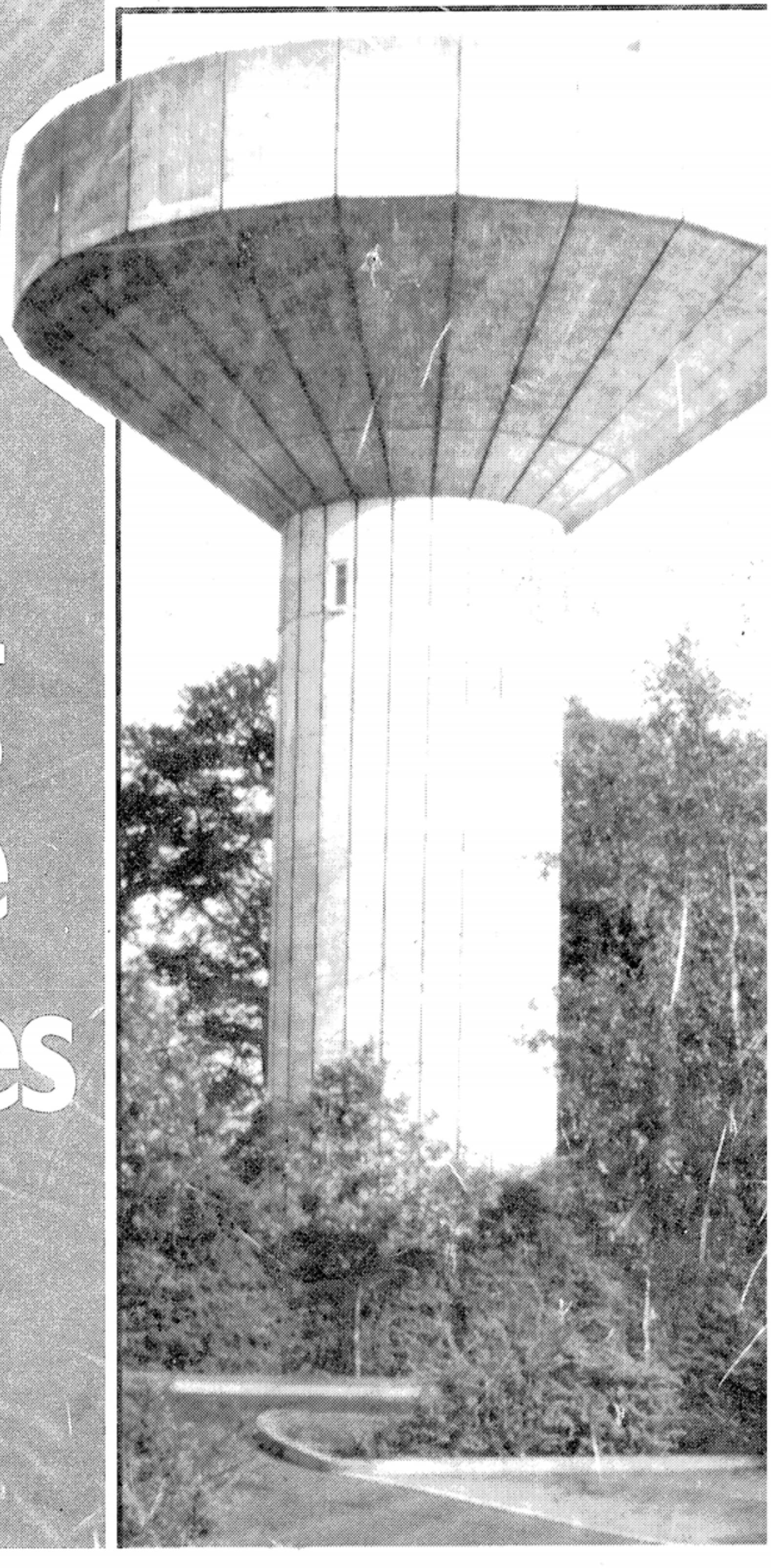
Design of Liquid Retaining Concrete Structures

Second Edition



R.D. Anchor

Design of Liquid Retaining Concrete Structures

Second edition

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Dedication

To Joy Elizabeth

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Preface to second edition

Ten years have passed since the first edition of this book was published, during which there has been considerable activity in drafting European and International Codes of Practice. In particular, a Euro-code for the design of structural concrete has been drafted, but there is as yet no extension of this document into the field covered by this book.

BS 5337:1976 Code of Practice for the structural use of concrete for retaining aqueous liquids was itself based on BSCP 110 for the design of normal structural concrete. In 1987, a fresh edition of the British Standard for normal concrete was issued as British Standard Code of Practice 8110 *Structural use of Concrete*. This was a replacement for BSCP 110. It was then necessary to revise BS 5337 to correspond with the provisions of BS 8110, and the revised code was published as BSCP 8007:1987. BS 5337 was the basis of the first edition of this book, and this second edition is based on the provisions of BS 8007. Reference is made to the USA and Australian Codes where appropriate.

Acknowledgements (*second edition*)

In the ten years that have passed since the publication of the first edition of this book, sadly, Professor Holmes and Alan Astill have died and many of my colleagues mentioned below have retired. However, Professor Barry Hughes has continued to add to my understanding of the subject, as have Andrew Beeby and Ted Thorpe. In this edition, the calculation sheets have been drawn by Chartwell Illustrators. To all my colleagues and friends, I give my thanks.

1992 R.D. Anchor

Preface to first edition

The design of any structure is a complicated matter—particularly so for civil engineering structures where the designer normally acts as structural engineer and 'architect'. Not only does the designer prepare the engineering design of the structure; he also has to consider the general layout, pipework, mechanical and electrical services, and not least, the appearance.

In a book of this size devoted to the elements of design which are applicable to liquid-retaining structures, it is not possible to deal with each separate type of reservoir or tank, and it is certainly not practicable to consider the non-structural details. I have aimed to give a full description of the design of each structural element, so that the reader will be able to design each element of any structure presented to him.

Detailed calculations are given for a range of structural types, and design tables and charts have been included to make the book complete in itself. The notation follows the international system, and metric units are used. The choice of units follows current UK engineering practice rather than adhering strictly to the SI system. The calculations are presented mostly to an accuracy of three significant figures, which is adequate in relation to the accuracy of the basic data and material strength. The author makes no apology for using a value of 10 kN/m^3 for the weight of water rather than the more accurate value of 9.81. The difference is less than 2%.

Acknowledgements (*first edition*)

I am conscious of the specific and indirect assistance that I have received from my friends and fellow-engineers in writing this book and would like to mention in particular Messrs. A. Allen, A. Astill, A. W. Hill, Professors M. Holmes and B. P. Hughes, and Dr. R. Savidge.

I must also thank Anthony J. Harman who has drawn the figures and calculation sheets with care and enthusiasm. Appendix A, together with the design charts in Appendix B, are reproduced by permission of the Cement and Concrete Association from *Handbook to BS 5337* by R. D. Anchor, A. W. Hill and B. P. Hughes (Viewpoint Publication 14-011). Figure 1.2 is reproduced by kind permission of Thomas Garland and Partners, Consulting Engineers, Dublin.

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Contents

Notation

a_a	distance between the point considered and the axis of the nearest longitudinal bar
a_{cr}	distance between the point considered and the surface of the nearest longitudinal bar
a'	distance between the compression face and the point at which the crack width is being calculated
A_s	area of steel reinforcement
A_{sv}	cross-sectional area of shear reinforcement
b	breadth (or width) of section
c	nominal cover to tension steel
c_{min}	minimum cover to the tension steel
d	effective depth of tension reinforcement; diameter of tank
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of steel reinforcement
f_b	average bond strength between concrete and steel
$f_{c'}$	characteristic cylinder strength of concrete at 28 days
f_{ct}	direct tensile strength of the concrete; tensile stress in concrete
f_{cu}	characteristic cube strength of concrete at 28 days
f_{st}	design steel stress in tension (allowable stress for limit-state design or permissible stress for alternative design)
f_{sv}	design steel stress in shear reinforcement (i.e. allowable stress for limit-state design or permissible stress for alternative design)
f_k	characteristic strength
f_t	ring tension per unit length
f_y	characteristic strength of the reinforcement
F_t	ring tension
h	overall depth of member
H	depth of liquid
h_c	diameter of column or column head
l_1	length of panel in the direction of span, measured from the centres of columns
l_2	width of a panel measured from the centres of columns
l_m	average of l_1 and l_2
L	length, span
m	bending moment per unit width

M	bending moment
M_d	design (service) moment of resistance
M_u	ultimate moment of resistance
n	total load per unit area (BS 8110 ultimate load)
n_b	number of bars in width of section
q	distributed imposed load per unit length or per unit area
r	radius of tank
s	spacing
s_{max}	estimated maximum crack spacing
s_{min}	estimated minimum crack spacing
t	thickness of wall of tank
T_1	fall in temperature from hydration peak to ambient
T_2	seasonal fall in temperature
v	shear stress; shear force per unit length
v_c	critical concrete shear stress for ultimate limit state
V	total shear force
w_g	unit weight
w_{max}	estimated maximum crack width
x	depth of the neutral axis
z	lever arm
α	coefficient of thermal expansion of mature concrete; coefficient
α_e	modular ratio
β	coefficient
γ_f	partial safety factor for load
γ_m	partial safety factor for material strength
ε_{cs}	estimated shrinkage strain
ε_m	average strain at the level at which cracking is being considered, allowing for the stiffening effect of the concrete in the tension zone (see Appendix A)
ε_{te}	estimated total thermal contraction strain after peak temperature due to heat of hydration
ε_1	strain at the level considered, ignoring the stiffening effect of the concrete in the tension zone
ε_2	strain due to stiffening effect of concrete
ε_{ult}	ultimate concrete tensile strain
ρ	steel ratio based on *bd*; density of liquid
ρ_c	steel ratio based on gross concrete section
ρ_{crit}	critical steel ratio, based on gross concrete section (pronounced 'rho crit')
ϕ	bar size

Metric units

The units of measure used in this book are those which are currently widely used in the United Kingdom. They are based on the metric SI system but are not quite 'pure'. For the benefit of readers who are not used to metric units, an approximate conversion is given below.

Quantity	*Unit*
Length	metre (m)
Member sizes, etc.	millimetre (mm)
Force	Newton (N) kilonewton (kN)
Stress, pressure	N/mm^2, kN/mm^2, kN/m^2
Volume	m^3, litres

Approximate conversions

1 metre	= 39 inches
100 mm	= 4 inches
300 mm	= 1 foot
10 kN	= 1 ton
7 N/mm^2	= 1000 lb/sq. in.
100 kN/m^2	= 1 ton/sq. ft.
1 m^3	= 1000 litres
1 litre	= 0.22 Imperial gallons = 0.2642 US gallons

Note: 1 N/mm^2 = 1 MN/m^2 = 1 MPa

Concrete strength

The formulae used in this book are based on British practice, where concrete strength is evaluated using test cubes rather than cylinders.

The relation between cube and cylinder strength is usually taken as

$$\frac{f_{c'}}{f_{cu}} = 0.78$$

but the ratio varies widely according to the type of aggregate.

Loads

Characteristic loads and strengths are those values used for design purposes and are based on a statistical evaluation.

Service loads and stresses are calculated with applied characteristic loads and generally with a partial safety factor for loads equal to 1.0.

Ultimate loads and stresses are calculated with applied characteristic loads and a partial safety factor for loads which is generally 1.4.

Design of
Liquid Retaining Concrete
Structures

1

Introduction

1.1 Scope

It is common practice to use reinforced or prestressed concrete structures for the storage of water and other aqueous liquids. Similar design methods may also be used to design basements in buildings where ground water must be excluded, and Chapter 8 deals with methods of preventing vapour transmission through concrete structures. Concrete is generally the most economical material of construction and, when correctly designed and constructed, will provide long life and low maintenance costs. The types of structure which are covered by the design methods given in this book are: storage tanks, reservoirs, swimming pools, elevated tanks, ponds, settlement tanks, basement walls, and similar structures (Figs 1.1 and 1.2). Specifically excluded are: dams, structures subjected to dynamic forces; and pipelines, aqueducts or other types of structure for the conveyance of liquids.

It is convenient to discuss designs for the retention of water, but the principles apply equally to the retention of other aqueous liquids. In particular, sewage tanks are included. The pressures on a structure may have to be calculated using a specific gravity greater than unity, where the stored liquid is of greater density than water. Throughout this book it is assumed that water is the retained liquid unless any other qualification is made. The term 'structure' is used in the book to describe the vessel or container that retains or excludes the liquid.

The design of structures to retain oil, petrol and other penetrating liquids is not included and is dealt with in specialist literature(1). Likewise, the design of tanks to contain hot liquids is not discussed(2,44).

1.2 General design objectives

A structure that is designed to retain liquids must fulfil the requirements for normal structures in having adequate strength, durability, and freedom from excessive cracking or deflection. In addition, it must be designed so that the liquid is not allowed to leak or percolate through the concrete structure. In the design of normal building structures, the most critical aspect of the design

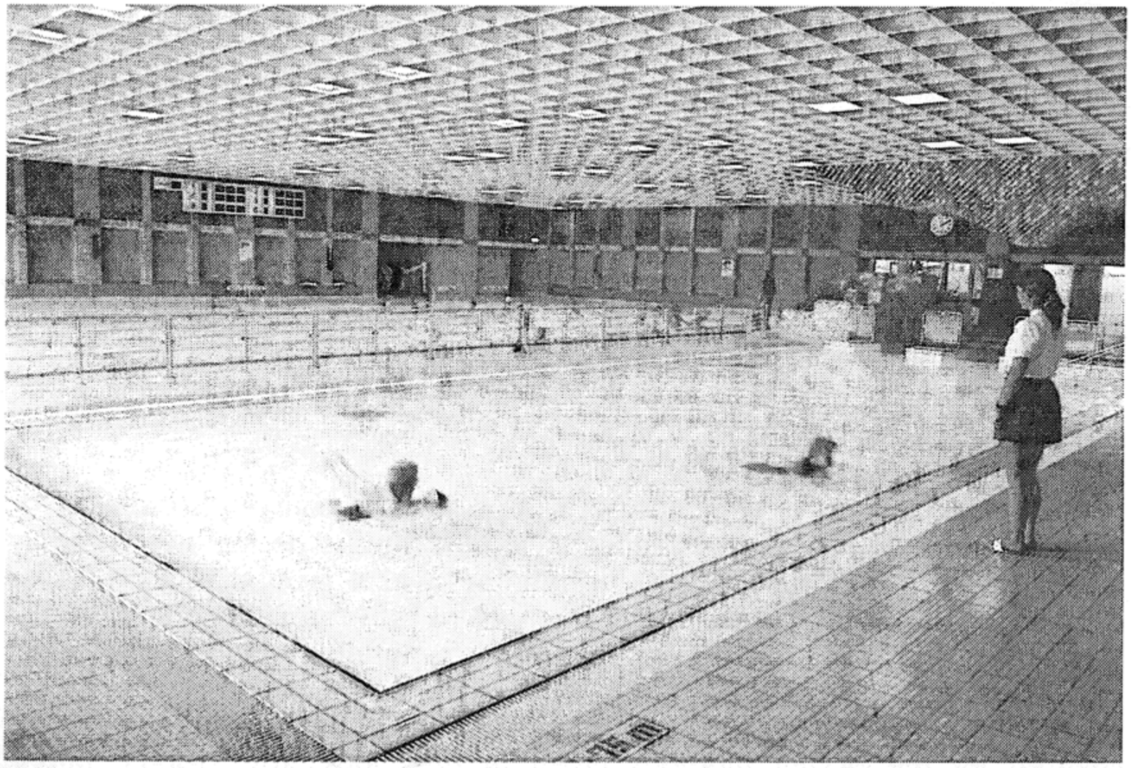


Fig. 1.1

is to ensure that the structure retains its stability under the imposed loads. In the design of structures to retain liquids, it is usual to find that, if the structure has been proportioned and reinforced so that the liquid is retained without leakage, then the strength is more than adequate. The requirements for ensuring a reasonable service life for the structure without undue maintenance are more onerous for liquid-retaining structures than for normal structures, and adequate concrete cover to the reinforcement is essential. Equally, the concrete itself must be of good quality, and be properly compacted: good workmanship during construction is critical.

Potable water from moorland areas may contain free carbon dioxide or dissolved salts from the gathering grounds which attack normal concrete. Similar difficulties may occur with tanks which are used to store sewage or industrial liquids. After investigating by tests the types of aggressive elements that are present, it may be necessary to increase the cement content of the concrete mix, use special cements or, under severe conditions, use a special lining to the concrete tank(3,4).

1.3 Fundamental design methods

Historically, the design of structural concrete has been based on elastic theory, with specified maximum design stresses in the materials at working

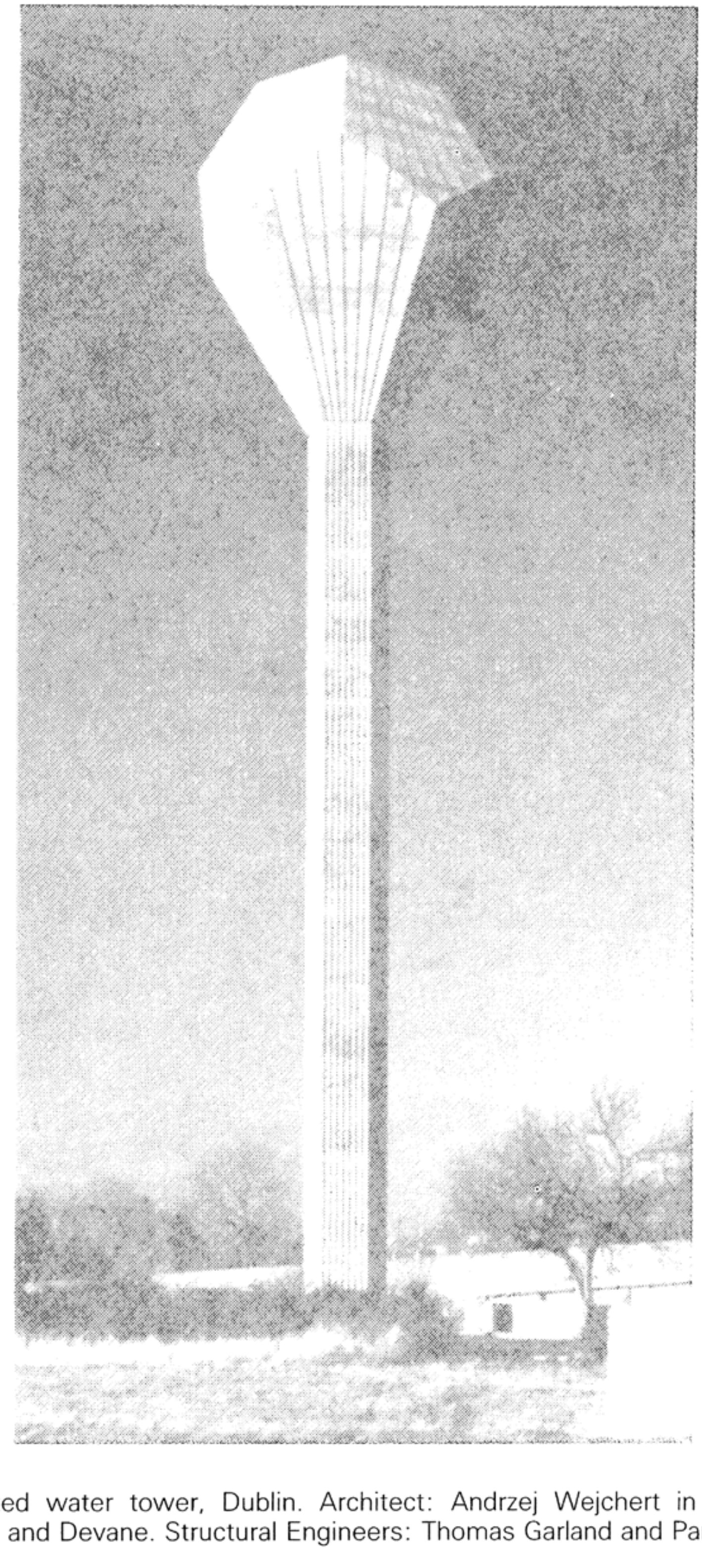


Fig. 1.2 Elevated water tower, Dublin. Architect: Andrzej Wejchert in association with Robinson, Keefe and Devane. Structural Engineers: Thomas Garland and Partners, Consulting Engineers

loads. More recently, limit state philosophy has been introduced, providing a more logical basis for determining factors of safety. In ultimate design, the working or characteristic loads are enhanced by being multiplied by a *partial safety factor*. The enhanced or ultimate loads are then used with the failure strengths of the materials to design the structure. Limit state design methods are now widely used throughout the world for normal structural design(5,6,7).

Formerly, the design of liquid-retaining structures was based on the use of elastic design, with material stresses so low that no flexural tensile cracks developed. This led to the use of thick concrete sections with copious quantities of mild steel reinforcement. The probability of shrinkage and thermal cracking was not dealt with on a satisfactory basis, and nominal quantities of reinforcement were specified in most codes of practice. More recently, analytical procedures have been developed to enable flexural crack widths to be estimated and compared with specified maxima(8). A method of calculating the effects of thermal and shrinkage strains has also been published(9). These two developments enable limit state methods to be extended to the design of liquid-retaining structures.

Limit state design methods enable the possible modes of failure of a structure to be identified and investigated so that a particular premature form of failure may be prevented. Limit states may be 'ultimate' (where ultimate loads are used) or 'serviceability' (where design or service loads are used).

In the UK, limit state design has been used successfully for over 10 years for the design of liquid retaining structures. The former BS 5337 allowed a designer to choose between elastic design and limit state design. However, as nearly all designers decided to use limit state design, BS 8007 solely recommends limit state design. In Australia and the USA, design methods based on elastic theory are specified in the national codes. Elastic design is a simpler process, but with the widespread use of computer facilities, there is no difficulty in preparing limit state designs.

At the time of writing, there is much activity in drafting 'European standards' and a final draft of EC2 for normal concrete structural design is in existence. Although there is a 'European' liquid retaining design code in the programme of work, no committee has yet been set up. The design of liquid-retaining structures in Europe is clouded in mystery. The author has unsuccessfully attempted to discover how such structures are designed in a number of European countries but without any detailed success. However, any design system that enables a serviceable structure to be constructed with due economy is acceptable. As has often been said 'A structure does not know how it has been designed'.

1.4 Codes of practice

Structural design is often governed by a Code of Practice appropriate to the location of the structure. Whilst the basic design objectives are similar in each code, the specified stresses and factors of safety may vary. It is important to consider the climatic conditions at the proposed site, and not to use a code of practice written for temperate zones in parts of the world with more extreme weather conditions.

Three widely used codes are:

1. British Standard Code of Practice BS 8007:1987 Design of concrete structures for retaining aqueous liquids(10).
2. American Concrete Institute ACI 350 R-83: Concrete Sanitary Engineering Structures(11).
3. Australian Standard AS 3735 – 1991 Concrete structures for retaining aqueous liquids.

All three codes include material specifications, joint details and design procedures to limit cracking.

BS 8007(10)

British Standard Code of Practice BS 8007:1987 is a revised version of BS 5337:1976, which itself derives from BSCP 2007. The relation between the normal concrete codes and the liquid retaining codes is shown below:

Normal code	*Liquid retaining code*
BSCP 114	BSCP 2007
BSCP 110	BSCP 5337
BSCP 8110	BSCP 8007

ACI 350 R-83(11)

The original report appeared in 1971 and was amended for the 1977 edition, and again in 1983. Structural design is included, with special emphasis on minimizing the possibility of cracking.

1.5 Impermeability

Concrete for liquid-retaining structures must have low permeability. This is necessary to prevent leakage through the concrete and also to provide adequate durability, resistance to frost damage, and protection against corrosion for the reinforcement and other embedded steel. An uncracked concrete slab of adequate thickness will be impervious to the flow of liquid if the concrete mix has been properly designed and compacted into position. The specification of suitable concrete mixes is discussed in Chapter 2. The minimum thickness of concrete for satisfactory performance in most structures is 200 mm. Thinner slabs should only be used for structural members of very limited dimensions or under very low liquid pressures.

Liquid loss may occur at joints that have been badly designed or constructed, and also at cracks or from concrete surfaces where incomplete compaction has been achieved. It is nearly inevitable that some cracking will be present in all but the simplest and smallest of structures. If a concrete slab cracks for any reason, there is a possibility that liquid may leak or that a wet patch will occur on the surface. However, it is found that cracks of limited width do not allow liquid to leak(12) and the problem for the designer is to limit the surface crack widths to a predetermined size. Cracks due to shrinkage and thermal movement tend to be of uniform thickness through the thickness of the slab, whereas cracks due to flexural action are of limited depth and are backed up by a depth of concrete that is in compression. Clearly, the former type of crack is more serious in allowing leakage to occur.

