount of effort required to mould a sample to a particular shape using a drop table. It is suitable for concrete of average workability. It is no longer widely used for the same reasons as the Vebe consistometer test.

9.5.2 Air Content of Plastic Concrete

The air content of plastic concrete should be measured as a quality control procedure to determine that a desired level of entrained air has not been exceeded, or that a specified air content has been achieved when an air entraining admixture is used to improve workability or durability.

The measurement of air content is described by NZS 3112: Part 1: 1986 Section 9. This method is based on determining the reduction in volume of a given unit volume of concrete when subjected to an increase in air pressure. From this figure and the pressure applied, the actual air content can be calculated. However, the equipment must be calibrated for the height above sea level at which it is being used. There are two types of air meter available to measure air content by the pressure method. Type A measures the displacement of the level of water in a column above the concrete surface when pressure is applied over the water. Type B measures the pressure drop in a pressurised chamber when the air in the chamber is equalised with the air in the concrete. The latter type is the most commonly-used in New Zealand.

Air in the aggregate pores is included in the air content measured by the pressure method. This air does not have the same effects on the concrete as the air in the paste fraction, so its volume is not included in specified air contents and must be subtracted from the total air measured by the pressure method. NZS 3112: Part 1:1986 allows for a correction factor to be applied to the concrete air content. The correction factor is determined by measuring the "air content" of a specified volume of aggregate with the air meter.

AS 1012.4.3-1999 describes a volumetric method of measuring the entrained air which is suitable for porous aggregate. The volumetric method entails the displacement of the entrained air with water.

9.5.3 Mass per Unit Volume

The mass per unit volume, or density, of freshly mixed concrete is used to determine the volume of concrete produced from a given mass of materials. Thus, it can be used to determine the volume of concrete delivered by weighing trucks prior to and after leaving the site.

Mass per unit volume is determined by a simple test in which the mass of concrete in a container of known volume is measured. The standard procedure for conducting this test is described in NZS 3112: Part 1:1986 Section 4.

9.5.4 Bleeding of Concrete

Tests for the bleeding characteristics of concrete are normally carried out in the laboratory to evaluate mix designs or the influence of different materials.

The procedure is described in NZS 3112: Part 1:1986 Section 8. A sample of the concrete to be tested is compacted into a cylindrical container. The container is then covered and placed on a level surface. Bleed water is drawn off with a pipette at regular intervals until the amount collected during a 30-minute period is less than 5 ml.

The results may be expressed either as the volume of bleed water collected in a given time per unit surface area of the cylinder, or the rate of bleeding.

Setting time of concrete

As for the bleeding characteristics, setting times are sometimes measured to evaluate mix designs or materials. It may be desirable to modify setting time if the concrete is to be placed in particularly warm or cool conditions, or if there is an extended transit time from the batching plant to the site.

Setting time is measured by the procedure described in NZS 3112:Part 1:1986 section 10. A sample of mortar is sieved from the concrete, and compacted into a container. The penetration resistance is measured at appropriate intervals. Initial and final set are defined by the times taken to achieve prescribed values of penetration resistance.

9.6 TESTS ON HARDENED CONCRETE

9.6.1 General

A variety of tests may be carried out on hardened concrete to characterise its properties or to measure its performance under service conditions.

There is usually more than one method of measuring any given property. Selection of an appropriate method depends on the purpose of the testing and whether the test is to be carried out on existing concrete or on specially cast specimens. Unless the testing is to establish whether the concrete meets a specified requirement, in which case the test method will be prescribed, options should be discussed with the testing laboratory before sampling is arranged.

The standard test methods named in the following sections are not the only standard methods available for measuring these properties. Australian, British, AASHTO and ANSI standards offer English-language alternatives, often the same or similar to the New Zealand and ASTM standards quoted.

9.6.2 Strength

Compressive strength is the property most often specified, and therefore is the property most often measured in quality control.

NZS 3112: Part 2:1986 Sections 3, 4 and 5 describes the procedures for moulding and curing specimens made from plastic concrete. NZS 3112: Part 2: 1986 Section 9 describes the procedures for obtaining cores from hardened concrete.

The size of the test specimen is determined by the nominal maximum aggregate size in the concrete. The diameter of a cast cylinder must be at least four times the nominal maximum aggregate size, and not less than 100mm. The nominal length of the cylinder is twice its diameter. Figure 9.6 shows typical moulds for casting compression test specimens. The most common specimen size is 200 x 100 mm.