1.6 Site conditions

The choice of site for a reservoir or tank is usually dictated by requirements outside the structural designer's responsibility, but the soil conditions may radically affect the design. A well-drained site with underlying soils having a uniform safe bearing pressure at foundation level is ideal. These conditions may be achieved for a service reservoir near to the top of a hill, but at many sites where sewage tanks are being constructed, the subsoil has a poor bearing capacity and the ground water table is near to the surface. A high level of ground water must be considered in designing the tanks in order to prevent flotation (Fig. 1.3), and poor bearing capacity may give rise to increased settlement. Where the subsoil strata dip, so that a level excavation intersects more than one type of subsoil, the effects of differential settlement must be considered (Fig. 1.4). A soil survey is always necessary unless an accurate

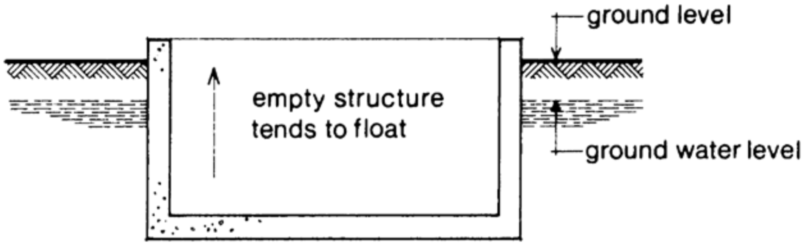


Fig. 1.3 Tank flotation due to ground water

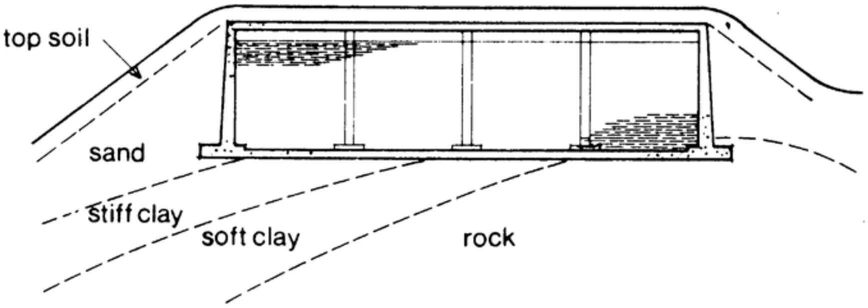


Fig. 1.4 Effect of varying strata on settlement

record of the the subsoil is available. Boreholes of at least 150 mm diameter should be drilled to a depth of 10 m, and soil samples taken and tested to determine the sequence of strata and the allowable bearing pressure at various depths. The information from boreholes should be supplemented by digging trial pits with a small excavator to a depth of 3–4 metres.

The soil investigation must also include chemical tests on the soils and ground water to detect the presence of sulphates or other chemicals in the ground which could attack the concrete and eventually cause corrosion of the reinforcement(4). Careful analysis of the subsoil is particularly important when the site has previously been used for industrial purposes, or where ground water from an adjacent tip may flow through the site. Further information is given in Chapter 2.

When mining activity is suspected, a further survey may be necessary and a report from the mineral valuer or a mining consultant is necessary. Deeper, randomly located boreholes may be required to detect any voids underlying the site. The design of a reservoir to accept ground movement due to future mining activity requires the provision of extra movement joints or other measures to deal with the anticipated movement and is outside the scope of this textbook(13). In some parts of the world, consideration must be given to the effects of earthquakes, and local practice should be ascertained.

1.7 Influence of construction methods

Any structural design has to take account of the constructional problems involved and this is particularly the case in the field of liquid-retaining structures. Construction joints in building structures are not normally shown on detailed drawings but are described in the specification. For liquid-retaining structures, construction joints must be located on drawings, and the contractor is required to construct the works so that concrete is placed in one operation between the specified joint positions. The treatment of the joints must be specified, and any permanent movement joints must be fully detailed. All movement joints require a form of waterstop to be included, but construction joints may generally be designed without using a waterstop. Details of joint construction are given in Chapter 5. In the author's opinion, the detailed design and specification of joints is the responsibility of the designer and not the contractor. The quantity of distribution reinforcement in a slab and the spacing of joints are interdependent. Casting one section of concrete adjacent to another section, previously cast and hardened, causes restraining forces to be developed which tend to cause cracks in the newly placed concrete. It follows that the quantity of distribution reinforcement also depends on the degree of restraint provided by the adjacent panels.

Any tank which is to be constructed in water-bearing ground must be designed so that the ground water can be excluded during construction. The two main methods of achieving this are by general ground de-watering, or by using sheet piling. If sheet piling is to be used, consideration must be given to the positions of any props that are necessary, and the sequence of construction which the designer envisages(14).

1.8 Design procedure

As with many structural design problems, once the member size and reinforcement have been defined, it is relatively simple to analyse the strength of a structural member and to calculate the crack widths under load: but the designer has to estimate the size of the members that he proposes to use before any calculations can proceed. With liquid-retaining structures, crack-width calculations control the thickness of the member, and therefore it is impossible to estimate the required thickness directly unless the limited stress

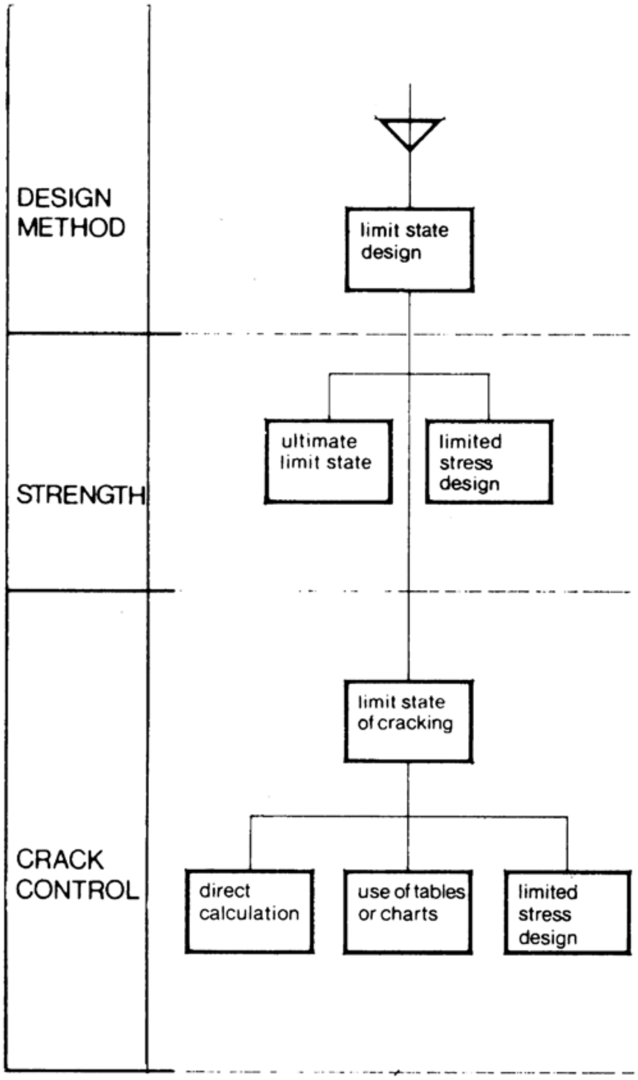


Fig. 1.5 Design methods

method of design is used. Because of these difficulties, design charts have been prepared which allow designers to choose directly the section thickness and quantity of reinforcement required. The design charts enable limit state designs to be prepared quickly and accurately (see Appendix A).

An intermediate method of design is also possible where the limit state of cracking is satisfied by limiting the reinforcement stress rather than by preparing a full calculation. This procedure is particularly useful for sections under combined flexural and direct stresses. Figure 1.5 illustrates the options available to the designer.

1.9 Code requirements (UK)

BS 8007 is based on the recommendations of BS 8110 for the design of normal structural concrete, and the design and detailing of liquid-retaining structures should comply with BS 8110 except where the recommendations of BS 8007 vary the requirements. The main variations are:

(*a*) Section 2 of BS 8007 takes precedence over Section 2 of BS 8110 in respect of the basis of design.

(*b*) The design ultimate anchorage bond length for horizontal bars in sections subject to direct tension should not exceed 0.7 times the values obtained from clause 3.12.8.4 of BS 8110.

(*c*) The maximum calculated design crack widths are either 0.2 mm or 0.1 mm depending on the exposure conditions, rather than an assumed value of 0.3 mm.

(*d*) The basis of design is the serviceability limit state of cracking rather than the ultimate limit state.

(*e*) For the design of flat slab roofs at serviceability limit states, the simplified method of design in clause 3.7.2.7 of BS 8110 may be used (with a proviso on the size of the column heads).

(*f*) The provisions of BS 8007 in respect of joints are to be used.

(*g*) The provisions of BS 8007 in respect of nominal cover are to be used.

(*h*) The provisions of BS 8007 in respect of exposure conditions are to be used.

(*j*) The provisions of BS 8007 in respect of minimum areas of reinforcement are to be used.

(*k*) BS 8007 contains restrictions relating to the spacing of bar reinforcement.

2

Basis of design and materials

2.1 Structural action

It is necessary to start a design by deciding on the type and layout of structure to be used. Tentative sizes must be allocated to each structural element, so that an analysis may be made and the sizes confirmed.

All liquid-retaining structures are required to resist horizontal forces due to the liquid pressures. Fundamentally there are two ways in which the pressures can be contained:

(*a*) By forces of direct tension or compression (Fig. 2.1).
(*b*) By flexural resistance (Fig. 2.2).

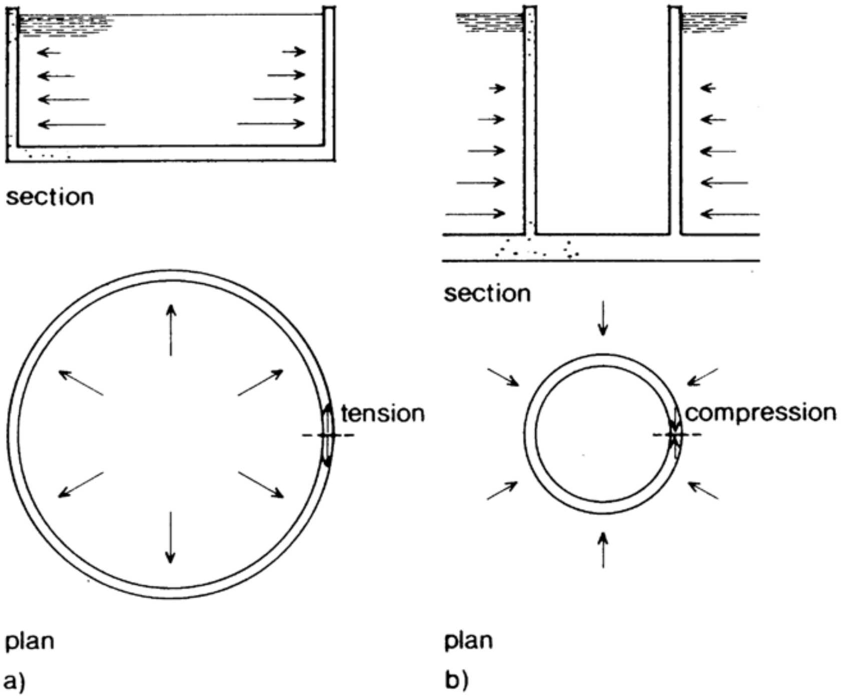


Fig. 2.1 Direct forces in circular tanks
(*a*) Tensile forces
(*b*) Compressive forces

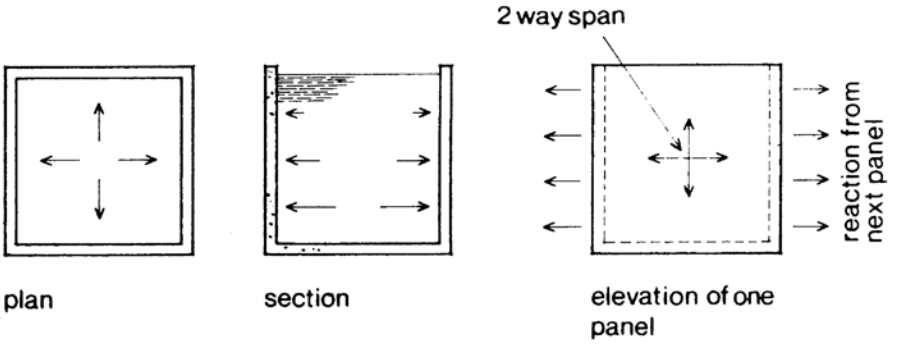


Fig. 2.2 Direct forces of tension in wall panels of rectangular tanks

Structures designed by using tensile or compressive forces are normally circular and may be prestressed. Rectangular tanks or reservoirs rely on flexural action using cantilever walls, propped cantilever walls or walls spanning in two directions. A structural element acting in flexure to resist liquid pressure reacts on the supporting elements and causes direct forces to occur. The simplest illustration (Fig. 2.3) is a small tank. Additional reinforcement is necessary to resist such forces unless they can be resisted by friction on the soil.

2.2 Exposure classification

Structural concrete elements are exposed to varying types of environmental conditions. The roof of a pumphouse is waterproofed with asphalt or roofing felt and, apart from a short period during construction, is never exposed to wet or damp conditions. The exposed legs of a water tower are subjected to alternate wetting and drying from rainfall but do not have to contain liquid. The lower sections of the walls of a reservoir are always wet (except for brief periods during maintenance), but the upper sections may be alternately wet and dry as the water level varies. The underside of the roof of a closed reservoir is damp from condensation. These various conditions are illustrated in Figure 2.4.

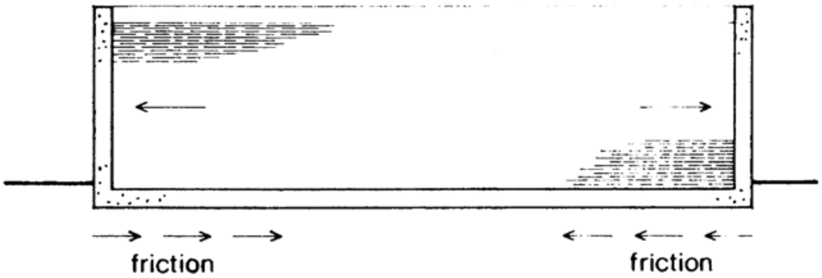


Fig. 2.3 Tension in floor of a long tank with cantilever walls

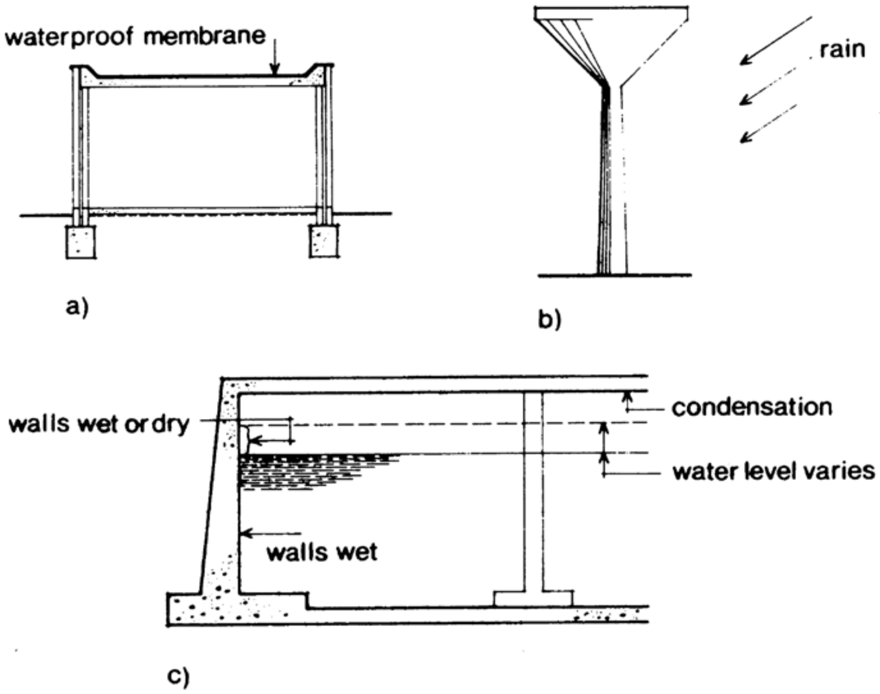


Fig. 2.4 Exposure to environmental conditions
(*a*) Pumphouse roof
(*b*) Water tower
(*c*) Reservoir

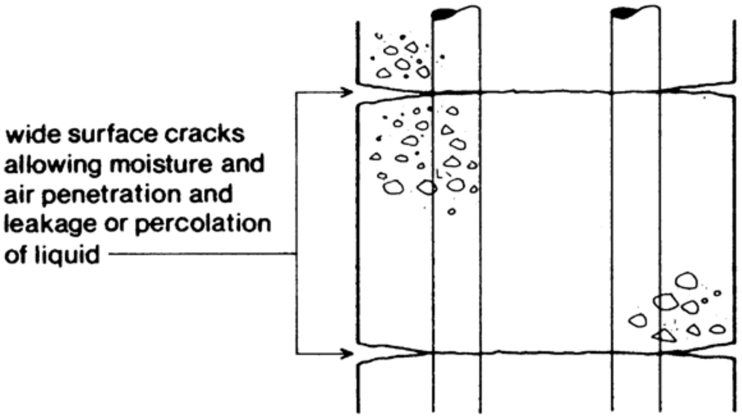


Fig. 2.5 Effect of cracks

Experience has shown that, as the exposure conditions become more severe, precautions should be taken to ensure that moisture and air do not cause carbonation in the concrete cover to the reinforcement thus removing the protection to the steel and causing corrosion which, in turn, will cause the concrete surface to spall(15). Adequate durability can be ensured by providing a dense well-compacted concrete mix (see Section 2.5.2) with a concrete cover of 40 mm, but it is also necessary to control cracking in the concrete, and prevent percolation of liquid through the member (see Fig. 2.5).

For design purposes, it is convenient to classify exposure conditions, and in previous codes this was achieved by using classes A, B, and C for various situations. However, some simplification has been achieved and both faces of a liquid-retaining structure are now to be designed for a crack width of 0.2 mm except where the element is visible and the appearance is important. In this case a design crack width of 0.1 mm is recommended. Where the environmental conditions are more demanding, then the concrete grade and the cover should be adjusted in accordance with the recommendations of BS 8110. Where aggressive ground water conditions exist the concrete mix to be specified will also need to be specially considered. Table 2.1 specifies the various exposure conditions which are recommended in BS 8110 (Table 3.2 of BS 8110). BS 8007 requires that all liquid-retaining structures should be designed for at least 'severe' conditions of exposure. Where appropriate the 'very severe' and 'extreme' categories should be used. As an example, a water tower near to the sea coast and exposed to salt water spray would be designed for 'very severe' exposure.

Table 2.1 Exposure conditions

Environment	Exposure conditions
Mild	Concrete surfaces protected against weather or aggressive conditions
Moderate	Concrete surfaces sheltered from severe rain or freezing whilst wet Concrete subject to condensation Concrete surfaces continuously under water Concrete in contact with non-aggressive soil
Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing or severe condensation
Very severe	Concrete surfaces exposed to sea water spray, de-icing salts (directly or indirectly), corrosive fumes or severe freezing conditions whilst wet
Extreme	Concrete surfaces exposed to abrasive action, e.g. sea water carrying solids or flowing water with pH $\leqslant$ 4.5 or machinery or vehicles

For situations where aesthetic conditions demand an even higher safety margin against the possibility of leakage or unsightly cracking, it is possible to design for no cracking to occur (see Section 3.9). As previously stated, in practice, this does not mean that cracking will be completely avoided, but any cracks which do occur should be very narrow.

Table 2.2 Examples of exposure classification

Exposure	Crack width (mm)	Element
Severe	0.2	Walls, floor and roof of underground tank (no aggressive soil)
Severe	0.1	Elevated water tower
Very severe	0.2	Tank exposed to wind-blown sea spray
Very severe	0.1	Elevated water tank exposed to wind-blown sea spray

2.3 Structural layout

The layout of the proposed structure and the estimation of member sizes must precede any detailed analysis. Structural schemes should be considered from the viewpoints of strength, serviceability, ease of construction, and cost. These factors are to some extent mutually contradictory, and a satisfactory scheme is a compromise, simple in concept and detail. In liquid-retaining structures, it is particularly necessary to avoid sudden changes in section, because they cause concentration of stress and hence increase the possibility of cracking.

It is a good principle to carry the structural loads as directly as possible to the foundations, using the fewest structural members. It is preferable to design cantilever walls as tapering slabs rather than as counterfort walls with slabs and beams. The floor of a water tower or the roof of a reservoir can be designed as a flat slab. Underground tanks and swimming-pool tanks are generally simple structures with constant-thickness walls and floors.

It is essential for the designer to consider the method of construction and to specify on the drawings the position of all construction and movement joints. This is necessary as the detailed design of the structural elements will depend on the degree of restraint offered by adjacent sections of the structure to the section being placed. Important considerations are the provision of ‘kickers’ (or short sections of upstand concrete) against which formwork may be tightened, and the size of wall and floor panels to be cast in one operation (Figs 2.8 and 2.9).

2.4 Influence of construction methods

Designers should consider the sequence of construction when arranging the layout and details of a proposed structure. At the excavation stage, and

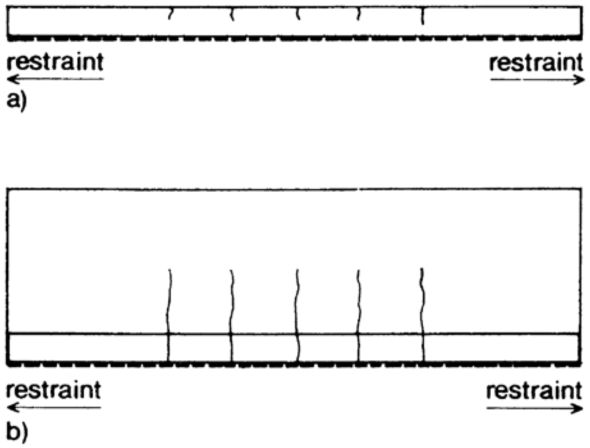


Fig. 2.6 Cracking due to restraint by frictional forces at foundation level
(*a*) Floor slab
(*b*) Wall

particularly on water-logged sites, it is desirable that the soil profile to receive the foundation and floors should be easily cut by machine. Flat surfaces and long strips are easy to form but individual small excavations are expensive to form. The soil at foundation level exerts a restraining force on the structure which tends to cause cracking (Fig. 2.6). The frictional forces can be reduced by laying a sheet of 1000 g polythene or other suitable material on a 75 mm layer of 'blinding' concrete. For the frictional forces to be reduced, it is necessary for the blinding concrete to have a smooth and level surface finish. This can only be achieved by a properly screeded finish, and in turn this implies the use of a grade of concrete which can be so finished(16,17). A convenient method is to specify the same grade of concrete for the blinding layer as is used for the structure. This enables a good finish to be obtained for the blinding layer, and also provides an opportunity to check the strength and consistency of the concrete at a non-critical stage of the job.

The foundations and floor slabs are constructed in sections which are of a convenient size and volume to enable construction to be finished in the time available. Sections terminate at a construction or movement joint (Chapter 5). The construction sequence should be continuous as shown in Figure 2.7 *a* and not as shown in Figure 2.7 *b*. By adopting the first system, each section that is cast has one free end and is enabled to shrink on cooling without restraint (a day or two after casting). With the second method, considerable tensions are developed between the relatively rigid adjoining slabs.

The maximum spacing between movement joints depends on the amount of reinforcement provided, but generally about 7.5 m is the economical maximum distance between partial contraction joints, and 15.0 m between full contraction joints. Alternatively, temporary short gaps may be left out, to be filled in after the concrete has hardened. A further possibility is the use of induced contraction joints, where the concrete section is deliberately reduced

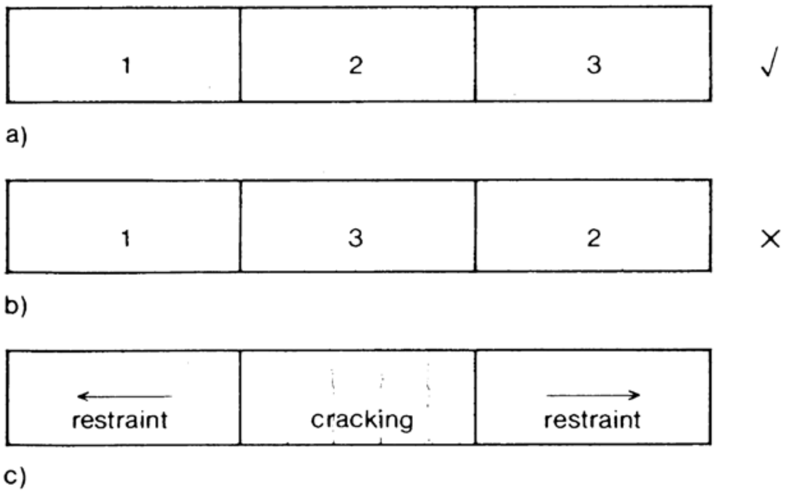


Fig. 2.7 Construction sequence
(*a*) Preferred sequence
(*b*) Not recommended
(*c*) Effect of method (*b*) on third slab panel

in order to cause cracks to form at preferred positions. These possibilities are illustrated in Figure 2.8. The casting sequence in the vertical direction is usually obvious. The foundations or floors are laid with a short section of wall to act as a key for the formwork (the kicker, Fig. 2.9). Walls may be concreted in one operation up to about 8 m height.