NZS 3112: Part 2: 1986 requires that core diameters meet the same requirements as cylinders, although if the maximum aggregate size is unknown the core must be at least 100mm diameter. It

Figure 9.5
Compression testing

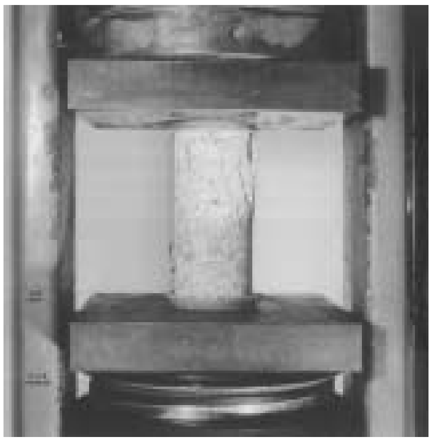


Figure 9.6
Moulds for the two sizes of concrete test cylinders

Figure 9.7
Restrained rubber capping system

also requires that wherever possible cores should be cut to a length to diameter ratio greater than 2. This would not be possible for many floor slabs. ASTM C42/C42M-99 takes a more practical approach providing correction factors for length to diameter ratios between 2 and 1, and allowing core diameters less than 95mm where it would otherwise be impossible to obtain cores with a length to diameter ratio greater than 1. Smaller cores will give more variable results. Results from two specimen sizes should not be combined in determining the average strength.

Compressive strength tests are highly sensitive to the geometry of the ends of the specimens. NZS 3112: Part 2:1986 Section 4 prescribes limits on planeness and perpendicularity, and describes the materials and methods used to realign the ends when necessary. Figure 9.7 shows a restrained rubber capping system.

The measurement of the compressive strength of concrete is described in NZS 3112:1986 Part 2 Section 6. It includes requirements for testing machines and testing procedures to ensure that test results from a single batch of concrete are as uniform as possible.

Strength tests are based on a set of at least two specimens tested at the same time. Strengths obtained from cores will not be the same as those obtained from cylinders of the same concrete. The relationship between core strengths and specified 28 day compressive strength depends on the age, curing history and compaction of the concrete. Guidance on the statistical analysis of strength test results is given in NZS 3112: Part 2:1986 Section 10. More detailed information on the analysis and interpretation of results is given in other references such as NZS 3109, and the ACI Concrete Manual.

Tensile strength determines the crack resistance of the concrete material – when the tensile strength is exceeded the concrete will crack. Tensile strength tests can be used to compare the likely performance and floor design requirements for crack resistance of different concrete mix designs and materials.

Tensile strength of concrete may be measured by splitting a cylindrical specimen along its length. The method is described by NZS 3112:Part 2: 1986 Section 8. The specimen is held horizontally in a jig between the platens of a testing machine and a compressive force applied until the cylinder splits, see Figure 9.8, overleaf.

This method avoids the problems of gripping and aligning the specimen accurately when testing in direct tension, ie pulling the ends of an elongated specimen apart.

Figure 9.8
The Brazil or splitting test

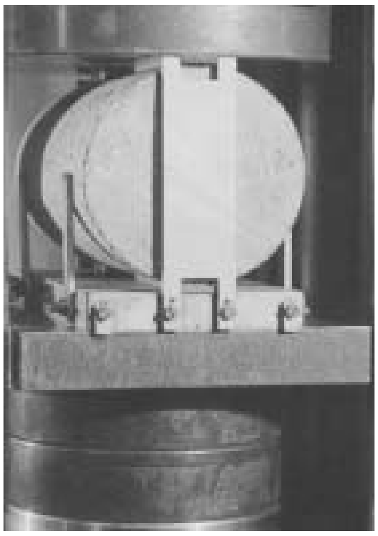
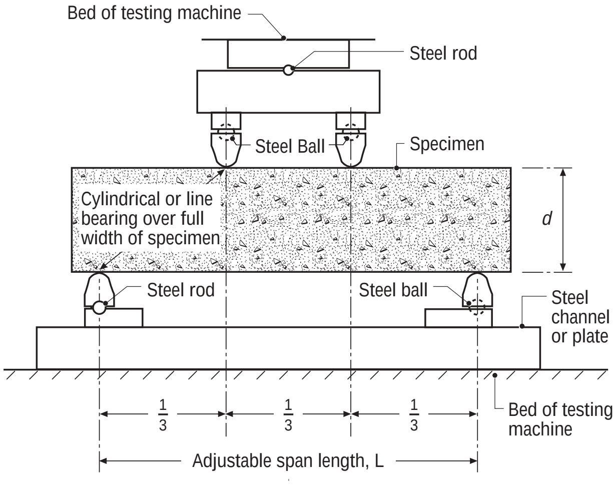


Figure 9.9
Flexural test by third point loading method



Flexural strength indicates the bending resistance of concrete, and can be used in design procedures to ensure that the floor or pavement will withstand the loads to be carried by the floor in service.

The test specimens and method are described in NZS 3112: Part 2:1986 Section 7. A beam specimen is supported near each end, and loaded in compression in a test machine as shown in Figure 9.9. This test configuration is known as third-point loading because the distance between each of the two end supports is divided into equal thirds by the upper load application points. Results are expressed as the modulus of rupture.

Although both this method and the tensile splitting test reflect the tensile properties of concrete their results are not directly related.

The **break off** test (ASTM C1150) was designed to establish when concrete in situ has attained a specified strength so that successive operations may be applied during construction, for example release of prestressing strands. The test involves isolating a cylindrical specimen at the concrete surface by casting an annular ring into the surface of fresh concrete. The specimen is then broken off by applying a load to the side of the specimen. The Scancem Break-off Tester is a proprietary device designed for this technique. A correlation needs to be established between the break-off number and strength of any given concrete. In practice, cylinder or core strengths are more often used. The test leaves a hole in the concrete surface.

The pull-off test is an indicator of tensile strength. It involves isolating a cylindrical specimen in the surface of hardened concrete with a core drill. The specimen is then pulled off in direct tension. The test can be used for similar purposes as the break-off test if the core is cut deep enough to represent the bulk of the concrete. If a short core is cut to represent the surface of the concrete the test can be used to indicate whether the concrete is strong enough for a coating or topping to be applied (the tensile strength of the concrete must exceed the stresses imposed by the drying of the surface treatment otherwise it will delaminate). The test leaves a hole in the concrete surface.

Pull-out tests are similar to pull-off tests in that the tensile strength of the concrete is measured by removing an insert in direct tension. ASTM C900 describes the equipment and method used to measure the pull-out strength using an insert cast into new concrete.

The **Lok test**, shown in Figure 9.10, is a proprietary system of this test. Similar tests (eg the proprietary **Capo test**) have been devised to measure the pull-out strength using an insert fastened into concrete after it has hardened , but they have a common problem of high variability. Again, all these methods leave a hole in the concrete surface.

Figure 9.10
The Lok test

Surface hardness is often used to give an indication of compressive strength. The most common technique is the measurement of **rebound number** using a **Schmidt Hammer** (ASTM C805). This involves measuring the height of rebound of a spring-driven mass after its impact on a concrete surface. Results are broadly related to compressive strength but are highly sensitive to surface properties. The technique is therefore best used for investigating the uniformity of a concrete surface, or changes in the nature of the surface with time rather than testing for a compliance with a specified strength, unless a correlation is available for the concrete in question. The test does not damage the concrete.

Penetration resistance can be used as an estimate of compressive strength. This approach involves firing a probe or pin into a hardened concrete surface with a known amount of energy and measuring the depth of penetration. The procedure is described by ASTM C803. The **Windsor Probe** is a proprietary device for measuring probe penetration. The small size of the penetrating devices means the result is affected by the nature of the coarse aggregate.

9.6.3 Density

The density of hardened concrete specimens is often measured in conjunction with compressive strength tests, either on cast cylinders or core samples, as an indication of concrete quality. The method is described by NZS 3112: part 3: 1986. It is based on displacement of the bulk volume of the specimen with an equal weight of water so is inappropriate for measuring the density of excessively honeycombed concrete or of no-fines concrete.

9.6.4 Dimensional changes

Limits on the **drying shrinkage** of concrete are sometimes specified by designers of floors and pavements to limit joint movement. Drying shrinkage may be measured as part of the process of mix design and material selection, or to show compliance with a specified requirement.