Reinforcement should be detailed to enable construction to proceed with a convenient length of bar projecting from the sections of concrete which are placed at each stage of construction. Bars should have a maximum spacing of 300 mm or the thickness of the slab and a minimum spacing dependent on size, but not usually less than 100 mm to allow easy placing of the concrete. Distribution or shrinkage reinforcement should be placed in the outer layers nearest to the surface of the concrete. In this position it has maximum effect.

2.5 Materials and concrete mixes

2.5.1 *Reinforcement*(18)

Although the service tensile stress in the reinforcement in liquid-retaining structures is not always very high, it is usual to specify high-strength steel with a ribbed or deformed surface. The difference in cost between high-strength ribbed steel and plain-surface mild steel is only about 3% (UK). This small extra cost is more than saved by the extra strength available and increased bond performance. Similar arguments affect the use of welded fabric reinforcement, where fixing costs are very much reduced and time saved. Traditionally, fabric has been used only in ground slabs but, where the

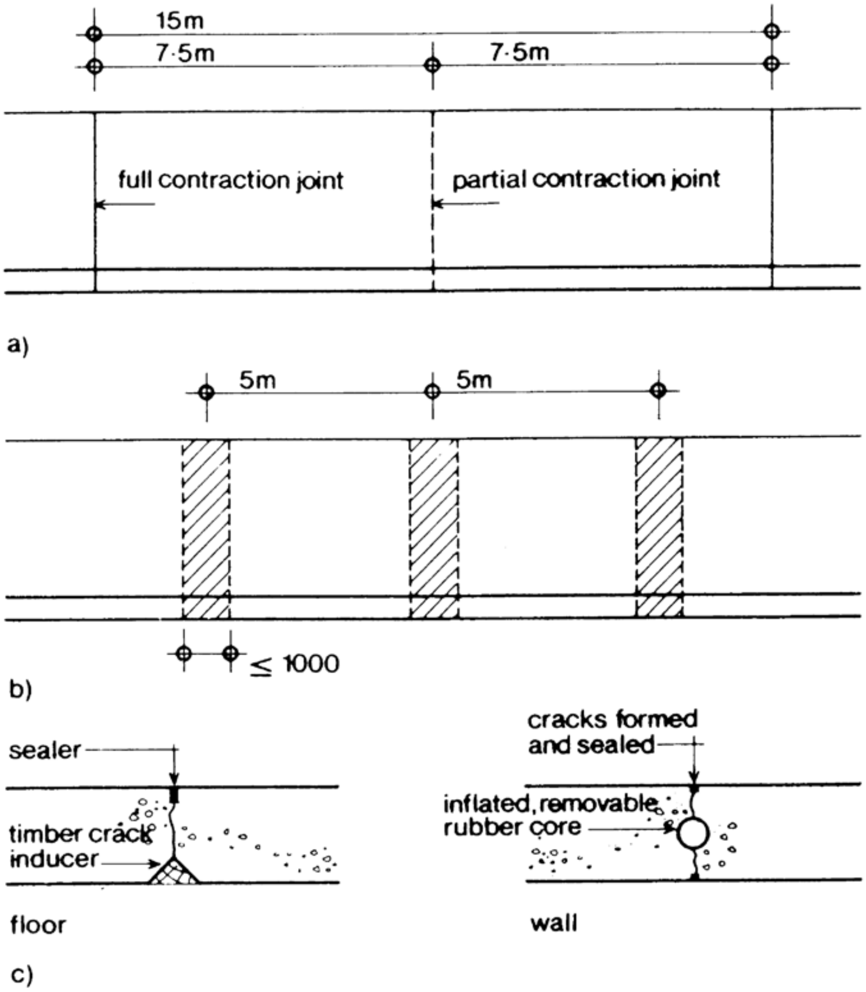


Fig. 2.8 Joints
(*a*) Typical layout of joints in a wall
(*b*) Typical layout of temporary gaps in construction
(*c*) Induced joints

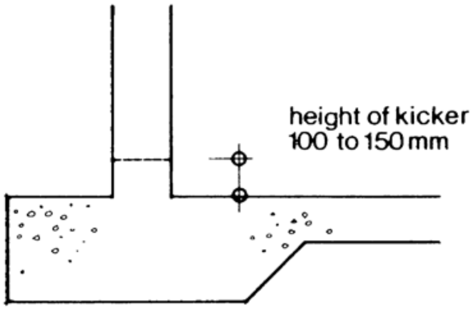


Fig. 2.9 Joint between floor and wall

quantity is sufficient, can now be obtained in sizes and types that allow it to be used in walls, floors and roofs.

The specified characteristic strengths of reinforcement available in the UK are given in Table 2.3. The specified characteristic strength is a statistical measure of the yield or proof stress of a type of reinforcement. For bars supplied in accordance with British Standards, the proportion of bars which fall below the characteristic strength level is defined as 5% (Fig. 2.10)(19,20,21). A material partial safety factor $\gamma_m = 1.15$ is applied to the specified characteristic strength to obtain the ultimate design strength.

Reinforcement embedded in concrete is protected from corrosion by the alkalinity of the cement. As time passes, the surface of the concrete reacts with carbon dioxide from the air and carbonates are formed which remove the protection. The specified cover of at least 40 mm is adequate for normal conditions, but where particularly aggressive conditions apply, it is worth considering the use of a special type of reinforcement. The possibilities are:

(*a*) galvanized bars (×1.5)
(*b*) epoxy-coated bars (×2.0)
(*c*) stainless steel bars (×10)

The numbers in brackets give an indication of the average cost of the special bars compared with normal steel. Special bars may sometimes be convenient to use in a particularly thin element where it is not possible to obtain the proper cover with several layers of steel.

2.5.2 *Concrete*

The detailed specification and design of concrete mixes is outside the scope of this book, but the essential features of a design are given below. Guidance on mix design may be found from the references(16,17).

Cements Normal Portland cement is generally used for liquid-retaining structures. It is not desirable to use rapid-hardening cement because of its greater evolution of heat which tends to increase shrinkage cracking. However, its use may be considered in cold weather. When there are sulphates in the ground water, or other chemical contaminants, the use of sulphate-resisting cement or super-sulphated cement may be essential.

Table 2.3 Types and strengths of reinforcement (UK)

BS	Type	Characteristic yield or proof stress f_y (N/mm^2)
4449	Plain mild steel	250
4449	Hot-rolled high-yield ribbed bars	460
4449	Cold-worked high-yield ribbed bars	460
4483	Welded fabric	460

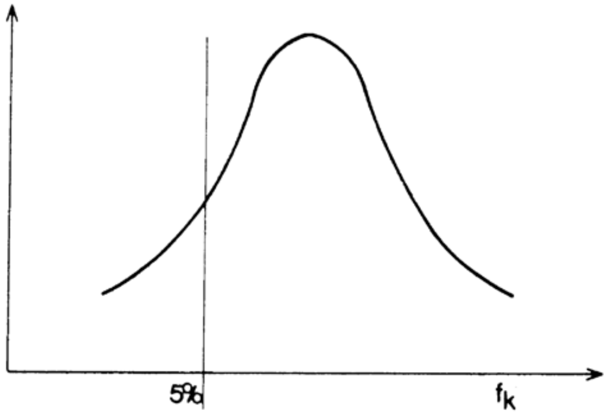


Fig. 2.10 Graphical definition of characteristic strength

Aggregates The maximum size of aggregate must be chosen in relation to the thickness of the structural member. A maximum size of 20 mm is always specified up to member thickness of about 300–400 mm, and may be used above this limit. Size 40 may be specified in very thick members. The use of a large maximum size of aggregate has the effect of reducing the cement content in the mix for a given workability, and hence reduces the amount of shrinkage cracking.

It is important to choose aggregates that have low drying shrinkage and low absorption. Most quartz aggregates are satisfactory in these respects but, where limestone aggregate is proposed, some check on the porosity is desirable. Certain aggregates obtained from igneous rocks exhibit high shrinkage properties and are quite unsuitable for use in liquid-retaining structures(22).

Local suppliers can often provide evidence of previous use which will satisfy the specifier, but some care is necessary in using material from a new quarry, and tests of the aggregate properties are recommended.

Admixtures Admixtures containing calcium chloride are not desirable as there is a risk of corrosion of the reinforcement. Other admixtures which improve workability or frost resistance may be used on their merits(43).

Concrete mix design The concrete must be designed to provide a mix which is capable of being fully compacted by the means available. Any areas of concrete which have not been properly compacted are likely to leak. The use of poker-type internal vibrators is recommended.

The cement content in kg/m^3 of finished concrete must be judged in relation to a minimum value to ensure durability, and a maximum value to avoid a high temperature rise in the freshly placed concrete.

Table 2.4 C35A mix to BS 8007

Minimum cement content	325 kg/m^3
Maximum cement content	400 kg/m^3
Maximum free water/cement ratio	0.55

Note: Some adjustments are specified when using cement replacement materials.

There has recently been a considerably increased use of pulverised fuel ash (pfa) and ground granulated blastfurnace slags (ggbs) as replacement materials for a proportion of the cement. The reason for considering these materials is to counteract the increase (over the years) of the strength of cement. In order to have sufficient cement content in the mix to give adequate durability, it is now necessary to specify high strengths with consequent high evolution of heat. In 1950 a concrete strength of 25 N/mm^2 was a normal specification, but now 35 N/mm^2 is normal and sometimes 40 N/mm^2. The proportion of pfa or ggbs that can be used to replace cement is limited in BS 8007 to 35% for pfa and 50% for ggbs.

The concrete specified in BS 8007 is grade C35A (Table 2.4). The water/cement ratio and the specified cement content are equivalent to a grade C40 mix in BS 8110. The reason for the change in label for the mix is to avoid suppliers having to add more cement to the mix simply to achieve a cube strength of 40 N/mm^2.

2.6 Loading

2.6.1 *Load arrangements*

Liqud-retaining structures are subject to loading by pressure from the retained liquid. Typical values of weights are listed in Table 2.5.

Table 2.5 Density of retained liquids

Liquid	Weight (kN/m^3)
Water	10.0
Raw sewage	11.0
Digested sludge aerobic	10.4
Digested sludge anaerobic	11.3
Sludge from vacuum filters	12.0

The designer must consider whether sections of the complete reservoir may be empty when other sections are full, and design each structural element for

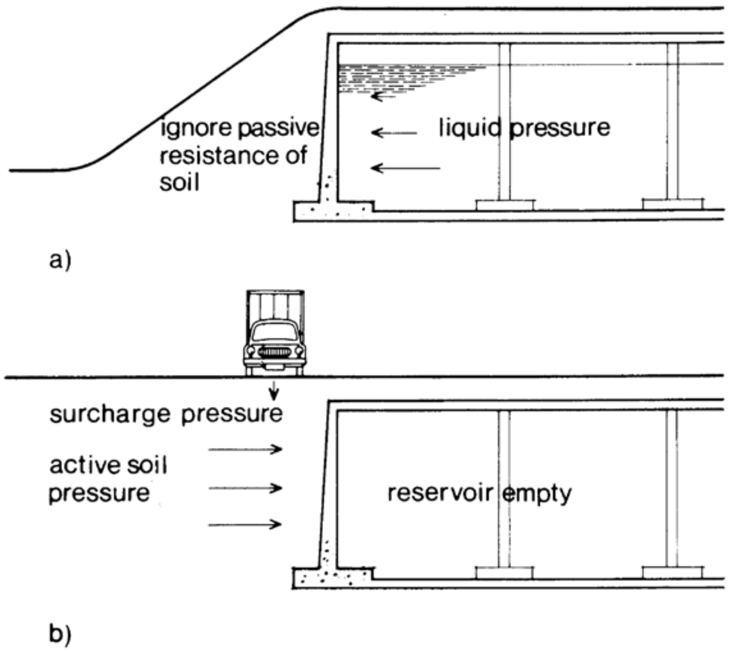


Fig. 2.11 Design loadings for external walls with soil fill
(*a*) Reservoir full
(*b*) Reservoir empty

the maximum bending moments and forces that can occur. Several loading cases may need to be considered. Internal partition walls should be designed for liquid loading on each side separately.

External reservoir walls are often required to support soil fill. The loading conditions to be considered are illustrated in Figure 2.11. When the reservoir is empty, full allowance must be made for the active soil pressure, and any surcharge pressures from vehicles. It is important to note that when designing for the condition with the reservoir full, no relief should be allowed from passive pressure of the soil fill. This is because of the differing moduli of elasticity of soil and concrete which prevent the passive resistance of the soil being developed before the concrete is fully loaded by the pressure from the contained liquid(23).

2.6.2 *Partial safety factors for loads*

When designing a structural element for the ultimate limit state, it is necessary to use partial safety factors (in conjunction with the characteristic applied loading) to provide the necessary margin against failure. The factors take account of the likely variability of the loading and the consequences of failure.

The partial safety factors appropriate for liquid retaining design are defined in BS 8007. The dead load factor is identical to that used for normal reinforced concrete design and $\gamma_f = 1.4$. As the imposed load due to a liquid is known precisely, a partial safety factor of $\gamma_f = 1.4$ may be used for loads due to retained liquid. A similar value of $\gamma_f = 1.4$ may also be used for pressures due to soil. These values are also recommended in BS 8110 for normal design. The roofs of underground structures are frequently covered with a layer of soil and hence any imposed loads due to vehicles will be distributed before reaching the structural roof slab. In these circumstances it will normally be appropriate to consider a single load case when designing the roof.

BS 8007 requires that for the ultimate limit state, liquid levels should be taken to the tops of walls (or the level of the underside of the roof slab) assuming that any outlet pipes at a lower level are blocked. This condition only applies to the ultimate limit state calculations and not to serviceability considerations.

Depending on the type of construction, and in particular, whether the roof is joined to the walls without a movement joint, any thermal expansion of the roof may cause loading on the perimeter walls. This effect is accentuated if there is effective passive pressure at the back of the walls.

2.7 Foundations

It is desirable that a liquid-retaining structure is founded on good uniform soil, so that differential settlements are avoided (Chapter 1). However, this desirable situation is not always obtainable. Variations in soil conditions must be considered, and the degree of differential settlement estimated(24). Joints may be used to allow a limited degree of articulation but, on sites with particularly non-uniform soil, it may be necessary to consider dividing the structure into completely separate sections. Alternatively, cut-and-fill techniques may be used to provide a uniform platform of material on which to found the structure.

Soils which contain bands of peat or other very soft strata may not allow normal support without very large settlements, and piled foundations are required(24).

The design of structures in areas of mining activity requires the provision of extra joints, or the division of the whole structure into smaller units. Prestressed tendons may be added to a normal reinforced concrete design to provide increased resistance to cracking when movement takes place(13).

The use of cantilever walls depends on passive resistance to sliding being provided by the foundation soil. If the soil is inundated by ground water, it may not be possible to develop the necessary soil pressure under the footing. In these circumstances, a cantilever design is not appropriate, and the overturning and sliding forces should be resisted by a system of beams balanced by the opposite wall, or by designing the wall to span horizontally if that is possible.

Walls which are designed as propped cantilevers, using the roof structure as a tie, are often considered to have no rotation at the footing (Fig. 2.12). The

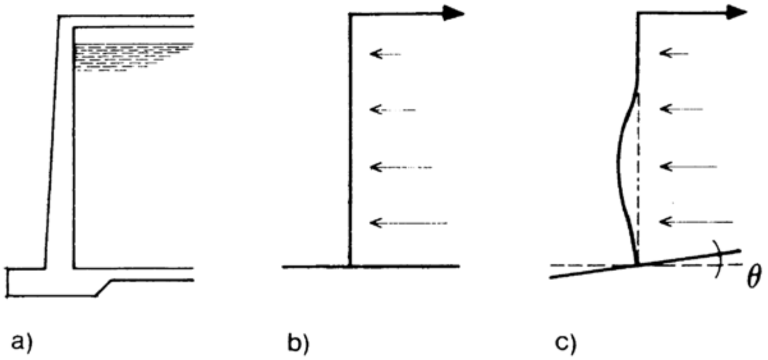


Fig. 2.12 Propped cantilever walls on a cohesive soil
(*a*) Structure
(*b*) Basic structural assumptions
(*c*) Rotation due to soil movement

strain in a cohesive soil may allow some rotation and a redistribution of forces and moments.

2.8 Flotation

An empty tank constructed in water-bearing soil will tend to move upwards in the ground, or float. This tendency must be counteracted by ensuring that the weight of the empty tank structure is greater than the uplift equal to the weight of the ground water displaced by the tank. The safety margin required

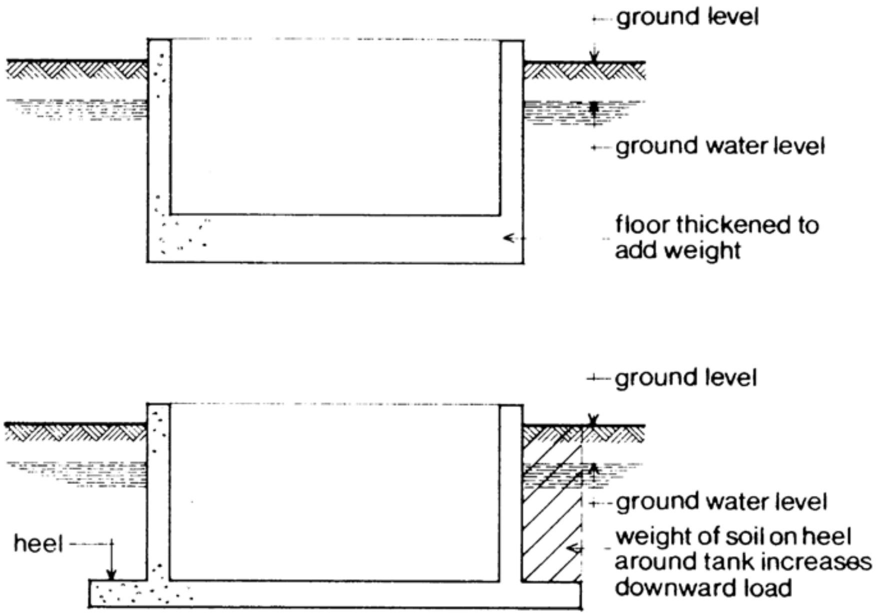


Fig. 2.13 Methods of preventing flotation
(*a*) Additional dead weight
(*b*) Provision of a heel

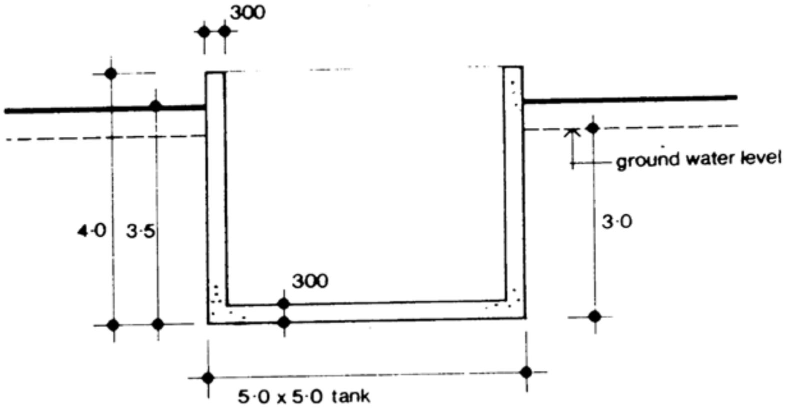
is a matter for the judgement of the designer. Dependent on the certainty with which the ground water level is known, the factor of safety may vary between 1.1 and 1.25.

The weight of the tank may be increased by thickening the floor or by providing a heel on the perimeter of the floor to mobilize extra weight from the external soil (Fig. 2.13). Whichever method is adopted, the floor must be designed against the uplift due to the ground water pressure. In calculating the weight of the soil over the heel, it is important to realize that the soil is submerged in the ground water. The effective density of the soil is therefore reduced (see calculation (*b*) below). If the floor is thickened, it is possible to construct it in two separate layers connected together by ties. This has the advantage that reduced thermal reinforcement appropriate to the upper thickness may be used.

The designer should consider conditions during construction, in addition to the final condition, and specify a construction sequence to ensure that the structure is stable at each phase of construction.

An example of the calculation is now given.

Example 2.1



A tank of overall size 5.0 × 5.0 m × 4.0 m deep is to be constructed with the underside of the floor at a level 3.5 m below ground level. The walls and floor are 300 mm thick.

Check stability against flotation if the ground water level is 0.5 m below the soil surface, and the required factor of safety against flotation is 1.15.

Weight of empty tank:
Floor $24 \times 0.3 \times 5.0 \times 5.0$ = 180
Walls $24 \times 0.3 \times 4.7 \times 4 \times 3.7$ = 500
Total weight of empty tank 680 kN

Uplift due to ground water
= $10 \times 5.0 \times 5.0 \times 3.0$ = 750 kN
Required dead weight = (factor of safety) × (uplift)
= 1.15×750 = 863 kN
∴ extra weight required = 863–680 = 183 kN

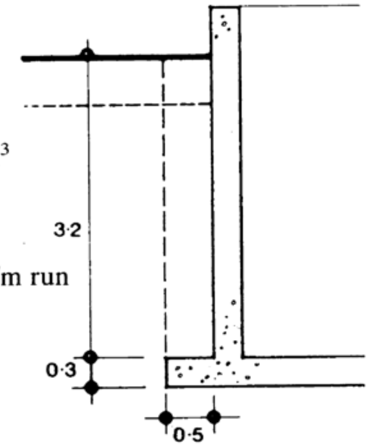
Try two possible solutions.
(*a*) Increase floor thickness to 675 mm.
Extra weight provided
= $24 \times 0.375 \times 5.0 \times 5.0$ = 225 kN

This is satisfactory.

(*b*) Provide heel on outside of floor.

Provide a heel 0.5 m wide around outside walls.
Weight of soil = 18 kN/m^3
Weight of soil submerged in ground water
= 18–10 = 8 kN/m^3
Weight of soil carried by heel if tank attempts to lift
= $8 \times 0.5 \times 3.2$ = 12.8 kN/m run
Perimeter of heel = 4×5.5 = 22 m
Total weight of soil = 12.8×22 = 282 kN

This is satisfactory.



Note: For simplicity, it is assumed that all the soil is submerged in water. This is conservative, as the upper 0.5 m is above ground water level.

3

Design of reinforced concrete

3.1 General(25,9)

The basic design philosophy of liquid-retaining structures is discussed in Chapter 2. In this chapter, detailed design methods are described to ensure compliance with the basic requirements of strength and serviceability.

In contrast with normal structural design, where strength is the basic consideration, for liquid-retaining structures it is found that serviceability considerations control the design. The procedure is therefore:

(*a*) Estimate concrete member sizes.
(*b*) Calculate the reinforcement required to limit the design crack widths to the required value.
(*c*) Check strength.
(*d*) Check other limit states.
(*e*) Repeat as necessary.

The calculation of crack widths in a member subjected to flexural loading can be carried out once the overall thickness and the quantity of reinforcement have been determined, but it is not possible to make a direct calculation. It is therefore convenient to use design tables or charts. The tables in Appendix A are arranged to enable the whole structural design to be carried out in one operation, including the checking of crack control and strength. The use and derivation of the tables is described in the Appendix.

3.2 Wall thickness

3.2.1 *Considerations*

All liquid-retaining structures include wall elements to contain the liquid, and it is necessary to commence the design by estimating the overall wall thickness in relation to the height.

The overall thickness of a wall should be no greater than necessary, as extra thickness will cause higher thermal stresses when the concrete is hardening.

The principal factors which govern the wall thickness are:

(*a*) ease of construction
(*b*) structural arrangement
(*c*) avoidance of excessive deflections
(*d*) adequate strength
(*e*) avoidance of excessive crack widths

The first estimate of minimum section thickness is conveniently made by considering (*a*), (*b*) and (*c*).

It will be found that a wall thickness of about 1/10 of the span is appropriate for a simple cantilever, and somewhat less than this for a wall which is restrained on more than one edge. Each consideration is discussed in the following sections.