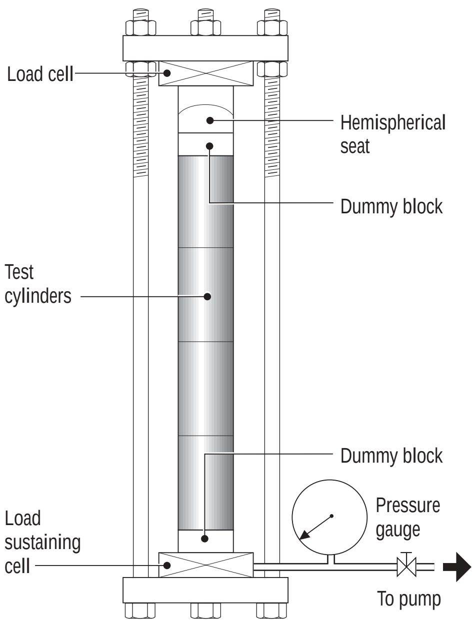
Figure 9.11
Shrinkage test measurement

The most widely used method for measuring drying shrinkage is set out in AS 1012.13 1992. This test measures length change after 56 days drying. NZS 3112: Part 3:1986 Section 3 also specifies a drying shrinkage test. The Australian test is preferred because the drying conditions more closely resemble the New Zealand natural environment than do those used in the New Zealand method. Both methods involve precision measurements of the length change of cast beam specimens following an initial seven-day wet-curing period. Consequently they do not measure length changes that occur soon after casting, such as those due to the addition of shrinkage compensating admixtures, which cause an initial expansion, or autogenous shrinkage, which may occur when the water to cement ratio is low.

It can be difficult to accurately predict the shrinkage of the concrete in situ from shrinkage test results. The impact of surface area to volume of the cast concrete, internal and external movement restraints, and the difference in curing and subsequent environmental conditions need to be considered. Nevertheless, laboratory shrinkage tests do allow the material performance of different mix design and materials to be compared. See also 12.2 in Part 1[2] of this publication.

Creep is the long-term deformation of concrete under load. Knowledge of creep characteristics is important for prestressed elements because creep will reduce the prestress over time.

Figure 9.12
A typical arrangement for testing the creep of concrete specimens



Methods for measuring the creep of concrete specimens under a uniaxial load are described by AS 1012.16 1996 and ASTM C512. Both involve precision measurements of length change of cast specimens that are held under a given load, usually around 40% of the compressive strength, for an extended period, (the ASTM method states one year). The set up is shown in Figure 9.12. The concrete will shrink during this time if allowed to dry, so shrinkage is measured on identical unloaded specimens stored close to the test specimens and shrinkage subtracted from the total length change measured on the loaded specimens. Alternatively, the specimens may be sealed with a water-vapour impermeable membrane to simulate concrete that would not be expected to dry in service. Creep itself is sensitive to environmental conditions, so the tests need to be carried out in a controlled environment. The exception is when comparative testing is performed simultaneously on different concretes.

Elastic modulus and **Poisson's ratio** reflect the short-term dimension changes of concrete under load. Methods for measuring static modulus of elasticity and Poisson's ratio are given in ASTM C469. They involve precision measurements of deformation of concrete specimens as load is applied at a given rate up to 40% of compressive strength in a test machine. ASTM C215 describes methods of measuring dynamic elastic modulus and Poisson's ratio by determining resonant frequencies obtained when a specimen is exposed to vibration or impact.

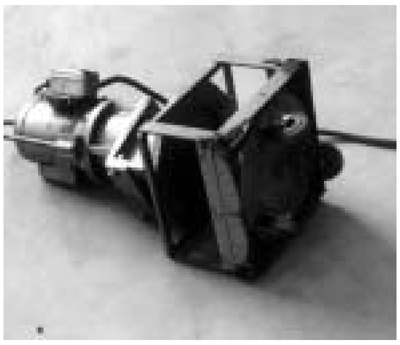
9.6.5 Abrasion resistance

Abrasion is defined as the wear of a surface due to rubbing and friction. It is distinct from erosion, which is usually used to describe wear by caused by moving fluids. Erosion is not considered here.

Abrasion resistance is one of the critical factors that determine the long-term performance of a concrete floor surface. It can be measured to help determine materials, mix design and construction processes to produce a desired level of performance, or to compare actual performance achieved against that which was specified. Abrasion resistance is very sensitive to the surface properties of the concrete, which will be greatly affected by the techniques used to finish and cure the concrete. It can be difficult to recreate on-site practices when making laboratory specimens, so to enable comparison of different mixes the approach is usually to test a surface prepared in a defined way, e.g. cast against a certain type of mould using a particular release agent.