Table 3.1 Approximate minimum thickness *h* (mm) of R. C. Cantilever wall subjected to water pressure

Height of wall (m)	Minimum wall thickness *h* (mm)
8	800
6	700
4	450
2	250

3.2.2 *Ease of construction*

If a wall is too thin in relation to its height, it will be difficult for the concrete to be placed in position and properly compacted. As this is a prime requirement for liquid-retaining structures, it is essential to consider the method of construction when preparing the design. It is usual to cast walls up to about 8 metres high in one operation and, to enable this to be successfully carried out, the minimum thickness of a wall over 2 metres high should be not less than 250–300 mm. Walls less than 2 metres high may have a minimum thickness of 200 mm. A wall thickness less than 200 mm is not normally possible, as the necessary four layers of reinforcement cannot be accommodated with the appropriate concrete cover on each face of the wall. The wall may taper in thickness with height in order to save materials. Setting out is facilitated if the taper is uniform over the whole height of the wall (Fig. 3.1).

3.2.3 *Structural arrangement*

Lateral pressure on a wall slab is resisted by a combination of bending moments and shear forces carrying the applied loads to the supports. The simplest situation is where the wall is a simple cantilever, with the maximum

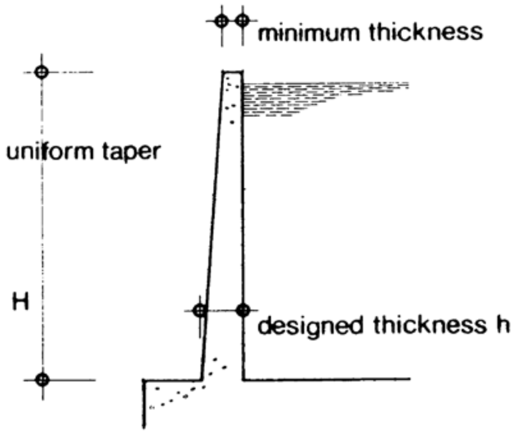


Fig. 3.1 Typical section through a wall

shear force and bending moment at the base. This situation will require the thickest wall section as the bending moment is comparatively large. The most favourable arrangement is where a wall panel is held at all four edges and may be structurally continuous along the edges. The slab spans in two directions and in each direction there may be positive and negative moments. Each of the moments will be appreciably less than in the case of the simple cantilever, and hence a thinner wall is possible with less reinforcement to control cracking. The particular structural arrangement that is appropriate for a given design will depend on the relative spans in each direction and whether movement joints are required at any of the sides of the panel.

3.2.4 *Strength in shear*

It is inconvenient to use shear reinforcement in slabs because it is difficult to fix, impedes placing of the concrete, and is inefficient in the use of steel. The wall thickness therefore should be at least sufficient to allow the ultimate shear forces to be resisted by the concrete in combination with the longitudinal reinforcement. It is, in theory, also necessary to ensure that any diagonal cracks due to shear at service loads are within the allowable limits, but in practice, other requirements will ensure that no check need be made. Suitable values of the maximum ultimate shear force on concrete in slabs are given in Table 3.2. The table is constructed from the formula given in Table 3.9 of BS 8110 which is:

$$v_c = 0.79 \times (f_{cu}/25)^{1/3} \times (100 \times A_s/(b \times d))^{1/3} \times (400/d)^{1/4}/\gamma_m$$

The steel ratio should not be taken as greater than 3
The value of effective depth should not be taken as greater than 400 mm
f_{cu} should not be taken as greater than 40 N/mm^2
γ_m is taken as 1.25

As this force varies, not only with concrete grade but also with the effective depth of the section and the reinforcement ratio in the section, some estimate of the reinforcement percentage must be made at the outset. For cantilever walls, a value of 0.5% is a reasonable starting point in the absence of previous experience.

Table 3.2 Allowable ultimate shear force in slabs (kN/m). Grade 35 concrete

Steel ratio $\frac{100A_s}{bd}$	Effective depth (mm)							
	140	190	240	330	430	630	730	920
0.17	71.3	89.6	106.8	135.6	168.4	246.7	285.9	360.3
0.25	81.1	101.9	121.5	154.2	191.5	280.6	325.1	409.8
0.50	102.1	128.4	153.0	194.3	241.3	353.5	409.6	516.3
0.75	116.9	147.0	175.2	222.4	276.2	404.7	468.9	591.0
1.00	128.7	161.8	192.8	244.8	304.0	445.4	516.1	650.5
1.50	147.3	185.2	220.7	280.2	348.0	509.9	590.8	744.6

Note: A_s is the area of steel that is fully anchored.

The maximum shear force in a cantilever occurs at the foot of the wall immediately above the base, and the shear stress in the concrete is also a maximum at this level. However, the critical level for checking the permissible shear stress is at a distance of twice the effective depth above the base level (Fig. 3.2). This point is explained in BS 8110 where a formula for an enhanced permissible design shear stress in this region is given as:

$$V_c \times 2d/a_v$$

This enhancement of the design stress is due to the proximity to a support, and can be applied at any section within a distance of twice the effective depth

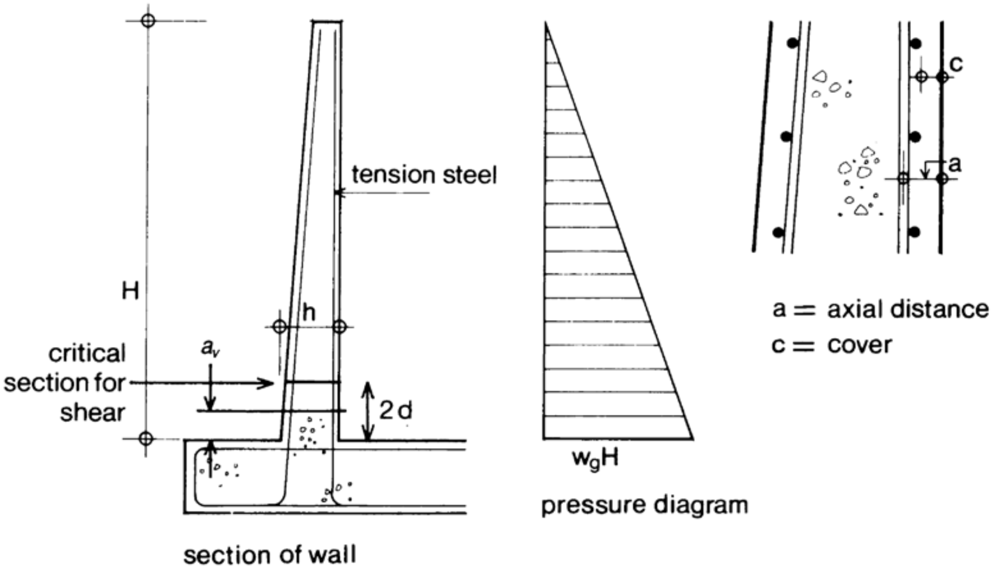


Fig. 3.2 Cantilever wall subjected to water pressure

from the face of the support. Assuming that the slab thickness is not greater than about one sixth of the span, then the section at $2d$ will be critical for design and the sections between this point and the support need not be checked.

Assume a free cantilever wall of uniform tapered section subjected to water pressure (Fig. 3.2).

H = height of liquid (m)
w_g = density of liquid (kN/m^3)
h = maximum overall thickness of section (mm)
d = maximum effective depth of section (mm)
a = depth of centre of tension steel from face of concrete (mm)
The overall thickness $h = d + a$
γ_f = partial safety factor for loads
The maximum ultimate shear force at the critical section of the cantilever is

$$V_u = \tfrac{1}{2} w_g \gamma_f (H - 2d)^2$$

The shear stress on the section $v = \dfrac{V_u}{bd}$

The distance from the face of the concrete to the centre of the tension steel a varies according to bar size and cover. Allowance should be made for any taper on the section. Assuming that the concrete cover is 40 mm and the bar size is 16 mm, the value of a is equal to $40 + 1.5 \times 16$ or about 65 mm. (Distribution reinforcement should be in the outer layer where it is more effective.) The required section thickness h may be calculated from given values of applied shear force and permissible stress to ensure that no shear reinforcement is required. An example of the calculation follows.

Example 3.1
Calculation of wall thickness for shear strength
Consider a cantilever wall of height H subject to water pressure.

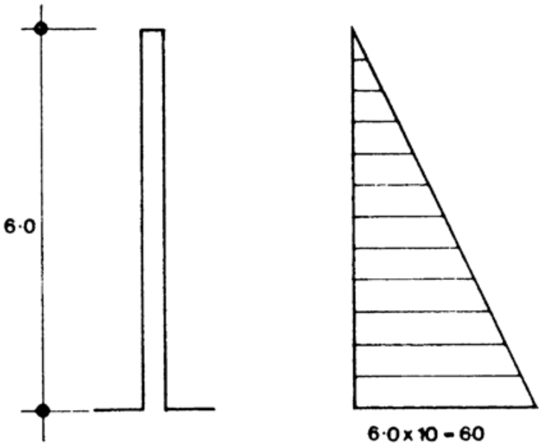
Height $H = 6.0$ m.
Density of water $= w_g = 10$ kN/m^3.
Partial safety factor $\gamma_f = 1.4$.

Assume tension reinforcement ratio

$$\frac{100A_s}{bd} = 0.5\%$$

Maximum applied ultimate shear force at base level

$$\begin{aligned} V_u &= 0.5 \times 10 \times 1.4 \times 6.0^2 \\ &= 252 \text{ kN/m} \end{aligned}$$



Assuming a wall thickness $h = 700$ (see Table 3.1), the critical section for shear will be at a level of (say) 1200 above the base, and the critical shear force will be

$$V_u = 252 \times \left(\frac{4.8}{6.0}\right)^2 = 161 \text{ kN/m}$$

Refer to Table 3.2, and for grade 35 concrete and 0.5% reinforcement ratio, the minimum effective depth required to resist an ultimate shear force of 161 kN/m may be estimated as 330 mm

and the overall wall thickness

$$h = d + a = 330 + 65 = 395 \text{ mm}$$

From shear considerations alone, the wall should have a minimum thickness of (say) 400 mm.

It is now necessary to re-check the calculation using the new value of d. It will be found that there is no significant change, and for a wall height of 6.0 m, a thickness of about 600 mm is necessary. In this case, shear is not critical.

3.2.5 *Deflection*

The lateral deflection of a cantilever wall which is proportioned according to the rules suggested in this chapter is likely to be no more than about 30 mm. A wall which is restrained by connection to a roof slab or by lateral walls will clearly deflect even less. Deflection of this magnitude will have no effect on the containment of liquid and, unless there is a roof slab supported by the wall with a sliding joint, there is no need to consider the amount of deflection. If pipes or other apparatus pass through a wall which may itself move slightly under load, the pipes must be arranged to be sufficiently flexible to allow for this movement.

Concrete codes allow members to have stiffness defined in terms of span/effective depth ratios as an alternative to calculating deflections. These values apply equally to normal and liquid-retaining structures(6). Typical values are given in Tables 3.10 to 3.12 in BS 8110. The values are based on limiting the deflections to span/250, assuming that the member is constant in depth and that the loading is uniform. In the case of a vertical cantilever wall subjected to liquid pressure, the loading will be of triangular distribution and the wall section may be tapered. If the values in BS 8110 are used as the basis for calculating the effective depth of the member, a slightly conservative design will result. Allowance may be made for the effect of the triangular load distribution by increasing the basic allowable ratio for a cantilever from 7 to 8.75. (This is based on a comparison of deflection coefficients.)

Table 3.3 is based on the recommendations of BS 8110. In Table 3.3(*b*), the ultimate moment *M* is taken at the root of a cantilever or at the point of maximum bending moment for a simply supported slab.

Table 3.3 Span/effective depth ratios for slabs up to 10 m span. (The effect of compression reinforcement has not been taken into account.)
(*a*) Basic Ratios

Condition	Ratio
Cantilever	7
Simply supported	20
Continuous	26

(*b*) Modification factors for tension reinforcement (at service stress $f_s = 250$ N/mm²)

$\frac{M}{bd^2}$	1.0	1.5	2.0	3.0	4.0	5.0	6.0
Factor	1.55	1.34	1.2	1.04	0.94	0.87	0.82

The discussion above deals with the deflection of cantilevers assuming a fixed base, but further lateral deflection may be caused by rotation of the base due to consolidation of the soil. This factor is of importance for high walls and relatively compressible ground. An estimation of the lateral deflection at the top of a wall due to base rotation may be made by considering the vertical displacements of the extremities of the foundation with the reservoir full, assuming that the wall and base are rigid, and subjected to rotation calculated from the differential soil consolidation at front and rear of the footing (Fig. 3.3).

$$\text{Rotation } \phi = \frac{a_1 - a_2}{B} \quad \text{and} \quad a_r = \phi H$$

The value must be added to the deflection due to the flexure of the wall calculated by

$$a_w = \frac{w_g H^5}{30EI}$$

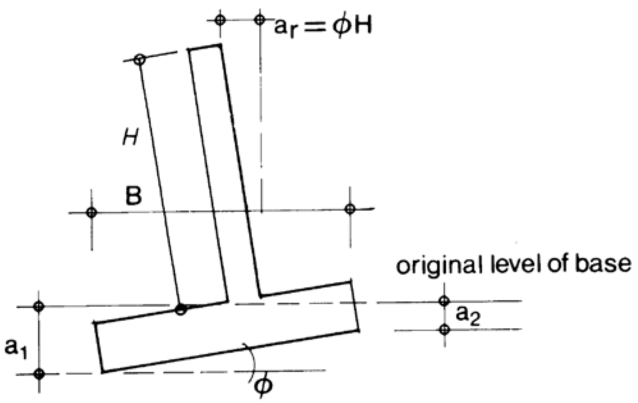


Fig. 3.3 Rotation of cantilever wall due to soil consolidation

In this formula, H may be taken to the top of the base slab.

Finally, the total deflection at the top of the wall

$$a = a_w + a_r$$

is compared with $H/250$ or any other requirement.

With a propped cantilever wall, deflection will not be critical, but the rotation of the base will alter the relation between the negative and positive moments in the wall. The moments may be calculated most easily using a computer program.

3.3 Cracking

If a reinforced concrete slab is laterally loaded, the concrete on the side of the tension reinforcement will extend and, dependent on the magnitude of loading (other factors being equal), it will eventually crack as the load is increased. At the instant that a crack forms, it will have a positive width. Further increases in load widen the cracks that have formed and increase the stress in the reinforcement (Fig. 3.4). For the same concrete section and load but with a greater quantity of reinforcement, the service stresses in the steel will be reduced, and the crack widths will be narrower.

The applied load is fixed by the structural arrangement and, using limit state design, the designer has to choose values of slab thickness, reinforcement quantity, and reinforcement service stress to ensure that the crack widths under service loads are within the appropriate values given by the class of exposure (Chapter 2), and that the ultimate load factor is satisfactory.

Although a crack width calculation may show that reinforcement service stresses as high as 280 N/mm^2 are possible with certain combinations of slab thickness and reinforcement, it is not advisable to choose these arrangements. It is suggested that an arbitrary upper limit of about 250 N/mm^2 is placed on the value of reinforcement service stress. The values given in Appendix A take account of this restriction.

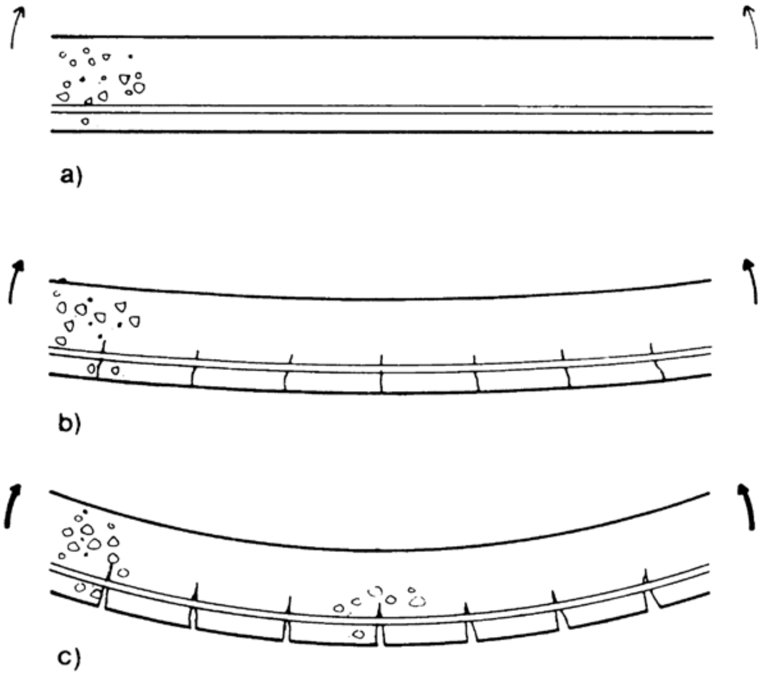


Fig. 3.4 Flexural cracking
(*a*) Concrete uncracked with low steel stress
(*b*) Fine cracks and increased steel stress
(*c*) Wide cracks and high steel stress

There is no single design that will simultaneously exactly meet all the required criteria, and a number of different solutions are possible, even for a given value of design crack width. The tables in Appendix A have been constructed to allow the designer to choose directly a section thickness and arrangement of reinforcement at a stated service stress.

The derivation of the tables is given in Appendix A. The tables assume various values of section thickness, cover, and reinforcement size and spacing. The value of reinforcement stress is then calculated for a stated value of crack width. The tables have been prepared for concrete grade 35 and steel grade 460.

The detailed methods of calculation for limit state design are considered in Sections 3.4 and 3.5. Where direct tensile forces are present in addition to flexural forces the designer should consider which force system is predominant. In a vertical wall, some horizontal tension will be present, adjacent to lateral walls. In a circular deep tank, there will be almost entirely tensile forces and no flexure towards the top of the wall. When flexural forces are predominant, the allowance for the tensile forces may be made by adding to the calculated reinforcement resisting flexure, an extra quantity calculated by reference to the service stress in the flexural steel. Where tension is predominant, crack widths can be calculated (see Section 3.6) and steel provided accordingly. If both flexure and direct tension are present to a significant degree, a calculation should be made using a modified strain diagram across the section to allow for the tensile force. The flexural crack width calculation may then be made. If a significant tensile force is present in a section, it is necessary to have reinforcement disposed in each face of the section in nearly equal quantities. Having regard to the avoidance of errors on site, it is good and sensible practice either to have equal steel arrangements in each face of a wall, or to have visibly distinct arrangements.

The precise calculation of the stress and strain diagrams for combined bending and tension results in a cubic equation of some complexity. Various designers' handbooks provide solutions using charts, and the alternative is to use a computer program. In some circumstances, it is possible in the first instance to choose a section and reinforcement by considering bending only, and then to modify the design using the formulae given in Section 3.7 in order to recalculate the strain from the depth of the neutral axis which is appropriate for the actual applied bending and tension. The results are then iterated until a satisfactory solution is obtained.

3.4 Calculation of crack widths due to flexure

The limit state of cracking is satisfied by ensuring that the maximum calculated surface width of cracks is not greater than the specified value, depending on the degree of exposure of the member (see Chapter 2). To check the surface crack width, the following procedure is necessary:

(*a*) Calculate the service bending moment.
(*b*) Calculate the depth of the neutral axis, lever arm and steel stress by elastic theory.
(*c*) Calculate the surface strain allowing for the stiffening effect of the concrete.
(*d*) Calculate the crack width.

The maximum service bending moment is calculated using characteristic loads with $\gamma_f = 1.0$.

The calculation for a slab is based on a unit width of 1 metre.

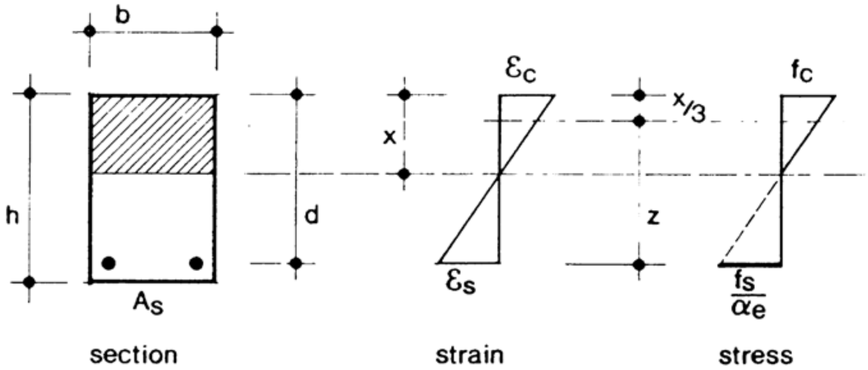


Fig. 3.5 Assumed stress and strain diagram—cracked section—elastic design

The depth of the neutral axis x is calculated (see Section 3.8.1) using the usual assumptions for modular ratio design (Fig. 3.5):

$$\frac{x}{d} = \alpha_e \rho \left(\sqrt{1 + \frac{2}{\alpha_e \rho}} - 1 \right)$$

A similar but more complex formula may be used when compression reinforcement is present

$$\rho = \frac{A_s}{bd}$$

$$\alpha_e = \text{modular ratio} = \frac{E_s}{E_c}$$

(*Note*: E_c should be taken as half the instantaneous value.)

Typical values of the short-term modulus of elasticity for concrete are given in Table 3.4, but it is usually sufficiently accurate to take a value of $\alpha_e = 15$ for all normal grades of concrete.

From x the lever arm z is found from

$$z = d - \frac{x}{3}$$

Table 3.4 Value of short-term modulus of elasticity of concrete

Concrete grade	Modulus of elasticity	Modular ratio
25	25	16
30	26	15
35	27	15
40	28	14

The tensile steel and concrete compressive stresses are then

$$f_s = \frac{M_s}{zA_s}$$

$$f_{cb} = \frac{2M_s}{zbx}$$

For the crack width formula to be valid, the compressive stress in the concrete and the tensile stress in the steel under service conditions must be less than the limiting values as follows:

Concrete: $f_{ch} \ngtr 0.45 f_{cu}$
Steel: $f_s \ngtr 0.8 f_y$

If these criteria are met, the formulae for crack width calculation may be used.

The average strain at the level surface ε_m is assessed by calculating the apparent strain (ε_1) (Fig. 3.6). This is then adjusted to take into account the stiffening effect of the concrete between cracks ε_2.

$$\varepsilon_m = \varepsilon_1 - \varepsilon_2$$

The formulae for the stiffening effect of the concrete between cracks include an assumed value of strain and therefore can only be used for particular values of design crack width.

The stiffening effect of the concrete may be assessed by deducting from the apparent strain a value obtained from the appropriate equation below.

For a limiting design surface crack width of 0.2 mm:

$$\varepsilon_2 = \frac{b_t(h-x)(a'-x)}{3E_sA_s(d-x)}$$

For a limiting design surface crack width of 0.1 mm:

$$\varepsilon_2 = \frac{1.5b_t(h-x)(a'-x)}{3E_sA_s(d-x)}$$

The value a' in these formulae is the distance from the compression face of the section to the point at which the crack width is being calculated. In the case of a slab, a' is equal to the overall depth h. The maximum width of a surface crack in a slab is along a line mid-way between two adjacent bars$^{(6)}$ (Fig. 3.7).

The calculated value of the lever arm z is subject to two restrictions. The upper surface of a slab is likely to be of less well consolidated concrete and it is therefore recommended that the value of lever arm should be limited to a value of 0.95d. This restriction corresponds with a similar recommendation

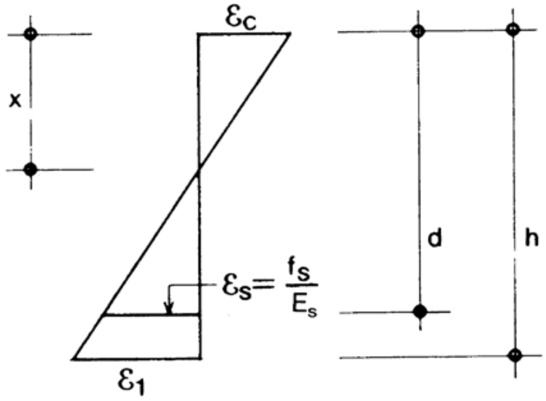


Fig. 3.6 Crack calculation—strain diagram

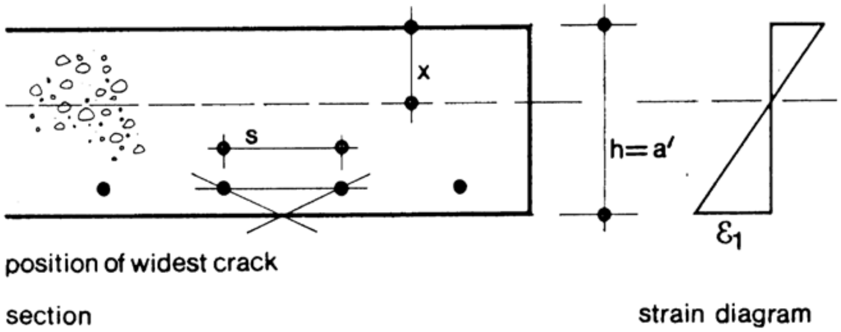


Fig. 3.7 Slab section indicating surface crack

applicable to ultimate load design. It is convenient to apply a similar restriction to a wall calculation.