Abrasion may be caused by many different mechanisms and as a result many different tests have been designed to simulate the different abrasive actions. Tests currently available in New Zealand are described below. All involve exposing the test surface to an abrasive action and measuring the volume or weight of material lost after a given period of exposure. These tests are not suitable for

Figure 9.13
Accelerated abrasion test equipment



measuring the abrasion resistance of coating systems.

AS/NZS 4456.9: 1997, developed from the **Sydney City Council test** and also known as the **tumbler test**, describes a method for measuring the abrasion resistance of clay or concrete segmental pavers. It can equally well be used to measure the abrasion resistance of specimens of concrete cast or cut to the appropriate size. The test simulates impact and scuffing actions of steel machinery, and also relates well to the high point load impact and grinding effects of high heel shoes on pavements. It involves exposing the surface of the specimens to the impact of free-falling steel balls. One advantage of the test is that a large number of replicate specimens are tested simultaneously, thus allowing for the variable nature of the surface. Another advantage is that guidance to the significance of the results has been published by various agencies, so the results can be related to in-situ performance.

ASTM C779 (procedure B) and ASTM C 944 describe methods that expose the concrete surface to rotating steel dressing wheels applied under a vertical load. They are designed to simulate rolling, pounding and cutting actions, e.g. of steel wheels. The two methods use different apparatus, C779 being suitable for in-situ testing or testing large laboratory samples, and C944 being designed to test the ends of core samples or smaller laboratory specimens. The tests provide good comparative data between different concretes but no in-situ performance criteria have been published.

The rolling steel wheel test is designed for measuring the abrasion resistance of a concrete floor in-situ and simulates the cutting, sliding and rolling actions of steel wheels on an industrial floor. It is less aggressive than the tumbler test. The UK Cement and Concrete Association developed the method.

9.6.6 Chemical resistance

Floors may be exposed to aggressive chemicals, particularly in manufacturing plants. In some cases Portland cement based concrete will not be sufficiently durable and a protective coating will be required. Tests for the chemical resistance of coatings are not discussed here. In other situations, careful selection of materials and mix designs may provide sufficiently durable concrete. Comparison of different mix designs during the design stage to achieve optimum performance is the most common reason for testing for chemical resistance. Where procedures have been standardized, such as sulphate resistance tests (ASTM C1012, AS 2350.14) or resistance to scaling caused by deicing chemicals (ASTM C672), previous experience may allow a minimum performance to be specified so that a single material may need to be tested for compliance.

Chemical resistance tests must mirror anticipated in-situ exposure conditions as closely as possible because chemical resistance can change with slight changes in chemistry, conditions or degree of exposure. Chemical resistance often depends on the cation associated with the aggressive anion – magnesium sulphate is more aggressive than sodium sulphate for example. Resistance will also depend on whether reaction products are removed, and whether the concrete is exposed to wetting and drying or temperature fluctuations. Perhaps because of this sensitivity there are few standardized tests for chemical resistance for concrete.

ASTM C267 is designed for testing the chemical resistance of mortars, grouts, surfacings and polymer concretes, and a similar approach can be taken for concrete. Cylindrical specimens are immersed in a test medium consisting of the agents that the material will be exposed to in service, and inspected at regular intervals. At each inspection (or more frequently if necessary) the test medium is renewed and changes in its appearance noted, the specimens are cleaned and weighed and examined for signs of deterioration, and the compressive strength of a set of specimens is measured. When the chemical reaction results in expansion, such as attack by sulphate solutions, length change is measured. Testing agencies may have established in house procedures for particular applications. This experience should be utilized to benefit from correlations made between previous test results and in-situ performance.

Accelerating chemical reactions by heating can produce effects that do not occur at normal ambient temperatures so tests should be carried out in the anticipated conditions. This means that testing takes several months, depending on the rate of deterioration observed.

Tests for surface scaling caused by **wetting and drying or freezing and thawing** in the presence of a salt solution typically involve creating a reservoir of the test solution on one surface of the specimen, exposing the specimen to alternating exposure conditions and observing changes in weight and appearance after a prescribed number of cycles. Again, the tests take months rather than days to complete. Tests that involve repeat cycling are rarely automated and are therefore labour intensive.

Even with "realistic" test conditions, it is difficult to predict the service life of a material from test results because performance depends also on the ability of the concrete surface to prevent ingress of the aggressive medium, which is greatly affected by the method of finishing and curing used on site and is hard to simulate on a small specimen made in the laboratory. If possible, test specimens should be cut from larger specimens prepared in a manner that better represents slab construction.