A second restriction is necessary in order to limit the depth of the neutral axis. In modular ratio design, the depth of the neutral axis depends on the steel ratio. In equating the tensile force in the steel to the compressive force in the concrete, as the steel ratio increases, the depth of the neutral axis is lowered in order to keep the forces in balance. After the steel ratio is reached where there is a balanced section, i.e. the permissible steel stress and the permissible concrete stress occur together, any further increase in the steel ratio will create a section which will fail in a brittle manner. The old Codes of Practice which specify modular ratio design do not correspond with recommendations in the newer Codes based on ultimate load design, but a reasonable limit is to adopt the same restriction as in ultimate load design of $0.5d$.

The value of ε_1 represents the calculated average elastic tensile strain in the concrete (using modular ratio theory), and the second term makes allowance for the stiffening effect of the concrete between actual cracks.

$$\varepsilon_1 = \frac{(h-x)}{(d-x)} \times \frac{f_s}{E_s}$$

The design surface crack width w is obtained from the formula

$$w = \frac{3a_{cr}\varepsilon_m}{1+2\left(\dfrac{a_{cr}-c_{min}}{h-x}\right)}$$

which is stated in BS 8007 and a_{cr} is the maximum distance between the concrete surface and the surface of the nearest bar. A negative calculated value of w indicates that the section is uncracked.

The symbols used are summarized below:

ε_1 is the strain at the level considered, ignoring the stiffening effect of the concrete in the tension zone.

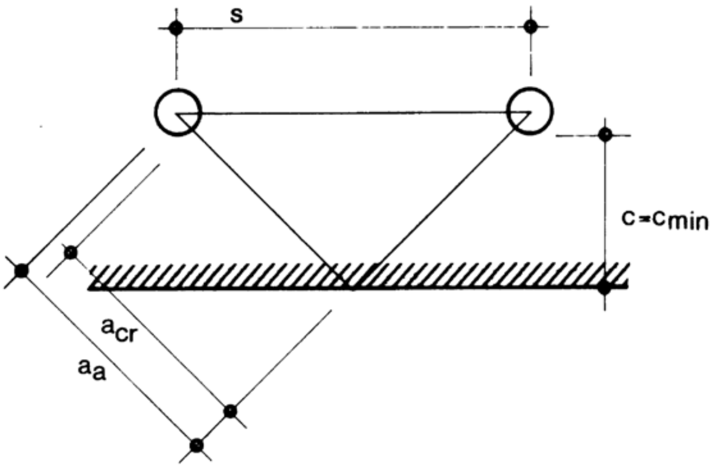


Fig. 3.8 Definition of a_{cr}

b_t is the width of the section at the centroid of the tension steel.
a' is the distance between the compression face and the point at which the crack width is being calculated.
A_s is the area of steel.
f_s is the service stress in the reinforcement.
a_{cr} is the distance between the point considered and the surface of the nearest longitudinal bar (see Fig. 3.8).
ε_m is the average strain at the level at which cracking is being considered, allowing for the stiffening effect of the concrete in the tension zone.
c_{min} is the minimum cover over the tension steel.
h is the overall depth of the member.
x is the depth of the neutral axis.
E_s is Young's modulus for steel.
E_c is Young's modulus for concrete.

An example of a crack width calculation is given below.

Example 3.2
To calculate the design crack width in a flexural member.
A wall of 300 mm overall thickness is reinforced with T16 bars at 200 mm centres.

Cover = 50 mm Concrete grade 35

Calculate the design crack width for an applied service moment of 44 kN m/m.

Reinforcement area = 1005 mm^2/m
$d = 300 - 50 - 8 = 242$ mm

$$\rho = \frac{A_s}{bd}$$

$$= \frac{1005}{1000 \times 242}$$

$$= 0.00415$$

Modular ratio:

$$E_s = 200\ \text{kN/mm}^2$$

$$E_c = \tfrac{1}{2} \times 27 = 13.5\ \text{kN/mm}^2$$

$$\alpha_e = E_s / E_c = 14.8$$

$$\alpha_e \rho = 0.0615$$

Depth of neutral axis is given by

$$\frac{x}{d} = 0.0615\left(\sqrt{1 + \frac{2}{0.0615}} - 1\right)$$

$$= 0.295 \quad \text{and} \quad \underline{x = 71.3}\ \text{mm}$$

Lever arm $z = d - \dfrac{x}{3}$

$$= 242 - 24 = \underline{218}\ \text{mm}$$

Steel tensile stress $f_s = M_s / (zA_s)$

$$f_s = 44 \times 10^6/(218 \times 1005) = 201\ \text{N/mm}^2$$

Concrete compressive stress

$$f_{cb} = \frac{2M_s}{zbx} = \frac{2 \times 44 \times 10^6}{218 \times 10^3 \times 71.3} = \underline{5.64}\ \text{N/mm}^2$$

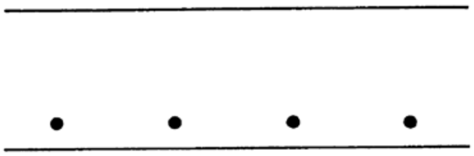
Check stress levels:

$$0.45 f_{cu} = 0.45 \times 35 \quad = 15.7$$

$$f_{cb} = 5.64 < 15.7 \quad \text{O.K.}$$

$$0.8 f_y = 0.8 \times 460 \quad = 368$$

$$f_s = 201 < 368 \quad \text{O.K.}$$



$$\varepsilon_1 = \frac{h - x}{d - x} \times \frac{f_s}{E_s}$$

$$= \frac{300 - 71.3}{242 - 71.3} \times \frac{201}{200 \times 10^3} = \underline{1.347 \times 10^{-3}}$$

Assume crack width = 0.2 mm

$$\varepsilon_2 = \frac{b_t(h - x)(h - x)}{3E_s A_s(d - x)}$$

$$= \frac{10^3(300-71.3)^2}{3\times 200\times 10^3\times 1005(242-71.3)}$$

$$= 0.508\times 10^{-3}$$

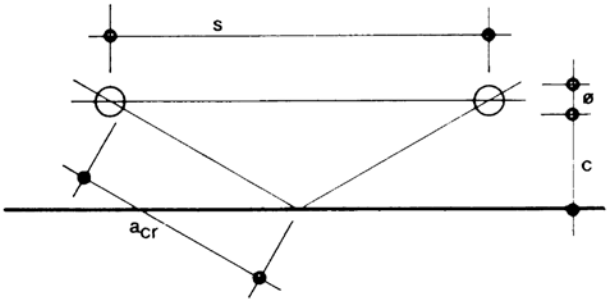
Average surface strain

$$\varepsilon_m = (1.347-0.508)\times 10^{-3} = \underline{0.839\times 10^{-3}}$$

Design surface crack width

$$w = \frac{3a_{cr}\varepsilon_m}{1+2\left(\frac{a_{cr}-c_{min}}{h-x}\right)}$$

$$a_{cr} = \sqrt{\left(\frac{s}{2}\right)^2+\left(c+\frac{\phi}{2}\right)^2}-\frac{\phi}{2}$$



$\phi = 16$

$c = 50 = c_{min}$

$s = 200$

$a_{cr} = 107.6\text{ mm}$

hence

$$w = \frac{3\times 107.6\times 0.839\times 10^{-3}}{1+2\left(\frac{107.6-50}{300-71.3}\right)} = \underline{0.18\text{ mm}}$$

Note: The result of this calculation is approximately 0.2 mm. If the result had been near to 0.1 mm it would have been necessary to repeat the calculation using the second formula for the stiffening effect of the concrete. Should the first calculation be negative then the section is uncracked. If the result is appreciably greater than 0.2 mm then a thicker section or more reinforcement is required.

3.5 Strength calculations

The analysis of the ultimate flexural strength of a section is made using formulae applicable to the design of normal structures. The partial safety factor for loads due to liquid pressure is taken as $\gamma_f = 1.4$.

The formulae for the calculation of the ultimate limit state condition are obtained from a consideration of the forces of equilibrium and the shape of the concrete stress block at failure, and the following formulae are based on the recommendations of BS 8110.

The partial safety factor for concrete is taken as $\gamma_c = 1.5$ and for steel $\gamma_s = 1.15$. After allowing for the partial safety factor for concrete, for the UK practice of testing concrete strength using cubes, and for the equivalent rectangular stress block, a value of $0.45 f_{cu}$ is used for the width of the stress block, and a depth equal to $0.9 \times$ depth to the neutral axis.

Using the rectangular stress block as illustrated in Figure 3.9, the following equations may be derived:

$$\text{Lever arm factor } z_1 = 1 - 0.45 x_1 \tag{1}$$

Force of tension = force of compression

$$\therefore \quad \frac{A_s f_y}{1.15} = 0.45 f_{cu} b \times 0.9 x_1 d \tag{2}$$

and

$$x_1 = \frac{2.15\, A_s f_y}{f_{cu} b d} \tag{3}$$

hence

$$z_1 = 1 - \frac{0.97\, A_s f_y}{f_{cu} b d} \tag{4}$$

Moment of resistance based on steel

$$M = \frac{A_s f_y}{1.15} z_1 d \tag{5}$$

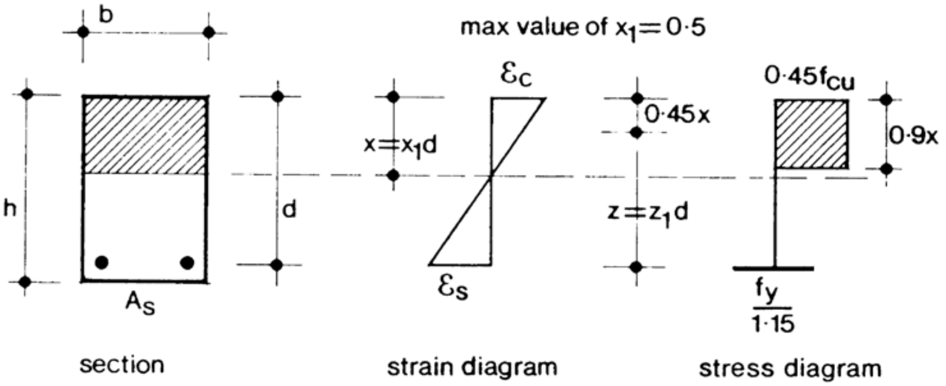


Fig. 3.9 Assumed stress diagrams—ultimate flexural limit state design

With the maximum permissible value of $x = d/2$, the maximum moment of resistance based on the concrete section is

$$M_u = 0.157 f_{cu} b d^2 \tag{6}$$

M_u represents the maximum ultimate moment which can be applied to the section without using compression reinforcement. The actual applied ultimate moment M will generally be less than M_u.

To calculate the area of reinforcement required to provide a given ultimate moment of resistance, it is convenient to rearrange Equations (1) to (6) to provide the depth of the neutral axis in terms of the applied ultimate moment and the maximum ultimate moment. The result is Equation (7) below.

$$x_1 = \left(1 - \sqrt{1 - 0.7\frac{M}{M_u}}\right) \Big/ 0.9 \qquad (7)$$

This value may be substituted in Equations (1) and (5) to calculate the required area of reinforcement.

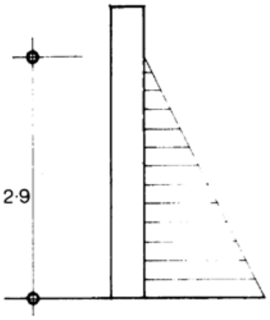
After the arrangement of reinforcement has been decided, the ultimate shear stress should be re-checked (see Section 3.2.3).

An example of a typical strength calculation follows.

Example 3.3

Strength calculation—Limit state design

Calculate the wall thickness and reinforcement required to provide the necessary load factor for a wall subjected to water pressure over a height of 2.9 m ($\gamma_f = 1.4$).



Applied ultimate moment

$$M_u = \tfrac{1}{6}(10 \times 1.4) \times 2.9^3$$
$$= 57 \text{ kNm/m}$$

Applied ultimate shear force

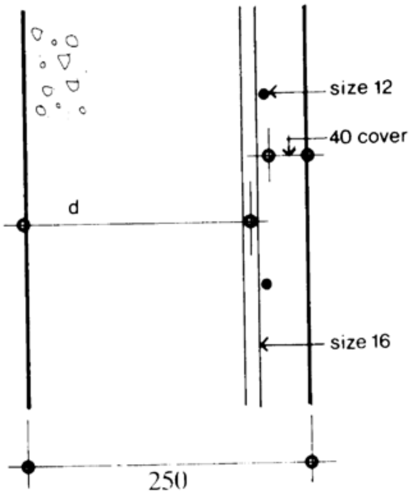
$$V_u = \tfrac{1}{2}(10 \times 1.4) \times 2.9^2$$
$$= 58.9 \text{ kN/m}$$

From Table 3.2, for an assumed steel ratio of 0.25%, the minimum required effective depth = 140 mm.

$$\text{Use wall thickness } h = 140 + \text{cover} + \frac{\phi}{2}$$

Say $\underline{h = 250 \text{ mm}}$

Overall wall thickness $h = 250$.
Assume bar size 16 mm.
Assume cover to distribution steel = 40 mm.
Assume distribution steel size 12.



$$d = 250 - 40 - 12 - 8$$
$$= 190 \text{ mm}$$

$$\text{Maximum } M_u = 0.157 f_{cu} b d^2$$
$$= 0.157 \times 35 \times 10^3 \times 190^2 \times 10^{-6}$$
$$= 198 \text{ kN m.}$$

Design ultimate moment = 57 kN m

$$\text{Neutral axis depth } x = \left[\left(1 - \sqrt{1 - 0.7\frac{M}{M_c}}\right) \Big/ 0.9\right] d$$

$$0.118 \times 190 = 22.4$$

$$z = d - \frac{x}{2} = 179 \text{ mm}$$

$$(z_{max} = 0.95d = 180 \text{ mm})$$

Area of tensile steel/metre width

$$A_s = \frac{1.15M}{f_y z}$$

$$= \frac{1.15 \times 57 \times 10^6}{460 \times 179} = \underline{796 \text{ mm}^2}$$

This is the minimum area of steel to give the specified load factor for the ultimate limit state. A suitable arrangement of reinforcement is T16 at 250 (804 from bar table) and this must be checked for crack width at working load.
Shear:

$$\text{Steel ratio } \rho = \frac{A_s}{bd} = \frac{804}{10^3 \times 190} = 0.42\%$$

Allowable shear force on concrete (Table 3.2)

$$V_u = 120 \text{ kN/m}$$

$$\text{Actual shear force} = 58.9 \text{ kN/m}$$

Satisfactory

3.6 Calculation of crack widths due to tensile forces

Structural tensile forces occur in rectangular tanks and similar structures due to the applied internal pressures, usually in combination with flexure. Pure tensile forces occur in a cylindrical tank (with a sliding joint at the base). A crack due to a tensile force is of greater significance than a crack due to flexure as the crack penetrates the full depth of the section, and is therefore more likely to allow leakage to occur.

BS 8007 recommends the calculation of crack widths due to direct tension by following a similar method of calculation as for flexural cracks. The apparent surface strain is calculated and modified for the stiffening effect due to the concrete between cracks.

The following formulae pertain to a situation where the whole section is in tension, but where there may be a modest applied service moment. The stress in each layer of steel is therefore different. In the case where there is only tension and no applied moment, the formulae may still be used but with $M = 0$ and generally equal steel in each face. The formulae apply when the ratio $M/(T \times h)$ is not greater than $0.5(1-2a_1)^2$. Figure 3.10 illustrates the section.

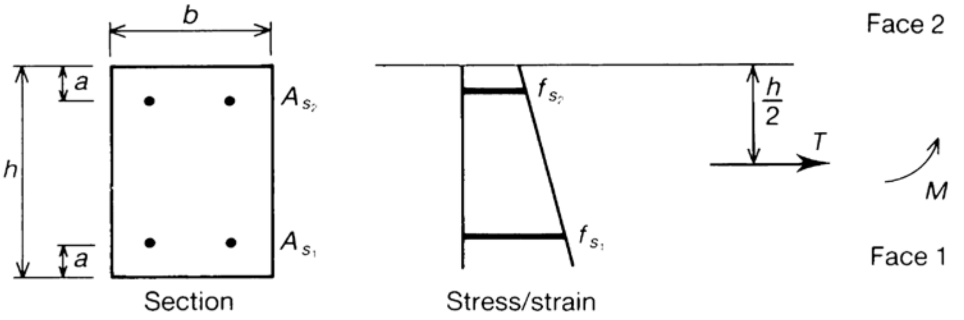


Fig. 3.10 Section with no compressive stresses

Formulae for calculation of crack widths in section subject to tensile force and moment (no compressive strain in section)

Applied service tension = T kN
Applied service moment = M kNm

Section size	Reinforcement
Breadth = b mm	Face 1 A_{s_1} mm^2
Depth = h mm	Face 2 A_{s_2} mm^2

Steel ratios	Axial cover = a mm
$\rho_1 = A_{s_1}/bh$	(both faces)
$\rho_2 = A_{s_2}/bh$	$a_1 = a/h$

Steel stresses
Face 1 f_{s_1} N/mm^2
Face 2 f_{s_2} N/mm^2

Resolve forces longitudinally

$$T = A_{s_1} \times f_{s_1} + A_{s_2} \times f_{s_2}$$

$$\frac{T}{bh} = \rho_1 \times f_{s_1} + \rho_2 \times f_{s_2} \tag{8}$$

Take moments about centre-line of section depth

$$M = (A_{s_1} \times f_{s_1} - A_{s_2} \times f_{s_2}) \times (0.5 \times h - a)$$

$$\frac{M}{bh^2} = (\rho_1 \times f_{s_1} - \rho_2 \times f_{s_2}) \times (0.5 - a_1) \tag{9}$$

From (8) $$\rho_2 f_{s_2} = \frac{T}{bh} - \rho_1 \times f_{s_1}$$

Inserting in (9) $$\frac{M}{bh^2} = \left(2 \times \rho_1 \times f_{s_1} - \frac{T}{bh}\right)(0.5 - a_1)$$

$$\therefore \quad f_{s_1} = \frac{M}{2bh^2 \rho_1 (0.5 - a_1)} + \frac{T}{2bh\rho_1} \tag{10}$$

and $$f_{s_2} = \frac{1}{\rho_2}\left(\frac{T}{bh} - \rho_1 f_{s_1}\right)$$

The strain associated with f_{s_1},

$$e_{s_1} = f_{s_1}/E_s$$

The strain gradient across the section

$$e_g = \frac{f_{s_1} - f_{s_2}}{(h - 2a) \times E_s}$$

The surface strain

$$e_1 = e_{s_1} + e_g \times a$$

Factor due to stiffening effect of concrete between cracks (assuming 0.2 mm crack width)

$$e_2 = 2 \times b \times h/(3 \times E_s \times A_s)$$

Average surface strain

$$e_m = e_1 - e_2$$

Design surface crack width

$$w = 3 \times a_{cr} \times e_m$$

where a_{cr} is the distance from the point mid-way between two bars at the surface to the surface of the nearest bar (see Fig. 3.8).

As with flexural crack width calculation, the factor for the stiffening effect of the concrete depends on the design crack width, and the appropriate formula for a design crack width of 0.1 mm is $e_2 = bh/(E_s \times A_s)$.

Examples of crack width calculation with applied direct tension and moment (tension across the whole section) follow.

Example 3.4
Example of calculation of crack width for section under direct tension
The calculation is prepared according to BS 8007.

Section properties

$h = 300$ mm	Reinforcement provided
$b = 1000$ mm	T16 at 200 each face
$c = 40$ mm	$A_s = 1010 \times 2 = 2020$ mm^2/m
$E_s = 200$ kN/mm^2	

Direct tension = 440 kN/m (characteristic value)

Apparent strain $e_1 = T/A_s \cdot E_s$
$= 440 \times 10^3/(2020 \times 200 \times 10^3)$
$= 1.09 \times 10^{-3}$

Required design crack width = 0.2 mm

Stiffening effect of concrete for 0.2 mm crack width

$$e_2 = 2bh/3E_sA_s$$
$$= 2 \times 10^3 \times 300/(3 \times 200 \times 10^3 \times 2020)$$
$$= 0.5 \times 10^{-3}$$

Average strain $e_m = e_1 - e_2 = (1.09 - 0.5) \times 10^{-3}$
$= 0.59 \times 10^{-3}$

Distance to point considered $= a_{cr} = (\sqrt{100^2 + 48^2}) - 8$
$= 111 - 8 = 103$ mm

Crack width $w = 3a_{cr}e_m$
$= 3 \times 103 \times 0.59 \times 10^{-3}$
$= 0.18$ mm

Satisfactory

Note: If the calculation provides an unsatisfactory crack width, then the reinforcement must be increased.

Example 3.5
Example of calculation of crack width for section under direct tension and flexure
The calculation is prepared according to BS 8007.

Section properties

$h = 250$ mm (to fit into structural arrangement)
$b = 1000$ mm
$a = 50$
$a_1 = 50/250 = 0.2$

Applied forces (characteristic)

$$T = 301 \text{ kN/m}$$

$$M = 12.1 \text{ kNm/m}$$

$$\frac{M}{Th} = \frac{12.1 \times 10^3}{301 \times 250} = 0.16$$

As this is less than 0.18 (from $\frac{1}{2}(1-2a_1)^2$), tension exists over the whole section.

Assume that the probable maximum tensile stress is 250 N/mm^2. With equal steel in each face,

$$A_s = \frac{301 \times 10^3}{250} = 1204 \text{ mm}^2/\text{m}.$$

If this area is provided in each face, sufficient steel is available to resist the tension T in only one face. This is conservative.

Assume T16 at 200 in each face ($A_s = 1005$ EF)

$$\text{steel ratio } \rho_c = A_s/\text{bh} = 1005/(1000 \times 250)$$
$$= 0.004$$

$$f_{s_1} = \frac{M}{2bh^2\rho_1(0.5-a_1)} + \frac{T}{2bh\rho_1}$$

$$= \frac{12.1 \times 10^6}{2 \times 10^3 \times 250^2 \times 0.004 \times 0.3} + \frac{301 \times 10^3}{2 \times 10^3 \times 250 \times 0.004}$$

$$= 80 + 151 = 231 \text{ N/mm}^2$$

$$f_{s_2} = \frac{1}{0.004}\left(\frac{301 \times 10^3}{10^3 \times 250} - 0.004 \times 231\right) = 70 \text{ N/mm}^2$$

Strain ε_{s_1} (associated with f_{s_1})

$$= \frac{f_{s_1}}{E_s} = \frac{231}{200} \times 10^{-3}$$

$$= 1.16 \times 10^{-3}$$

Strain gradient

$$e_g = \frac{f_{s_1} - f_{s_2}}{(h-2a)E_s}$$

$$= \frac{231 - 70}{(250 - 100) \times 200 \times 10^3}$$

$$= \frac{161}{150 \times 200} \times 10^{-3}$$

$$= 5.367 \times 10^{-6}$$

Surface strain
$$e_1 = e_{s_1} + e_g \cdot a$$
$$= 1.16 \times 10^{-3} + 5.367 \times 10^{-6} \times 50$$
$$= (1.16 + 0.27) \times 10^{-3}$$
$$= 1.43 \times 10^{-3}$$

Stiffness factor for concrete between cracks

$$e_2 = \frac{2bh}{3E_sA_s}$$
$$= \frac{2 \times 10^3 \times 250}{3 \times 200 \times 10^3 \times 1005}$$
$$= 0.83 \times 10^{-3}$$
$$\therefore \quad \varepsilon_m = \varepsilon_1 - \varepsilon_2 = (1.43 - 0.83) \times 10^{-3}$$
$$= 0.60 \times 10^{-3}$$

Crack width
$$w = 3a_{cr}\varepsilon_m$$
$$a_{cr} = 104 \text{ mm}$$
$$\therefore \quad w = 3 \times 104 \times 0.60 \times 10^{-3}$$
$$= 0.19 \text{ mm}$$

Satisfactory

Note: It will generally be surprising if the required answer is obtained at the first attempt and some iteration will be required.

3.7 Calculation of crack widths due to combined tension and bending (compression present)

3.7.1 *Defining the problem*

Some judgment is usually required when estimating crack widths due to the effects of direct tension combined with bending. The solution of the equations for a section under bending forces is straightforward as demonstrated in Section 3.4. The depth of the neutral axis can be calculated without difficulty. However, when tensile force is added to a section in bending, the position of the neutral axis changes so that a smaller fraction of the concrete section is in compression. As yet more tension is applied, the neutral axis will move outside the section, and the whole section will be in tension (see Section 3.6). The formulae based on the modular ratio method of elastic design for bending combined with tension are cubic in form and a direct design is not possible.

The most satisfactory approach is to consider the relation between the applied bending moment and the applied tensile force. The ratio M/T gives the value of the necessary eccentricity of a tensile force to produce the bending moment. A large value of M/T in relation to the section thickness indicates that the bending moment is predominant. A small value of M/T indicates that tension predominates (Fig. 3.11). If one of the applied forces is small the simplest design approach is to prepare a design for the predominant force, and then to modify it by approximate methods.

When the tensile applied force is not too large the following equations may be used. It is convenient to prepare a design for bending only (possibly by reference to the tables in Appendix A) and then to modify it by adding a modest amount of reinforcement. The equations may then be used to check the allowable values of applied loads. It is not possible to use the formulae without assuming a concrete section together with a quantity of reinforcement. In many practical cases, the neutral axis will be found to lie outside the section and the method described in Section 3.6 should be used.

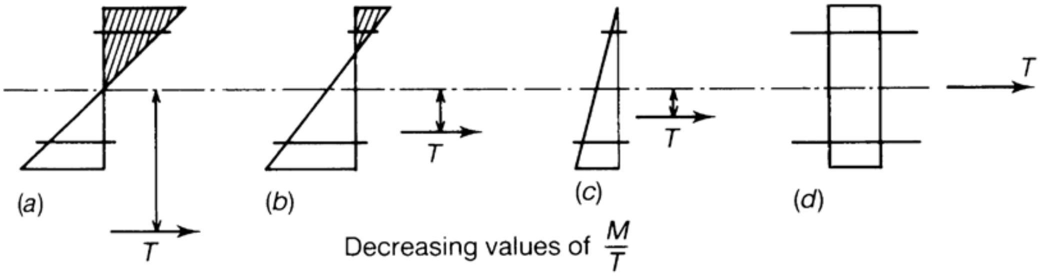


Fig. 3.11 Section with compressive stress

3.7.2 *Formulae*

Formulae for section subject to applied tension and bending when both tensile and compressive stresses occur across the section.

$$-\frac{T}{bhf_c}=\frac{1}{2}\frac{x}{h}+\rho_c(\alpha_e-1)\left(1-\frac{a}{h}\cdot\frac{h}{x}\right)-\rho_t\alpha_e\left(\frac{h}{x}-\frac{a}{h}\cdot\frac{h}{x}-1\right) \qquad (11)$$

$$\frac{M}{bh^2f_c}=\frac{1}{2}\frac{x}{h}\left(\frac{1}{2}-\frac{1}{3}\frac{x}{h}\right)+\rho_c(\alpha_e-1)\left(1-\frac{a}{h}\cdot\frac{h}{x}\right)\left(\frac{1}{2}-\frac{a}{h}\right)$$

$$+\rho_t\alpha_e\left(\frac{h}{x}-\frac{a}{h}\cdot\frac{h}{x}-1\right)\left(\frac{1}{2}-\frac{a}{h}\right). \qquad (12)$$

Symbols:

T	Applied tensile force (kN)
M	Applied service bending moment (kNm)
b	Width of section (usually 1000) (mm)
h	Overall depth of section (mm)

x	Depth of neutral axis (mm)
f_c	Maximum stress in concrete (N/mm^2)
a	Axial cover to reinforcement (mm)
ρ_t	Tension steel ratio $= A_{st}/bh$
ρ_c	Compression steel ratio $= A_{sc}/bh$
a_e	Modular ratio (can be taken as 15)
A_{sc}	Area of compression steel
A_{st}	Area of tension steel

Note: All ratios in the formulae are related to the overall depth of the section.

The formulae are most easily handled using a small computer, and may be simplified if the area of steel in each face of the section is equal. The sequence of calculation is:

(*a*) Assume a section thickness.
(*b*) Assume a steel ratio in tension and compression.
(*c*) Determine the axial cover to the steel.
(*d*) Determine the permissible concrete compressive stress.
(*e*) Insert trial values of x until the required values of T and M are obtained (it will not, in general, be possible to satisfy both conditions simultaneously)

When a satisfactory solution has been obtained, the tensile steel stress may be checked from

$$f_{st} = \alpha_e f_c(d-x)|x$$

The calculation for crack width then follows the sequence given in Section 3.4.

Example of the calculation

Example 3.6
Combined bending and tension – compression on one face of section

Design a section for applied forces (characteristic) of:

Tensile force = 78 kN/m
Moment = 57 kNm/m

Try section thickness $h = 250$ mm $d = 200$ mm
Axial cover $a = 50$ mm $(a_1 = a/h = 0.2)$

Area of steel/face to resist tensile force only at a stress of (say) 200 N/mm^2 = 1100 mm^2

Try T16 at 150 mm (each face)
$A_s = 1340$ (each face) $\rho_c = 1340/(10^3 \times 250) = 0.00536$

To estimate the depth of the neutral axis inspiration or a computer is necessary.

A trial value of $x/h = x_1$ might be between 0.2 (depth of compression steel) and 0.5 (maximum value with no tensile force).

$$\text{Take } x_1 = 0.25$$

Substitute in (11):

$$-\frac{T}{bhf_c} = 0.5 \times 0.25 + 0.00536 \times 14 \times (1 - 0.2/0.25)$$

$$- 0.00536 \times 15 \times \left(\frac{1}{0.25} - 0.2/0.25 - 1\right)$$

$$= 0.125 + 0.015 - 0.177$$

$$= -0.037$$

$$\therefore \quad f_c = \frac{78 \times 10^3}{10^3 \times 250 \times 0.037} = 8.43 \text{ N/mm}^2$$

Substitute in Equation (12):

$$\frac{M}{bh^2 f_c} = 0.5 \times 0.25 \times (0.5 - 0.25/3) + 0.00536 \times 14 \times (1 - 0.2/0.25)(0.5 - 0.2)$$

$$+ 0.00536 \times 15 \times \left(\frac{1}{0.25} - 0.2/0.25 - 1\right)(0.5 - 0.2)$$

$$= 0.125 \times 0.417 + 0.015 \times 0.3 + 0.177 \times 0.3$$

$$= 0.052 + 0.0045 + 0.053$$

$$= 0.1095$$

Use the same f_c as in Equation (11) and check that the calculated M is at least as great as the applied M.

$$M_{\text{calc}} = 0.1095 \times 10^3 \times 250^2 \times 8.43 \times 10^{-6}$$

$$= 57.7 \text{ kNm/m}$$

$$\text{Applied } M = 57.0$$

Satisfactory

The procedure to check the crack width follows in a similar manner to that described in previous sections, using the maximum steel stress

$$f_{st} = \alpha_e f_c (d - x)/x$$

$$= 15 \times 8.43(200 - 62.5)/62.5$$

$$= 278 \text{ N/mm}^2$$

The resulting crack width is 0.28 mm. A further calculation is necessary in order to reduce this to 0.2, using more reinforcement and/or a thicker section.

3.8 Limiting stress design

Crack widths may also be controlled by limiting the stress in the tensile reinforcement under service conditions to the values given in Table 3.5. This method of satisfying the limit state crack control requirements is sometimes known as the 'deemed to satisfy' method of design, in that use of this procedure is deemed to satisfy the crack width limitation requirement.

The calculations are prepared using the elastic modular ratio method of design. For an accurate analysis, the depth of the neutral axis must be calculated for the applied direct tensile force and bending moment, but for many structures it will be safe, and sufficiently accurate, to consider separately the effects for each applied force. The calculated quantities of reinforcement are then added together. The necessary formulae are given in Sections 3.8.1 and 3.8.2, and an example of a design is given below (Example 3.7).

Table 3.5 Allowable steel stresses in direct or flexural tension for serviceability limit states
BS8007(10)

Design crack width (mm)	Allowable stress (N/mm^2)	
	Plain bars	Deformed bars
0.1	85	100
0.2	115	130

3.8.1 *Flexural reinforcement*

Figure 3.12 illustrates the assumptions made using the elastic modular ratio method of design with a cracked section and no tensile force in the concrete.

The depth of the neutral axis depends on the steel ratio ρ and may be obtained by equating the force of tension in the steel with the force of compression in the concrete:

$$f_s A_s = 0.5 f_{cb} b x \tag{13}$$

From the strain diagram:

$$\frac{f_s / E_s}{d-x} = \frac{f_{cb} / E_c}{x} \tag{14}$$

From (13)

$$f_s / f_{cb} = \frac{0.5bx}{A_s}$$

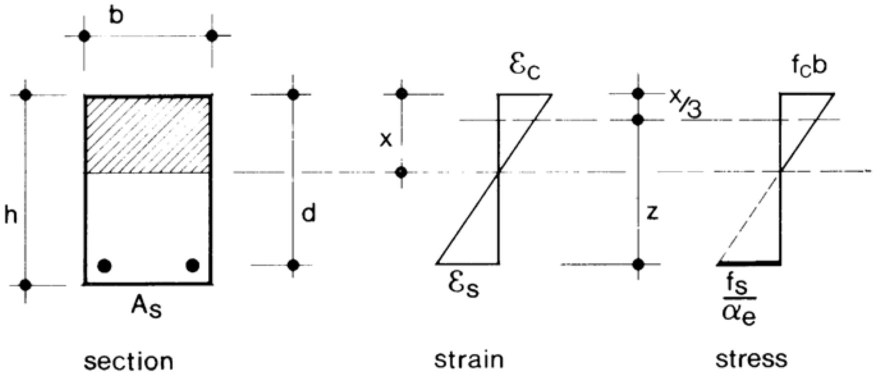


Fig. 3.12 Assumed stress and strain diagrams—cracked section—elastic design

From (14)

$$f_s/f_{cb} = \frac{E_s}{E_c} \cdot \frac{d-x}{x} = \frac{\alpha_e(d-x)}{x}$$

$$\therefore \quad \frac{0.5bx}{A_s} = \frac{\alpha_e(d-x)}{x}$$

or

$$0.5bx^2 = \alpha_e A_s(d-x)$$

Writing $\rho = \dfrac{A_s}{bd}$ and $x = x_1 d$

$$0.5x_1^2 bd^2 = \alpha_e \rho bd^2(1-x_1)$$

or

$$x_1^2 = 2\alpha_e\rho(1-x_1)$$

and

$$x_1 = \alpha_e\rho\left(\sqrt{1+\frac{2}{\alpha_e\rho}}-1\right) \tag{15}$$

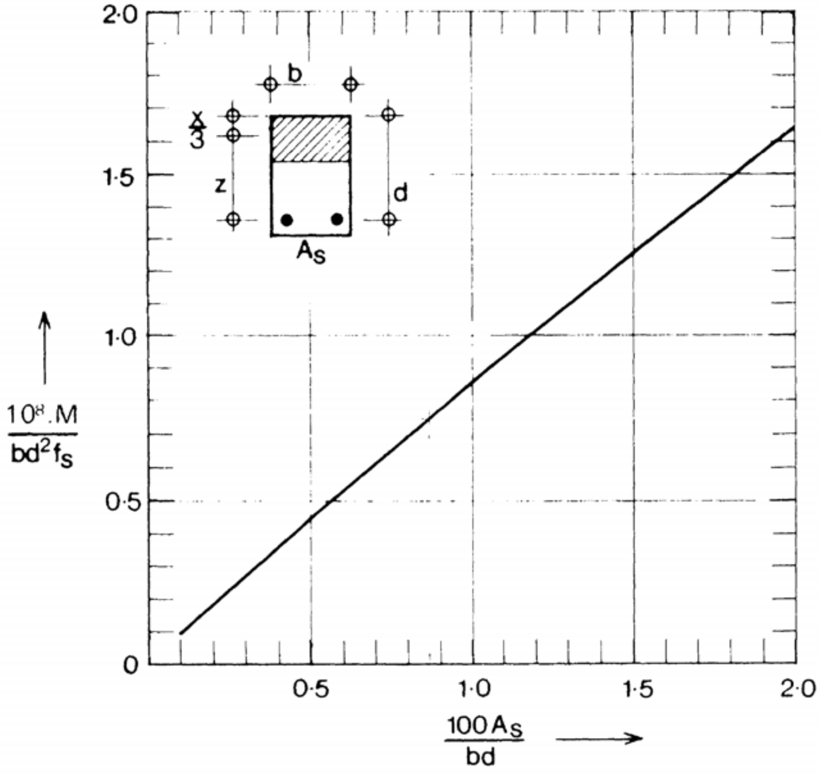


Fig. 3.13 Elastic design chart—cracked section (*b* and *d* in mm)
f_{st} = stress in tension reinforcement (N/mm^2)
M = applied moment (kN m)
α_e = 15 = modular ratio
A_s = area of tension steel (mm^2)

The value of the modular ratio $\alpha_e = E_s/E_c$ may be taken as 15.

$$\therefore \quad x_1 = 15\rho\left(\sqrt{1+\frac{2}{15\rho}}-1\right) \tag{16}$$

Solving equation (16) gives x_1.
Lever arm factor $z_1 = 1 - x_1/3$.

The moment of resistance of the section based on the steel stress is given by

$$M_r = A_s f_s z$$

or

$$\frac{M_r}{bd^2 f_s} = \rho z_1 = \rho(1 - x_1/3) \tag{17}$$

This relation is plotted in Figure 3.13 (using steel percentage)

3.8.2 *Tension reinforcement*

Assuming equal steel in each face of the section, the area of steel required to resist a structural tensile service load of T at a service stress of f_s is given by

$$A_s = \frac{T}{2f_s} \tag{18}$$

The value of f_s used in Equation (18) must be identical to the value used in Equation (17).

Example 3.7
Limiting stress design
Calculate the necessary reinforcement in a wall panel subject to a bending moment of 35 kNm/m together with a direct tensile force of 50 kN/m.

Wall thickness $h = 250$ mm
Concrete grade 35
Limiting crack width = 0.1 mm
Cover $c = 52$ mm
Steel $f_y = 460$ N/mm^2

Allowable tensile stress in steel = 100 N/mm^2

$$\text{Effective depth } d = 250 - 52 - \frac{\phi}{2} \text{ (assume bar size 16 mm)}$$

$$= 250 - 52 - 8$$

$$= 190 \text{ mm}$$

$$\frac{M \times 10^8}{f_{st} bd^2} = \frac{35 \times 10^8}{100 \times 10^3 \times 190^2} = 0.97$$

From graph $\rho = 1.14$

Area of tensile steel to resist bending moment

$$A_s = \frac{1.14 \times 10^3 \times 190}{100}$$

$$= 2160 \text{ mm}^2$$

Area of steel to resist direct tensile force

$$= \frac{T}{f_{st}} = \frac{50 \times 10^3}{100} = 500 \text{ mm}^2$$

∴ Total area of tensile steel in face of section resisting tension due to bending moment

$$A_1 = 2160 + \tfrac{1}{2} \times 500$$

$$= \underline{2410 \text{ mm}^2}$$

On opposite face, a minimum of

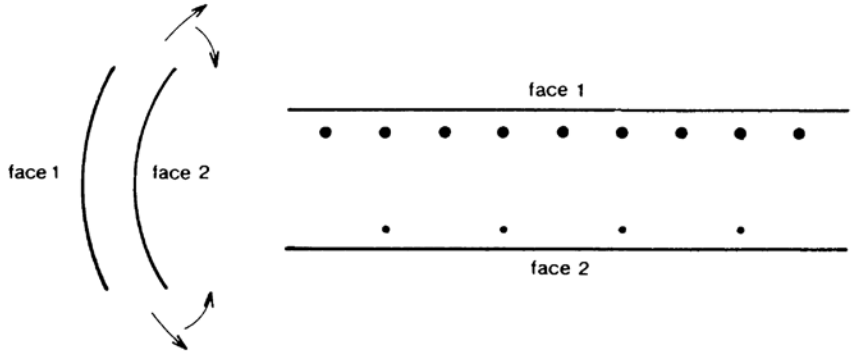
$$A_2 = \tfrac{1}{2} \times 500 = 250 \text{ mm}^2$$

Minimum steel required to resist early thermal movement is greater than this value—say 0.15% each face.

$$\therefore \quad A_2 = 0.15\% \times 1000 \times 250$$

$$= \underline{375 \text{ mm}^2}$$

Face 1 use T20 at 125 (2510)
Face 2 use T12 at 250 (452)



3.9 Design for no cracking

The calculation of crack widths under flexural loading is a relatively recent innovation. Previously it was usual to design on the theoretical basis of 'no cracking' under service loads, assuming an uncracked concrete section and limiting the tensile stress in the concrete. The method was in use for many years, but is not as economical as the limit state method previously described. Concrete sections designed by this method tend to be thick and have relatively large amounts of reinforcement. In the superceded BS 5337(10) this design method was referred to as the Alternative Method of Design. The fundamental assumptions are:

(1) In calculations relating to resistance to cracking, the concrete is assumed to be capable of resisting a limited tensile stress and the whole section, including cover to the reinforcement, is taken into account.
(2) In strength calculations, it is assumed that the concrete has no tensile strength. The necessary formulae may be developed from elastic theory.

3.9.1 *Resistance to cracking*

The section to be analysed is shown in Figure 3.14. The depth of the neutral axis x is determined by equating the forces of tension in the concrete and steel with the force of compression in the concrete.

Tensile force in the steel $= A_s f_s$
Tensile force in the concrete $= \frac{1}{2} f_{ct} b(h-x)$
Compressive force in the concrete $= \frac{1}{2} f_c bx$

$$\therefore \quad A_s f_s + \tfrac{1}{2} f_{ct} b(h-x) = \tfrac{1}{2} f_c bx \tag{19}$$

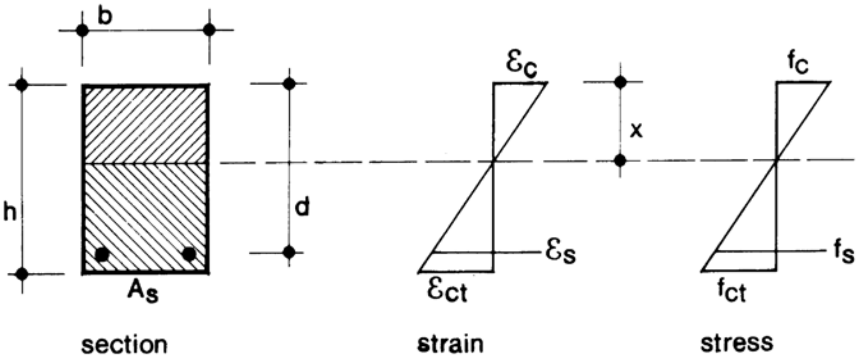


Fig. 3.14 Assumed stress and strain diagrams—uncracked section—elastic design

Also from the geometry of the strain diagram and putting the modular ratio $\alpha_e = E_s / E_c$

$$\frac{\varepsilon_c}{x} = \frac{\varepsilon_s}{d-x} = \frac{\varepsilon_{ct}}{h-x}$$

and hence

$$\frac{f_c}{x} = \frac{f_s}{\alpha_e(d-x)} = \frac{f_{ct}}{h-x} \tag{20}$$

Substituting in (19)

$$\frac{A_s f_{ct} \alpha_e (d-x)}{(h-x)} + \tfrac{1}{2} f_{ct} b(h-x) = \frac{\frac{1}{2} f_{ct} bx^2}{(h-x)}$$

or

$$A_s \alpha_e (d-x) + \tfrac{1}{2} b(h-x)^2 = \tfrac{1}{2} bx^2 \tag{21}$$

If

$$\rho_c = \frac{A_s}{bh} \tag{22}$$

$x = x_1 h$ and $d = d_1 h$

From (21) and (22) $$x_1 = \frac{0.5 + \rho_c \alpha_e d_1}{1 + \rho_c \alpha_e} \tag{23}$$

The value of the modular ratio α_e may either be taken to allow for the concrete which is displaced by the reinforcement or this small difference may be ignored. A value of $\alpha_e = 15$ is sufficiently accurate for most conditions.

The applied moment may be equated to the moment of the tensile forces in the steel and concrete about the centre of gravity of the compressive force in the concrete. This is at a distance $x/3$ from the compression face of the section. Hence

$$M = A_s f_s\left(d - \frac{x}{3}\right) + \tfrac{1}{2} f_{ct} b(h-x)\tfrac{2}{3}h \tag{24}$$

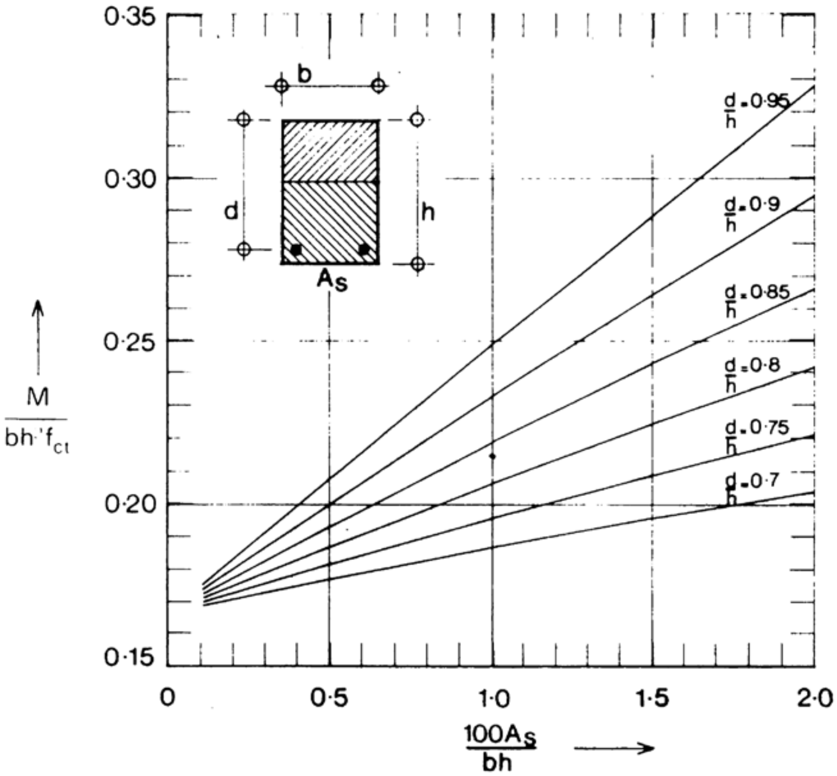


Fig. 3.15 Elastic design chart for uncracked section (*b*, *d* and *h* in mm)
f_{ct} = tensile stress in concrete (N/mm²)
M = applied moment (kN m)
α_e = 15 = modular ratio
A_s = area of tension steel (mm²)

From (20), (22) and (24),

$$M = \rho_c bhf_{ct}\frac{\alpha_e(d-x)}{h-x}\left(d-\frac{x}{3}\right)+\tfrac{1}{3}f_{ct}b(h-x)h$$

or

$$\frac{M}{bh^2f_{ct}} = \frac{(1-x_1)^2+\rho_c\alpha_e(d_1-x_1)(3d_1-x_1)}{3(1-x_1)} \tag{25}$$

Values of the moment factor from Equation (25) are plotted in Figure 3.15 (using steel percentage).

The permissible values of tensile stress in the concrete are given in Table 3.6.

Table 3.6 Permissible concrete stresses in calculations relating to the resistance to cracking for reinforced concrete

	Permissible concrete stresses in N/mm^2	
	Tension	
Concrete grade	Direct	Due to bending
Grade 35	1.55	2.18
Grade 30	1.44	2.02
Grade 25	1.31	1.84

3.9.2 *Strength*

The calculation to determine the strength of the section is made by elastic theory at service loads, assuming that the concrete has cracked in tension. This assumption is at variance with the assumption made in Section 3.9.1 for controlling cracking and can only be substantiated on the basis that *if* the section cracks, the reinforcement will still be adequate to prevent failure.

The equations used are identical with those used in Section 3.8. The allowable stresses in the reinforcement depend on the Code of Practice being followed, and suitable values are given in Table 3.7.

3.9.3 *Section thickness*

The section thickness may be calculated by considering the moment of resistance of an *uncracked* section assisted by an assumed quantity of reinforcement. The moment of resistance of such a section depends on the overall thickness, the ratio of effective depth to overall depth, and the permissible tensile stress in the concrete. Section 3.9.1 provides the theory and Figure 3.15 assists with the initial assessment of the section thickness. The quantity of reinforcement may be assumed to be 0.5% in order to arrive at a section thickness, and checked after the steel required has been more accurately calculated.

Table 3.7 Permissible steel stresses in strength calculations

Condition	Class of exposure	Permissible stress in N/mm^2 (Deformed bars)
Direct tension	A	100
Flexural tension		
Shear	B	130
Compression	A and B	140

Note: These values are in accordance with BS 5337 (now superceded): Class A is equivalent to 0.1 mm design crack width, Class B is equivalent to 0.2 mm design crack width.