9.6.7 Moisture content

Before applying a coating or covering to a concrete floor it is important that the moisture content of the floor be low enough to avoid condensation of evaporating water causing adhesion failure or degradation of materials in contact with the concrete. To avoid such problems, manufacturers of coatings and adhesives often specify a maximum moisture content or water vapour transmission rate at the time of application. Evaporation and condensation of moisture depends on the temperature and humidity of the concrete and surrounding air, so measurements should be made at the conditions expected once the floor is in service. ASTM E1907 describes practices for determining whether a floor is in an acceptable condition to receive moisture sensitive coverings.

Moisture content gives an indication of the amount of water present at a given time as liquid or vapour, depending on how it is measured. A qualitative method of indicating whether there is a significant moisture content is to seal a clear plastic sheet over the surface – if condensation appears on the underside or the concrete darkens overnight then the concrete is still too wet to seal (ASTM D4263). This test is qualitative, subjective and highly sensitive to the relative temperatures of the plastic sheet, the concrete and the surrounding air but is simple and non-destructive. A quantitative, objective method is to measure the weight loss when a sample of concrete taken from the floor is artificially dried.

Electrical resistivity, impedance, capacitance and **conductivity** of concrete are also related to moisture content and can be used to monitor the drying rate of a floor in-situ. These properties vary with concrete materials and mix designs so are best used to monitor changes rather than absolute values.

Relative humidity (RH) in the concrete pores can be measured to obtain an indication of moisture content, although the RH is related to the pore structure so can vary between different concretes with the same moisture content. To measure RH, a sealed volume of air is equilibrated with the air in the concrete pores, either by sealing a container over the surface. Capacitance-based humidity gauges are often used to measure the humidity. Enough time, at least 24 hours for surface measurements, must be left for the sealed air space to equilibrate with the air in the concrete. Many manufacturers of floor coverings use RH-based application criteria.

Moisture vapour transmission rate is probably the critical factor rather than the absolute moisture content since it is the evaporation of moisture from the concrete that causes problems, not the moisture in the concrete. This can be measured by placing a dish of calcium chloride in a sealed air space on the concrete surface and monitoring its weight increase over at least 72 hours (ASTM

F1869). A maximum moisture vapour transmission rate is often specified by manufacturers of impermeable coatings/coverings.

For all methods except those that involve sampling below the concrete surface, the effects of permanent curing compounds or other surface treatments must be taken into account. It is often recommended that moisture properties be measured by two different methods, including one that measures the rate of moisture loss.

9.6.8 Flatness

Flatness requirements will depend on the use of the floor and the nature of the final surface finish.

The regularity of floors has traditionally been measured as deflections from a leveled or freestanding 10 foot (3m) straightedge. A **maximum gap between straightedge and floor** would be specified. The method has several deficiencies: it is difficult to apply to large areas, it is difficult to randomly sample a floor, it has not been standardized and has poor reproducibility, it is difficult to meet specified tolerances with 100% compliance, and it does not account for repeated irregularities. More sophisticated measuring techniques are available.

Figure 9.14
An F-meter

Figure 9.15
An F-meter in use

ASTM E1155 prescribes a method of measuring and describing the **flatness** and **levelness** of a floor by the characteristic F numbers, FF and FL. Straight lines are marked on the floor surface and point elevations measured at regular intervals along the lines. Measurements may be made by any of several different techniques. Elevation differences between all adjacent reading points are calculated and evaluated statistically to obtain estimates of FF and FL. The method was designed for randomly trafficked surfaces. Three is an approximate correlation between the flatness number and the straight edge tolerance.

ASTM E 1486 describes a method for determining the flatness, levelness and wheel path characteristics using waviness indices. This method can be used for either randomly- or defined wheel path-trafficked floor surfaces. The measurement technique is similar to that of ASTM E1155, but the results are analysed differently. It was designed primarily to measure floor surface wavelengths that most affect forklift rideablility.

When assessing the “as built” characteristics for compliance with a specification, levelness should be measured within 72 hours of concrete placement and before formwork and other support is removed to avoid the effects of shrinkage, curling and deflection after construction. Flatness would normally be measured at the same time although it is unlikely to be affected with age.

The treatment of construction joints in flatness measurements needs careful consideration. They constitute a discontinuity that may or may not be trafficked depending on location. A separate measurement and analysis of the joints may be appropriate.