Table 3.8 Values of factor K (N/mm^2) in formula $M_r = Kbh^2 f_{ct}$. Uncracked section $\alpha_e = 15$

$\frac{100A_s}{bh}$	0	0.5	0.75	1.00	1.25	1.50
$\frac{d}{h}$						
0.75	0.167	0.182	0.189	0.196	0.202	0.209
0.80	0.167	0.187	0.197	0.206	0.216	0.224
0.85	0.167	0.193	0.206	0.219	0.231	0.243
0.90	0.167	0.200	0.217	0.233	0.249	0.264

The moment of resistance is given by

$$M_r = Kbh^2 f_{ct}$$

where K has a value taken from Table 3.8, and the applied moments are calculated using the service loads. The section thickness is determined to ensure that the moment of resistance of the section is greater than the applied moment.

3.9.4 *Calculation of reinforcement*

The choice of the section thickness should ensure that there are no flexural cracks in the section under service loads, but the quantity of reinforcement is decided by assuming a cracked section as shown in Figure 3.12 and providing steel at a low stress to resist the tension due to flexural action. The permissible stresses are given in Table 3.7.

3.9.5 *Combined flexure and tension*

When a section is subjected to both direct tension and flexural action, some adjustment has to be made as the permissible stresses for each type of action are different. The calculations are conveniently carried out using an equivalent transformed section, i.e. considering the whole section in terms of concrete with the steel contributing at a ratio of α_e times its own area.

The maximum tensile stress on the whole transformed section due to the applied moment is

$$f_m = \frac{M(d-hx)}{I_e}$$

and the stress due to the direct force is

$$f_d = \frac{F}{bh}$$

The section should be proportioned such that

$$\frac{f_m}{f_{mp}} + \frac{f_d}{f_{dp}} \leqslant 1.0$$

where h_x is the depth to the neutral axis
I_e is the second moment of area of the transformed section
f_{mp} is the permissible stress due to bending
f_{dp} is the permissible stress due to direct tension

A typical calculation is given in Example 3.8.

Example 3.8
Design for no cracking—bending and tension

Design a section to resist an applied bending moment of $M = 90$ kN m/m in addition to a direct force $F = 75$ kN/m. Grade 35 concrete. High-yield steel. Design crack width = 0.2 mm, i.e. class B exposure.

(1) *Section thickness*

Assume 0.5% reinforcement and $\frac{d}{h} = 0.85$. $\qquad f_{ct} = 2.32$

If only the moment applied, from Table 3.8, $K = 0.193$

$$\therefore \quad M = Kbh^2 f_{ct}$$

$$\therefore \quad 90 \times 10^6 = 0.193 \times 10^3 \times 2.32h^2$$

$$\therefore \quad h = 448 \text{ mm}$$

With a tensile applied force in addition, some extra thickness will be required.

Try

$$h = 500 \text{ mm}$$

$$M_r = 0.20 \times 10^3 \times 500^2 \times 2.32 \times 10^{-6} = 116 \text{ kNm/m}$$

$$\frac{f_m}{p_m} = \frac{2.32 \times 90/116}{2.32} = 0.78$$

For tensile force

$$f_t = \frac{75 \times 10^3}{500 \times 10^3} = 0.15 \text{ N/mm}^2$$

$$p_t = 1.66$$

$$\therefore \quad \frac{f}{p_t} = \frac{0.15}{1.66} = 0.09$$

$$\therefore \quad \frac{f_m}{f_{mp}} + \frac{f_t}{f_{tp}} = 0.78 + 0.09 = 0.87$$

which is <1.0 and ∴ satisfactory

(2) *Strength*

Concrete Grade 35. $p_{cb} = 12.8$ N/mm^2

High yield deformed bars $p_{st} = 130$ (class B exposure—0.2 mm crack width)

Modular ratio $\alpha_e = 15$. Cover = 40 mm

Axial distance = 40 + 12 + 10 = 62 mm

From previous calculation $h = 500$ mm, and $d = 438$. Assuming 0.5% steel the depth of the neutral axis under flexure only is given by

$$x = 0.3d = 0.3 \times 438 = 130 \text{ mm (formula 15)}$$

$$\text{Lever arm} = \left(1 - \frac{0.3}{3}\right) \times 438 = 0.90 \times 438 = 394 \text{ mm}$$

$$\therefore \quad A_{st} = \frac{M}{f_{st} z} = \frac{90 \times 10^6}{130 \times 394} = 1757 \text{ mm}^2$$

check

$$\rho_c = \frac{1757}{10^3 \times 500} = 0.35\%$$

$$\frac{d}{h} = \frac{438}{500} = 0.87$$

∴ assumptions in (1) above are satisfactory as actual ρ_c is < assumed value and actual $\frac{d}{h}$ is $\ngtr$ the assumed value.

Reinforcement required for resistance to tensile force

$$= \frac{F}{p_{st}} = \frac{75 \times 10^3}{130} = 577 \text{ mm}^2 \text{ provided in each face equally.}$$

$$\therefore \quad \text{Total } A_s = 1757 + \frac{577}{2} = 2045 \text{ mm}^2 \quad \text{(tension face)}$$

$$A'_s = \frac{577}{2} = 290 \text{ mm}^2 \quad \text{(compression face)}$$

Minimum reinforcement in each face may be calculated or nominal. Assume that 0.35% total is required, half in each face.

$$\tfrac{1}{2} \times 0.35\% \times bh = 0.175\% \times 10^3 \times 500$$
$$= 875 \text{ mm}^2$$

∴ Provide:

'Tension' face 2045 mm^2:	T20 at 150 (2090)
'Compression' face 875 mm^2:	T16 at 200 (1010)
	with $h = 500$.

3.10 Bond and anchorage

At the overlap of bars transmitting tension, it is preferable to be generous with the length of overlap to avoid cracking at the ends of the lapped bars.

Distribution reinforcement or any reinforcement acting to resist early thermal stresses should be designed to have lap lengths sufficient to resist the yield or proof strength of the bar using the appropriate bond strength at 3 days maturity (Table 3.9). If a greater steel ratio than required is actually provided, the lap lengths can be reduced in proportion. For limit state design, the required lap lengths are given in Table 3.10. Again, it is possible to reduce the laps by the proportion of steel area provided/steel area required. The lap lengths in Tables 3.9 and 3.10 are based on calculations using the permissible bond stresses and other values in BS 8007 and BS 8110 and apply to bars up to 20 mm size with 40 mm cover. Where the bar size is greater, reference should be made to BS 8110 for further requirements.

Table 3.9 Tension lap lengths for the critical reinforcement ratio for shrinkage and thermal movement design in slabs up to size 20 bars

	Plain round mild steel	Deformed high-yield steel (ribbed)
f_y (N/mm^2)	250	460
concrete grade		
25	46ϕ	58ϕ
30	42ϕ	52ϕ
35	39ϕ	48ϕ

Table 3.10 Tension lap lengths for ultimate limit state design

Concrete grade	Plain round mild steel	Deformed high-yield steel (ribbed)
	$f_y = 250$	$f_y = 460$
25	39ϕ	40ϕ
30	36ϕ	37ϕ
35	33ϕ	34ϕ

Note: Factors of 1.4 or 2.0 must be applied in certain circumstances (see BS 8110 for details):

1.4 for bars resisting forces of direct tension
1.4 for horizontal bars near to the top of a section
1.4 where a lap occurs at the corner of a section
2.0 where two of these circumstances occur together

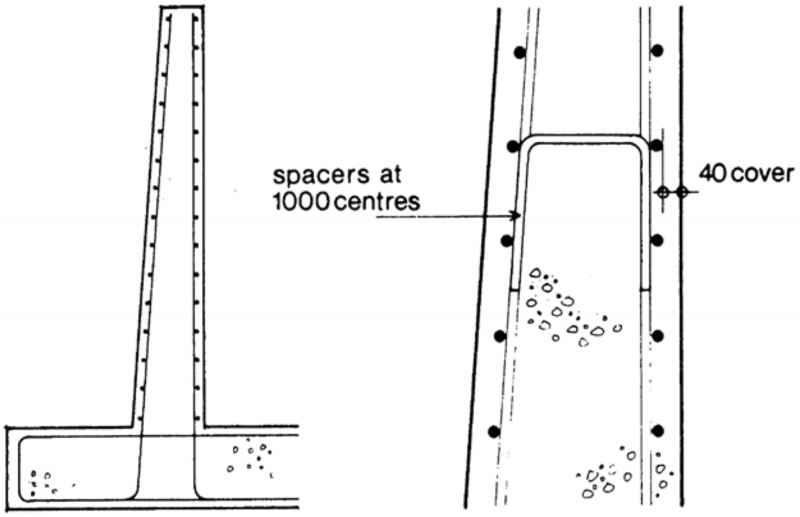


Fig. 3.16 Detailing of spacer reinforcement

3.11 Detailing[26,27,28]

The reinforcement detailing requirements for water-retaining structures follow the usual rules for normal structures. Bars should be detailed for continuity on the liquid faces and sudden changes of reinforcement ratio should be avoided. The distribution reinforcement in walls should be placed in the outer layers where it has maximum effect. Spacers should be detailed to ensure that the correct cover is maintained (Fig. 3.16).

Welded fabric reinforcement is normally used to reinforce floor slab panels and may also be used in some walls where the required reinforcement area is not too large. BS 8110 contains special rules for evaluating the tension lap lengths required for fabric.

4

Design of prestressed concrete

4.1 The use of prestressed concrete

Prestressed concrete is a structural material in which compressive stresses are induced in the concrete before imposed loading is applied. The magnitude of the induced stresses is arranged so that, after the application of imposed loads, the stresses in the concrete are still largely compressive. Prestressing can be applied in a slab in one direction or in two orthogonal directions in the plane of the slab. Prestressed concrete is divided into two types.

(1) Pre-tensioned: in which long wires are stretched on a tensioning bed in the factory. Concrete is then placed in moulds around the wires which are released when the concrete is hardened. The wires are then cut at intervals along the length to create separate elements.
(2) Post-tensioned: in which concrete elements are cast in place and subsequently stressed with external or internal wires.

Prestressed concrete would appear to have a considerable advantage for use in liquid-retaining structures in that the concrete is in compression, there are no cracks, and leakage is not possible. In practice, it is difficult to make use of this advantage, but various applications are discussed in the following sections.

It is not possible in a book of this length to deal fully with the theory and practice of prestressed concrete, and for further information the reader is referred to *Design of Prestressed Concrete* by Bate and Bennett(29).

4.2 Materials

4.2.1 *Concrete*

The maximum stresses on the concrete are not usually very high in relation to the strength of the concrete. A strength of 40 N/mm^2 will generally be satisfactory and provide sufficient durability. It is important to ensure adequate workability in order to achieve full compaction.

4.2.2 *Prestressing tendons*(18,30)

Prestressing wires or strands of the normal commercial grades may be used. The jacking force is limited to 75% of the characteristic ultimate strength of the tendon, and losses due to friction, slip of the grips, relaxation of the steel, and elastic contraction, shrinkage and creep of the concrete must be considered when arriving at the final active prestressing forces. Tendons may be drawn through ducts cast into the concrete section or placed on the outside of a wall (and subsequently covered with pneumatically placed concrete to give cover to the steel and ensure durability).

A proprietary system of winding wire under tension around circular tanks is widely used.

Frictional losses in post-tensioned prestressed concrete can occur between the cables and the sides of the ducts, caused by the unintentional curvature of the duct, and the frictional forces will be increased by the duct not being exactly in line, and having irregularities in the profile. To allow for the curvature, the loss may be calculated from

$$P_0(1-e^{-\mu x/R})$$

To allow for irregularities, the loss may be taken as

$$P_0(1-e^{-kx})$$

where

P_0 = prestressing force at the jack
$e = 2.718$
R = radius of curvature of the duct
μ = coefficient of friction
  $\mu = 0.55$ for steel on concrete
  $\mu = 0.30$ for steel on steel
k = a constant depending on the type of duct which may vary between 33×10^{-4} and 17×10^{-4}.

4.3 Precast prestressed elements

It is not generally possible to make use of precast prestressed wall elements because of the difficulty in making an economical connection between the units longitudinally and also between the wall units and the floor. However, the use of a precast prestressed roof to a reservoir or tank may be economical. Reservoir roofs are generally supported by reinforced concrete columns at centres of about 5 to 7 metres. Where climatic conditions require speedy construction due to approaching bad weather, a precast design enables the construction period to be shortened.

The design of roof elements follows the normal principles of prestressed concrete design. Because of condensation within a confined space, the underside of the roof will normally be rated as severe exposure (see Chapter

2) and the units may be designed as Class 1 with no tensile stresses on the underside with full imposed loading. The imposed loads may include soil cover and vehicles during construction.

4.4 Cylindrical prestressed concrete tanks(31,32)

4.4.1 *Loads and horizontal forces*

A cylindrical tank (with a vertical axis) is a convenient structure to contain liquid. The radial pressure due to the liquid is uniform at all points on the circumference at a given depth. Each circular slice of the tank wall at a given level is therefore in equilibrium under the applied liquid pressure, and horizontal ring tensile forces are developed in the wall. The pressure and forces vary linearly with depth from zero at the liquid surface to a maximum at the tank floor, and hence the induced horizontal ring tension also varies. In reinforced concrete construction, the wall deflects outwards by an amount which varies with depth (ignoring any effect of the floor) (Fig. 4.1).

The use of prestressed concrete enables compressive ring forces to be induced in the wall which counteract the tensile forces due to the liquid. Stressing tendons may be placed more closely in the lower part of the wall and more widely spaced in the upper sections. Thus, in theory, it is possible to arrange for zero stresses in the concrete with a tank full of liquid. In practice, the variation in spacing of the wires will not be continuously variable and, in order to avoid leakage, it is desirable to have a larger prestressing force throughout the wall height than is necessary to counteract exactly the applied loading, so that there is always a slight residual compressive stress in the concrete.

4.4.2 *Base restraint*

The discussion in Section 4.4.1 has ignored the effect of the connection

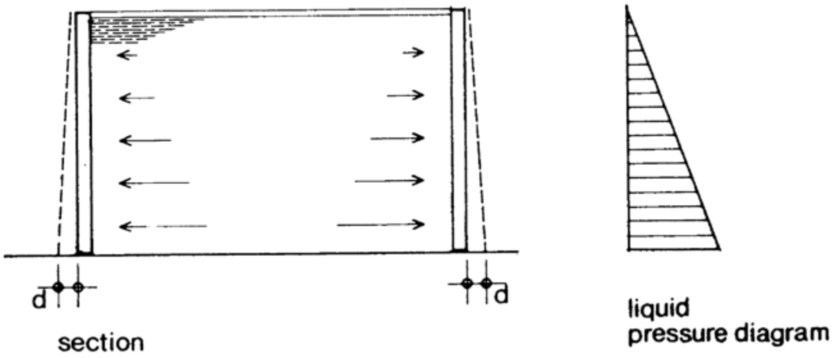


Fig. 4.1 Outward deflection of cylindrical tank (free at base)

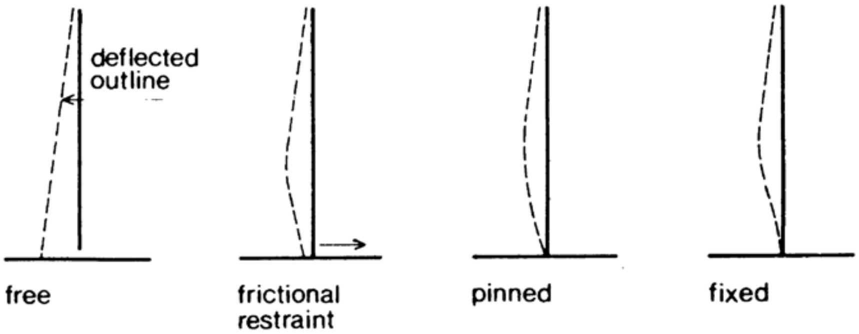


Fig. 4.2 Effect of base restraint on a loaded prestressed concrete tank

between the wall and the floor. There are three possible types of connection (Fig. 4.2):

(*a*) fixed
(*b*) pinned
(*c*) free to slide

It is clear that if a fixed joint is used, it is not possible to prestress the concrete wall effectively near to the base, as no inward radial movement is possible.

A nominally free joint provides the least resistance to circumferential prestress, but in practice it is not possible to avoid frictional forces between the wall and base due to the deadweight of the wall. In view of the doubt about the extent of any restraint, the circumferential prestressing should be designed on the basis of no restraint. This is a safe procedure. The effect of restraint is to cause a varying vertical bending moment in the wall section.

The radial deflection of the wall under load is shown in Figure 4.2 for the various forms of restraint.

Fixed restraint has been shown to be disadvantageous when considering the circumferential prestress, and is also difficult to achieve. The tank base is founded on soil which will deflect under the weight of the tank and the moment due to fixity, and the base itself is to some degree flexible (Fig. 4.3).

A compromise is possible between the three types of fixity which is convenient in practice. The joint is made nominally free to slide during the prestressing operation, and then pinned in position so that under full-load conditions the joint acts as a pin.

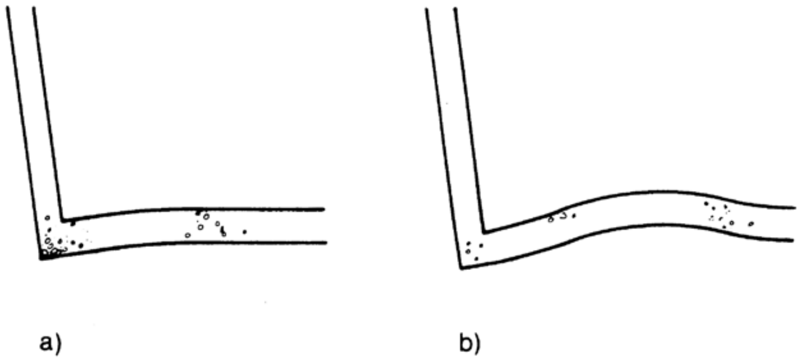


Fig. 4.3 Rotation of a 'fixed' wall footing
(*a*) Due to settlement
(*b*) Due to flexibility of the footing

4.4.3 *Vertical design*

Reinforcement must be provided in the vertical direction to resist the following forces:

(*a*) Bending moments induced by the variation of prestress with depth when the tank is empty.
(*b*) Bending moments induced due to base fixity.
(*c*) Load out-of-balance moments created during the prestressing operation.
(*d*) Bending moments due to variation in temperature between the outside and inside of the tank.

Forces (*a*) and (*b*) have been discussed already while (*d*) is of importance only in countries where the sun can cause a high surface temperature on the concrete. In temperate climates it is not usual to take account of this effect.

Vertical bending moments may be resisted by normal reinforcement or by further vertical prestressing or a combination of both. For tanks up to about 7 m deep, it will be more economical to use normal reinforcement(31).

5

Distribution reinforcement and joints: Design against shrinkage and thermal stresses

Cracks are induced in reinforced concrete members by the action of applied loads by hydration of cement and by environmental conditions. This chapter considers the effect of environmental conditions on concrete slabs which are assumed to be otherwise unloaded(9,33). A typical practical example is in the longitudinal direction of a cantilever reservoir wall. In circumstances where a wall spans in two directions, the steel to resist thermal stresses can also be considered as resisting flexural action, i.e. the required areas of reinforcement for each action are not additive.

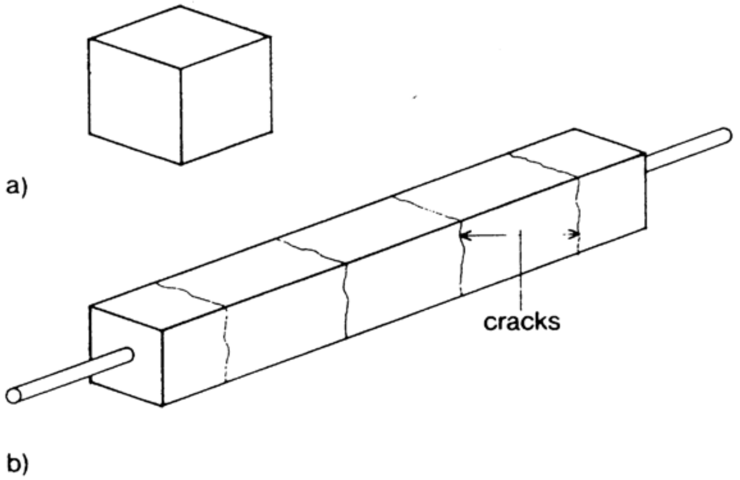


Fig. 5.1 Cracking in concrete elements
(*a*) Small cube free to move
(*b*) Long reinforced element restrained by reinforcement

5.1 Cracking in reinforced concrete

If a small cube of concrete is cast (such as a test cube), it will not exhibit cracking apparent to the naked eye. However, if a long specimen with relatively small cross-sectional dimensions is made containing reinforcement, and the ends of the reinforcement are held to prevent any movement, after a few days it will be found that fine lateral cracks are present (Fig. 5.1). Depending on the relation between the quantity of reinforcement, the bar size, and the cross-sectional area of concrete, either a few wide cracks, or a larger number of fine cracks will form. The cracks are induced by the resistance of the reinforcement to the strains in the concrete caused by chemical hydration of the cement in the concrete mix(34).

5.2 Causes of cracking

5.2.1 *Heat of hydration*

When materials are mixed together to make concrete, a chemical reaction takes place between the cement and water during which heat is evolved. This heat of hydration causes the temperature of the concrete to rise until the reaction is complete, and the heat is then dissipated to the surroundings. A typical curve illustrating the temperature rise in concrete during the first few days after mixing is shown in Figure 5.2*a*. By the sixth day, the temperature is usually back to normal. The value of the maximum temperature is dependent on the quantity of cement in the mix, the thickness of the concrete section, and any insulation that is provided, deliberately, or by formwork. Concretes which are rich in cement will emit larger quantities of heat than concretes with a low cement content(35). A thick concrete section (over about 800 mm) will not cool very quickly, because the ratio of surface area to total heat emitted is lower. Recent work on the avoidance of cracking has shown that it may be advantageous to allow thick sections to cool slowly by preventing rapid loss of heat. This is achieved by covering the exposed concrete with an insulating blanket. For normal structural work, the formwork should not be removed for three or four days, otherwise cold winds may cause surface cracking of the warm concrete(36,37).

During the period when the concrete temperature is increasing, expansion will take place. If the expansion is restrained by adjoining sections of hardened concrete, some creep will occur in the relatively weak concrete, relieving the compressive stresses induced by the attempted expansion. As the concrete subsequently cools, it tries to shorten but, if there are restraints present, tensile strains will develop leading to cracking (Fig. 5.2*b*). This is known as 'early thermal movement'.