9.6.9 Water permeability/absorption

The ability of concrete to take up moisture will play a major part in its ability to resist the ingress of aggressive agents. Conversely, the more permeable it is the faster it will dry after curing, and the faster the effects of any leaks in damp proofing will be noticed. Permeability/absorption properties are measured less often than other properties for floors, and more often for reinforced concrete structures subject to corrosion of reinforcement, or leakage of contents or groundwater. Moisture transfer occurs by several physical mechanisms but testing is usually broken down to “permeability” tests and “absorption” tests.

Permeability

Permeability is used to describe the resistance of concrete fluid under pressure at the concrete surface. Permeability can be measured on site or on laboratory specimens. The passage of the fluid can be monitored by flow at the downstream side of a laboratory specimen, the lowering of water level at the upstream side or penetration depth. Both site and laboratory tests suffer from the effects of localized voids, which result in misleadingly high permeabilities being measured, and from difficulties in sealing the test concrete to avoid loss of fluid in directions other than that intended. Site measurements of water permeability are affected by the moisture content of the concrete. Laboratory water permeability tests are often based on saturated flow to overcome this problem.

One standardized test for permeability is DIN 1048, which measures water penetration into a saturated sample. However there are several other proprietary/literature techniques and major testing laboratories have developed their own in-house procedures. Special procedures are needed to measure the permeability of low permeability concretes.

Absorption

Absorption is the uptake of water into dry or partly saturated concrete, principally by capillary suction. Again, it can be measured on site or in the laboratory and is affected by the moisture content of the sample at the start of the test. Absorption is measured more often than permeability because it simulates natural wetting and drying processes. Absorption reflects the properties of the pore structure which determine concrete durability so is increasingly being used as an indicator of concrete quality.

There are several standardized procedures for measuring absorption, including ASTM C642, ASTM C1151, AS 1012.21. The difference between the weight of a specimen in different moisture conditions as measured by NZS 3112: part 3:1986 Section 5 can also be used as an indication of the capacity of the concrete to take up water. These methods may be used for concrete made in the laboratory or samples taken from a site.

Major testing laboratories often use in-house methods to measure the absorption of moisture at a concrete surface with time, which is expressed as **sorptivity**. Tests are carried out in the laboratory on dried specimens. Either the weight (volume) of water absorbed or the depth of penetration can be measured. BS 1881-208, also known as the ISAT procedure, can be carried out on site.

The results from absorption and sorptivity tests are affected by the moisture content of the concrete and the procedure used to dry the specimens before (or after) testing. Permeability and absorption tests are described in more detail by the UK Concrete Society (TR31)[41].

9.6.10 Slip Resistance

Clause D1.3.3(d) of the New Zealand Building Code (1998) requires that access routes have adequate slip-resistant walking surfaces under all conditions of normal use. Compliance with this clause may be verified by confirming that the surface has a minimum coefficient of friction under the expected conditions of use, as described in Verification Method D1/VM1.

Slip resistance may be measured in the laboratory as part of the process of selecting a surface finish, flooring system or remedial treatment, or in-situ to assess the behaviour of an existing floor or to monitor the effects of wear and maintenance.

Coefficient of friction is measured by either of two methods described by AS/NZS 3661.1 1993. This standard also specifies friction requirements for surfaces.

For wet conditions the **pendulum friction tester** is used. This device has a rigid swinging arm which contacts the surface with a rubber slider. It is designed to represent the response of a pedestrian wearing suitable footwear by replicating the effect of aquaplaning when running or turning abruptly.

The **Tortus floor friction tester** is specified for dry surfaces. It drags a cylindrical rubber slider across the test surface to simulate the action of pedestrians moving slowly across the surface.

Results from both tests are corrected to adjust for sloping surfaces. The tests are non-destructive.

9.6.11 Location of reinforcement

The location and depth of steel reinforcement in hardened concrete can be measured with an electromagnetic covermeter. Although this is a non-destructive technique, it is recommended that a hole be drilled over reinforcement at least on location to calibrate the equipment for the size of steel being detected and the particular concrete used. The size and condition of the reinforcement thus exposed should be noted: is it the size specified, and is the surface free from visible contaminants and corrosion products.

Figure 9.16
Covermeter in use

9.6.12 Chemical properties

Hardened concrete can be analysed chemically to identify its composition. The most common technique used is determination of the cement content by analysis of silica and calcium contents, typically carried out to ascertain whether the mix design used was appropriate or as specified. Methods are also available to determine the presence of chemical contaminants. Chemical admixtures are difficult to detect once the concrete is more than a few days old, but the presence of mineral admixtures (supplementary cementitious materials) can be determined from silica and calcium contents.

Chemical analysis is very precise. Only small quantities, typically a few grams, are analysed so the accuracy of the analysis is critically dependant on the original sample being representative of the concrete in question, and on how the sample is reduced to produce and analytical specimen.

BS1881: part 124: 1998 specifies some analytical techniques for hardened concrete. Many analyses can now be performed by instrument rather than by wet chemistry. Specialist advice should be sought on the availability and limitations of particular techniques.

9.6.13 Appearance

Much information can be gained from a qualitative inspection of a concrete surface: the nature of any cracking and when it appeared, the uniformity of colour, evidence of large-scale deflections or unevenness, the surface texture, the presence of dusting, laitance, voids, efflorescence, delaminations (detected by sound) and the condition of joints.

The surfaces of core samples taken for physical testing or chemical analysis should always be inspected for indicators of the concrete quality: did it segregate, was it compacted properly, did it bleed excessively, are the quantities and distribution of aggregate and paste fractions satisfactory, is there a distinct surface layer? Was the concrete particularly easy or hard to cut, did the cooling fluid disappear, did the core break while being cut?

Examination of a slice of concrete by petrographic microscope can give information about the mix proportions, size and distribution of voids, nature and extent of cracking and other forms of deterioration. It is particularly useful for distinguishing localized surface properties from those of the bulk of the concrete.

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Part 3 Tests on Hardened Concrete other than for Strength
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AS 1012.4.3 Methods of Testing Concrete – Determination Of The Air Content Of Freshly-Mixed Concrete – Measuring Air Content When Concrete Dispersed In Water.
AS 1012.13 Methods of Testing Concrete – Determination of the Drying Shrinkage of Concrete for Samples Prepared in the Field or in the Laboratory
AS 1012.16 Methods for Testing Concrete – Determination of Creep of Concrete Cylinders in Compression
AS 1012.21 Methods for Testing Concrete – Determination of Water Absorption and Apparent Volume of Permeable Voids in Hardened Concrete.
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AS/NZS 4456.9 Masonry Units and Segmental Pavers – Method for Determining Abrasion Resistance
ASTM C42 Test method for obtaining and testing drilled cores and sawed beams of concrete.
ASTM C215 Test method for fundamental Transverse, Longitudinal and Torsional Resonant Frequencies of concrete specimens
ASTM C267 Standard Test Methods for chemical Resistance of Mortars, Grouts, and Monolithic Surfacings and Polymer Concretes
ASTM C469 Test method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
ASTM C512 Test Method for Creep of Concrete in Compression
ASTM C642 Test Method for Density Absorption and Voids in Hardened Concrete
ASTM C672 Test method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
ASTM C779 Test Method for Abrasion Resistance of Horizontal concrete surfaces
ASTM C803 Test method for Penetration Resistance of Hardened Concrete
ASTM C805 Test method for Rebound Number of Hardened Concrete
ASTM C900 Test Method for Pullout Strength of Hardened Concrete
ASTM C944 Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating Cutter method
ASTM C1012 Length Change of Hydraulic Cement Mortars Exposed to a Sulphate Solution
ASTM C1151 Test Method for Evaluating the Effectiveness of materials for Curing Concrete
ASTM D4263 Standard Test Method for Indicating Moisture by the plastic Sheet method
ASTM E1155 Test Method for determining FF/FL (Floor Flatness and Floor Levelness)
ASTM E1486 Test Method for Determining Floor Tolerances Using Waviness, Wheel path and Levelness Criteria
ASTM E1907 Standard Practices for Determining Moisture-Related Acceptability of Concrete Floors to Receive Moisture Sensitive Finishes
ASTM F710 Standard Practice for Preparing Concrete Floors to Receive Resilient Flooring
ASTM F1869 Standard Test Method for measuring Moisture Vapour Emission Rate of Concrete Subfloor using Anhydrous Calcium Chloride
BS 1881-124 Testing Concrete. Methods for Analysis of Hardened concrete
BS 1881-208 Testing concrete. Recommendations for the Determination of the Initial Surface Absorption of Concrete
DIN 1048

Further information about concrete testing can be found in the following:

- American Concrete Institute "Concrete Manual"
- American Society for Testing and Materials "Significance of tests and properties of Concrete and Concrete-making materials", ASTM STP169C.
- Concrete Society (UK) "Permeability Testing of Site Concrete", Technical Report 31.

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