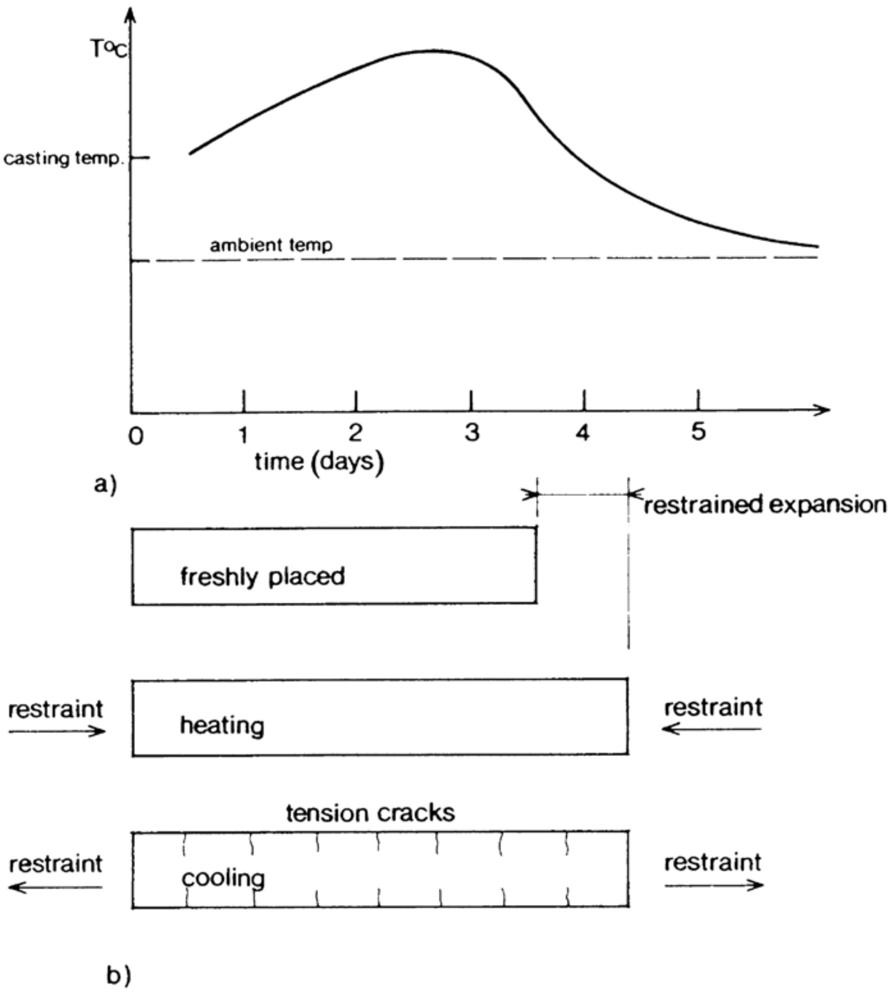


Fig. 5.2 (a) Rise in temperature of freshly placed concrete;
(b) Thermal strains

5.2.2 *Drying shrinkage*

As concrete hardens and dries out, it shrinks. This is an irreversible process. If a reinforced concrete member is considered under no external stress (Fig. 5.3), it will be apparent that free shrinkage of the concrete is prevented by the steel reinforcement. The steel is therefore in compression and the concrete in tension, with longitudinal bond forces present on the surface of the reinforcement. The magnitude of these forces is dependent on the concrete properties, and the ratio of the area of steel to the area of concrete. If a high ratio of steel is present, and there is no external restraint applied to the element, no cracks will form but, if the steel ratio is relatively small or external restraints are present, cracking is certain. The cracks may form at close centres and be fine in width, or may be further apart and be correspondingly wider in order to accommodate the total strain. It is

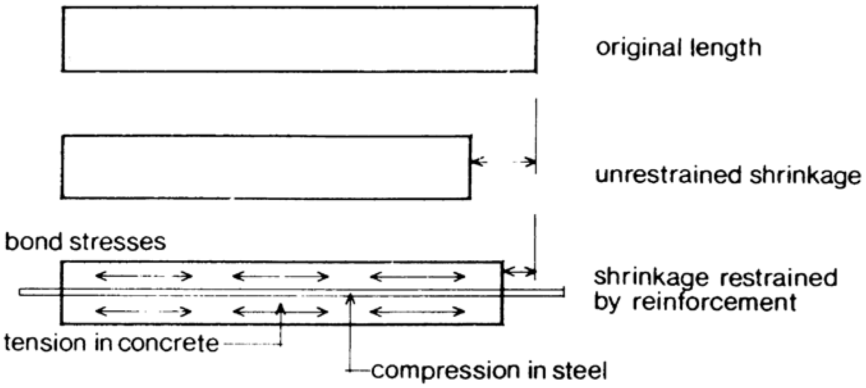


Fig. 5.3 Drying shrinkage in reinforced concrete

important that there is sufficient reinforcement to control the cracking. If this is not the case, a few very wide cracks will form, and the reinforcement will yield at the crack positions(9). The various possibilities are illustrated in Figure 5.4.

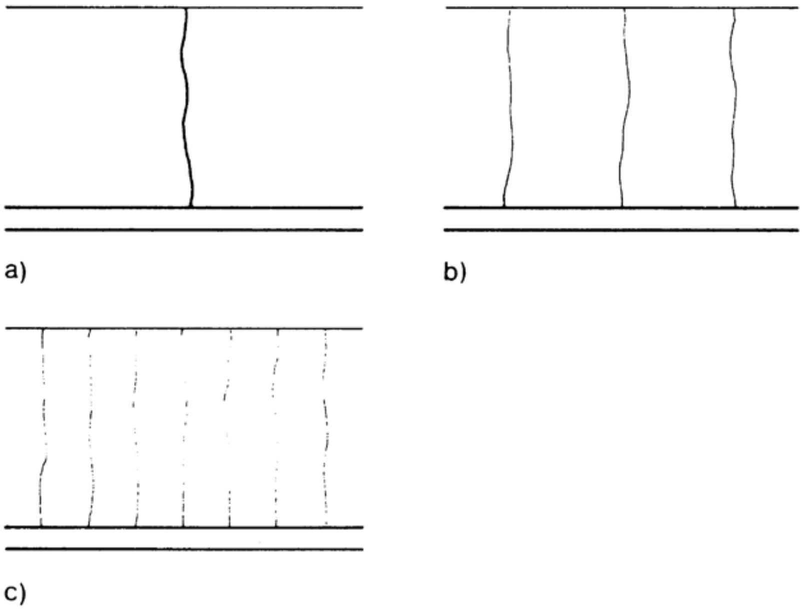


Fig. 5.4 Development of cracks related to reinforcement in a wall
(*a*) Insufficient steel—wide cracks, steel yields
(*b*) Controlled cracks—average spacing and width
(*c*) High proportion of steel—well-controlled cracks of narrow width and close spacing

5.2.3 *Environmental conditions*

An elevated concrete water tower will be subjected to strains due to changes between summer and winter temperatures. In temperate climates (such as the

UK), it is not usual to consider these effects for normal types of structures, but in countries where temperatures are more extreme, some allowance may need to be made(38).

The effect of the sun in heating part of the surface of a structure may produce differential strains between one side and another. Again, in temperate climates, these are usually ignored in calculation, but in hot countries they will have to be considered(38).

5.3 Crack distribution(34)

Cracking due to early thermal movement may be controlled by reinforcement (Fig. 5.5). The objective is to distribute the overall strain in the wall between reinforcement and movement joints, so that the crack widths are acceptable or, if considered desirable, that the concrete remains uncracked. There is no single solution to the design problem of controlling early thermal cracking. The designer may choose to have closely spaced movement joints with a low ratio of reinforcement, or widely spaced joints with a high ratio of reinforcement. The decision is dependent on the size of the structure, method of construction to be adopted, and economics.

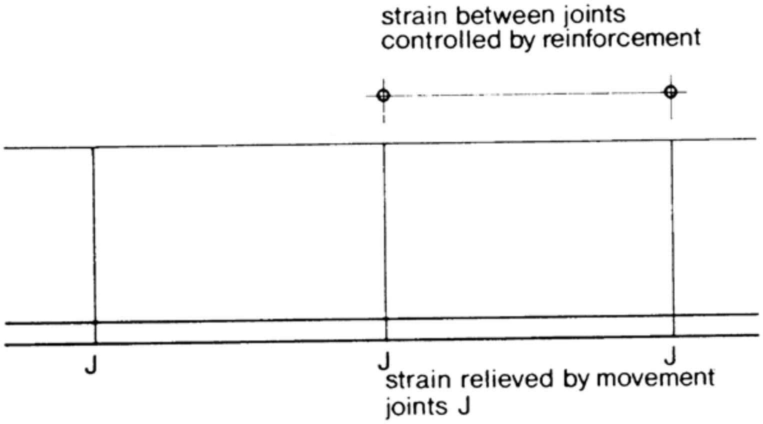


Fig. 5.5 Relation of movement joints and reinforcement in controlling strain in a wall

5.3.1 *Critical steel ratio*

Figure 5.6 shows part of a reinforced concrete slab. The concrete section is taken as an area on each side of the bar, corresponding to half the distance to the next reinforcing bar. Alternatively, a unit width of one metre may be defined, and the total area of steel within this width may be read from bar spacing tables.

The steel ratio in a section is defined as the ratio

$$\rho_c = \frac{A_s}{bh}$$

and is often expressed as a percentage, where

A_s is the total area of reinforcement (in both faces)
b is the width of the section
h is the overall thickness of the section.

If a section contains a low steel ratio, the strength of the steel at yield will be less than the ultimate concrete strength in tension and when cracks form, the steel will yield. Thus the cracks will be wide and unrestrained.

At a certain critical steel ratio, the reinforcement yields and the concrete reaches its ultimate tensile stress at the same time. If the section is reinforced, so that the actual steel ratio is not less than the critical ratio, then any cracks which form will be restrained and of moderate width.

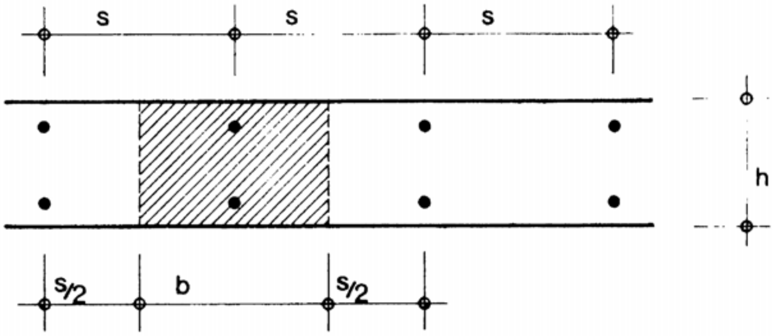


Fig. 5.6 Typical section of reinforced concrete slab

Considering the section in Figure 5.6, the critical steel ratio may be obtained by equating the yield force in the steel with the tensile force in the concrete. The compressive force in the steel between cracks may be neglected. Hence

$$A_s f_y = bhf_{ct}$$

and

$$\left(\frac{A_s}{bh}\right)_{crit} = \rho_{crit} = \frac{f_{ct}}{f_y}$$

The control of cracking is critical during the early life of the concrete, and therefore a value of concrete tensile strength at 3 days should be used. Typical values of ρ_{crit} and f_{ct} are given in Table 5.1.

5.3.2 *Crack spacing*

If sufficient reinforcement is provided to control the cracking ($\rho_c > \rho_{crit}$), then the probable spacing of the cracks may be estimated. As the shrinkage strain increases cracks form in sequence when the bond force between the reinforcement and the concrete becomes greater than the tensile strength of the concrete. The bond stress between concrete and the surface of the steel that

Table 5.1 Values of critical steel ratios

Concrete grade (N/mm^2)	f_{ct} (N/mm^2)	f_y (N/mm^2)	ρ_{crit} %
35	1.6	460	0.35
		250	0.64
30	1.3	425	0.31
		250	0.52
25	1.15	425	0.27
		250	0.46

f_{ct} = direct tensile strength of the concrete at 3 days.
f_y = characteristic yield strength of reinforcement.
ρ_{crit} = critical steel ratio (the minimum required to control cracking in a restrained section).

accompanies the formation of a crack extends for a length equal to half the crack spacing (Fig. 5.7). Equating the two forces gives

$$f_b s \Sigma u_s = f_{ct} bh \tag{1}$$

where

Σu_s = total perimeter of bars in the width considered
f_b = average bond stress adjacent to a crack
s = bond length necessary to develop cracking force
f_{ct} = tensile stress in concrete
bh = area of concrete

Writing steel ratio $\rho_c = \dfrac{A_s}{bh}$ (neglecting concrete area taken up by steel) and the ratio

$$\frac{\text{total perimeters}}{\text{total steel area}} = \frac{\Sigma u_s}{A_s}$$

$$= \frac{\pi\phi \times (\text{number of bars})}{\dfrac{\pi}{4}\phi^2 \times (\text{number of bars})} = \frac{4}{\phi} \tag{2}$$

where ϕ = bar size (or equivalent size for square or ribbed bars), substitution in (1) gives

$$f_b s\left(\frac{4}{\phi}\ A_s\right) = f_{ct} bh$$

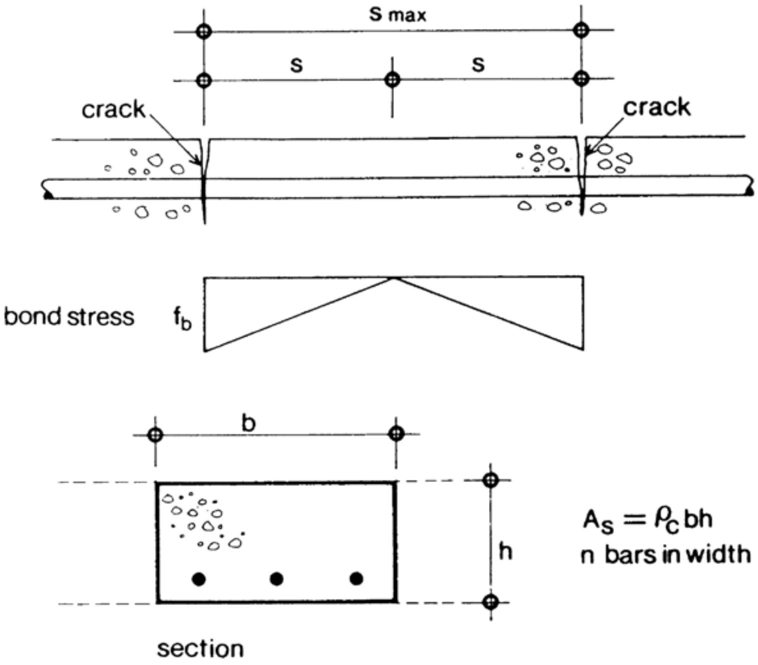


Fig. 5.7 Bond stress and crack formation

$$\therefore \quad s = \left(\frac{f_{ct}}{f_b}\right)\left(\frac{1}{\rho_c}\right)\left(\frac{\phi}{4}\right)$$

$$= \left(\frac{f_{ct}}{f_b}\right)\frac{\phi}{4\rho_c}$$

and the maximum crack spacing

$$s_{max} = 2s = \left(\frac{f_{ct}}{f_b}\right)\frac{\phi}{2\rho_c} \qquad (3)$$

The ratios (f_{ct}/f_b) for various types of bars are given in Table 5.2. The ratios relate to concrete properties at an age of about 3 days.

It is apparent from the formula that crack spacing is influenced directly by

Table 5.2 Ratios of $\left(\frac{f_{ct}}{f_b}\right)$ at early ages

Bar type	Ratio
Plain bars	1
Deformed bars type 1 (Twisted squares)	4/5
Deformed bars type 2 (Ribbed)	2/3

bar diameter (other variables being constant). This confirms the judgement of a previous generation of engineers who preferred small bars at close centres for crack control.

For a given steel ratio, it is possible to choose the bar size (within limits) so that the crack spacing is small, and the total contraction strain is accommodated by the formation of many fine cracks, or by choosing a larger bar size, to cause cracks to form at wider centres. If joints are placed at the assumed centres of crack formation, the concrete will effectively be uncracked.

5.3.3 *Crack widths*

The number and width of cracks which form will depend on the total contraction strain that is unrelieved by joints in the length of the section. The contraction strain is the sum of the shrinkage strain and the thermal strain (due to changes in the ambient temperatures after the structure is complete). Assuming that the steel ratio is greater than ρ_{crit} and with full restraint (i.e. no joints), the tensile strain in the concrete may be assumed to vary from zero adjacent to a crack to a value of ε_{ult} (ultimate concrete strain) midway between cracks at a distance s_{max} apart. The average tensile strain in the uncracked concrete is therefore $\frac{1}{2}\varepsilon_{ult}$. The strain due to the maximum crack width, which is the difference between the total contraction strain and the strain remaining in the concrete, is therefore given by

$$\left(\frac{w}{s_{max}}\right) = \varepsilon_{te} + \varepsilon_{cs} - \tfrac{1}{2}\varepsilon_{ult}$$

where w = maximum crack width
s_{max} = maximum spacing of cracks
ε_{te} = thermal contraction from peak temperature
ε_{cs} = total shrinkage strain
$\varepsilon_{\mathrm{ult}}$ = ultimate concrete tensile strain.

There is not sufficient information available to enable precise values for the various coefficients to be given, but ε_{ult} may be assumed to be 200 microstrains. The shrinkage strain in the concrete, minus creep strain, is about 100 microstrains and therefore in the formula above equates with the value of $\frac{1}{2}\varepsilon_{ult}$. The remaining strain to be considered, ε_{te}, is therefore due to cooling from the peak of hydration temperature T_1 to ambient temperature. There is also a further variation in temperature T_2 due to seasonal changes after the concrete in the structure has hardened.

When considering the strain due to temperature T_1, an effective coefficient of expansion of one half of the value for mature concrete should be used due to the high creep strain in immature concrete. For mature concrete and seasonal variations due to temperature T_2, the tensile strength of the concrete is lower compared with the bond strength, hence s is much less for mature concrete when T_2 is appropriate; hence the actual contraction can be effectively halved. The strain equation now becomes

$$\begin{aligned}\left(\frac{w}{s_{max}}\right) &= \tfrac{1}{2}\alpha(T_1+T_2)+100-(\tfrac{1}{2}\times 200)\\ &= \tfrac{1}{2}\alpha(T_1+T_2)\end{aligned}$$

$\therefore$ Crack width $w = s_{max}\tfrac{1}{2}\alpha(T_1+T_2)$

where α = coefficient of linear expansion of concrete.

Typical values for T_1 applicable to conditions in the UK are given in Table 5.3.

Table 5.3 Typical values of T_1 in °C for OPC concretes, where more particular information is not available.

	Walls						Ground slabs		
	1			2			3		
Section thickness (mm)	Steel formwork: OPC content (kg/m^3)			18 mm plywood formwork: OPC content (kg/m^3)			OPC content (kg/m^3)		
	325	350	400	325	350	400	325	350	400
300	11	13	15	23	25	31	15	17	21
500	20	22	27	32	35	43	25	28	34
700	28	32	39	38	42	49	—	—	—
1000	38	42	49	42	47	56	—	—	—

Note: (1) For suspended slabs cast on flat steel formwork, use the data in column 1.
(2) For suspended slabs cast on plywood formwork, use the data in column 3.
The table assumes the following:
(*a*) that the formwork is left in position until the peak temperature has passed
(*b*) that the concrete placing temperature is 20 °C
(*c*) that the mean daily temperature is 15 °C
(*d*) that an allowance has not been made for solar heat gain in slabs.

T_1 should not be taken as less than 20 °C for walls or 15 °C for slabs. Suitable values of T_2 depend on the change in environmental temperature between casting and subsequent use. For construction in summer, there will be a change to winter temperatures, and a value for T_2 of 20°C would be appropriate. For concrete cast in winter, the subsequent rise in temperature during the summer months will tend to cause expansion rather than contraction and the effect on the concrete will tend to reduce any cracking. A suitable value for T_2 would be 10°C. These values are applicable to UK climatic conditions.

If a structure holds warm liquid, as in a swimming pool, then prudence suggests that some allowance should be made for the warming effect of the retained liquid. The tendency is for the warm face to expand and cracks will tend to close. However, as there will generally be a temperature gradient through the wall between one face and the other, cracks will tend to form on the cooler face. This can simply be explained by equating the longitudinal forces. The compression caused by the thermal warming must be balanced by the tension on the opposite face. There is no precise method of calculation of the results of temperature stresses as many of the coefficients and restraint factors are speculative, but a method of providing some assessment of the conditions is provided in reference (44).

The theory given in this section has been developed by Professor B. P. Hughes, University of Birmingham$^{(9)}$.

5.3.4 *Thick sections*

The theory outlined above allows a calculated steel ratio to be assessed and the necessary reinforcement is then calculated as the product of the steel ratio

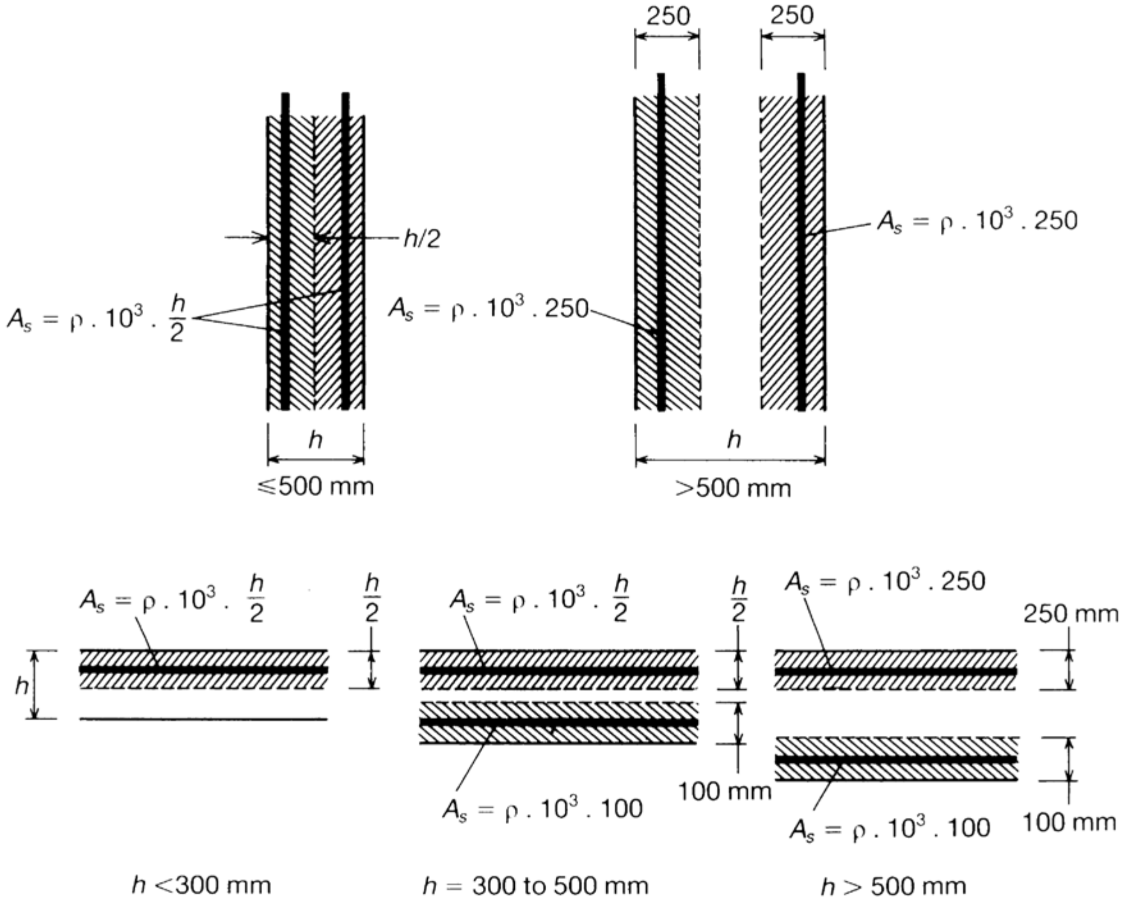


Fig. 5.8 (a) Surface zones in walls and suspended slabs
(b) Surface zones in ground slabs

Note 1. Take care not to halve the calculated value of ρ as well as taking $h/2$ as a multiplier
2. ρ is the steel ratio, not percentage

and the section thickness. This total area of steel is equally divided between each face of the slab. The strains in a slab are largely due to temperature gradients across the depth of the slab. In the centre of the depth of a slab, away from the faces, it has been found that the strains are much smaller. Indeed, the best method of reducing the thermal strains in a newly placed slab is to keep the faces warm rather than to allow them to cool as quickly as possible. In a thick slab (over 500 mm) it has been found that it is not necessary to increase the total amount of thermal reinforcement beyond that necessary for a slab of 500 mm thickness. Having calculated the required steel ratio, it can be converted to the reinforcement area by considering two surface zones in the slab (Fig. 5.8). Each surface zone is of thickness equal to one half the overall depth of the slab, but with a maximum value of 250 mm. the calculated reinforcement for each surface zone is placed adjacent to that face. It follows that for a slab over 500 mm in thickness, the thermal steel in each face remains constant.

Example 1

Slab $h = 400$ mm
Required ratio = 0.40%
Provide 0.40% × 200 × 1000 in each face = 800 mm^2/m
Use T12 at 125 (905) in each face.

Example 2

Slab $h = 700$ mm
Required ratio = 0.40%
Provide 0.40% × 250 × 1000 in each face = 1000 mm^2/m
Use T16 at 200 (1010) in each face.

5.3.5 *External restraint factors*

As discussed above, the external restraints in a typical continuous structure may be taken as 50%, and this is included in the formulae quoted. In certain situations the actual restraint will be less than 50% and this is shown diagrammatically in Figure 8.3 of BS 8007. As the practical use of these easements is limited they are not presented in detail here, but discussion of the details is to be found in the reference where research work by Harrison(45) is reported.

5.4 Joints(40,41)

5.4.1 *Construction joints*

It is rarely possible to build a reinforced concrete structure in one piece. It is therefore necessary to design and locate joints which allow the contractor to construct the elements of the structure in convenient sections. In normal structures, the position of the construction joints is specified in general terms by the designer, and the contractor is allowed to decide on the number of joints and their precise location subject to final approval by the designer.

In liquid-retaining structures this approach is not satisfactory. The design of the structure against early thermal movement and shrinkage is closely allied to the frequency and spacing of all types of joints, and it is essential for the designer to specify on the drawings exactly where construction joints will be located. Construction joints should be specified where convenient breaks in placing concrete are required. Concrete is placed separately on either side of a construction joint, but the reinforcement is continuous through the joint. At a horizontal construction joint, the free surface of the concrete must be finished to a compacted level surface. At the junction between a base slab and a wall, it is convenient to provide a short 'kicker' which enables the formwork for the walls to be placed accurately and easily. A vertical joint is made with formwork. Details are shown in Figure 5.9.

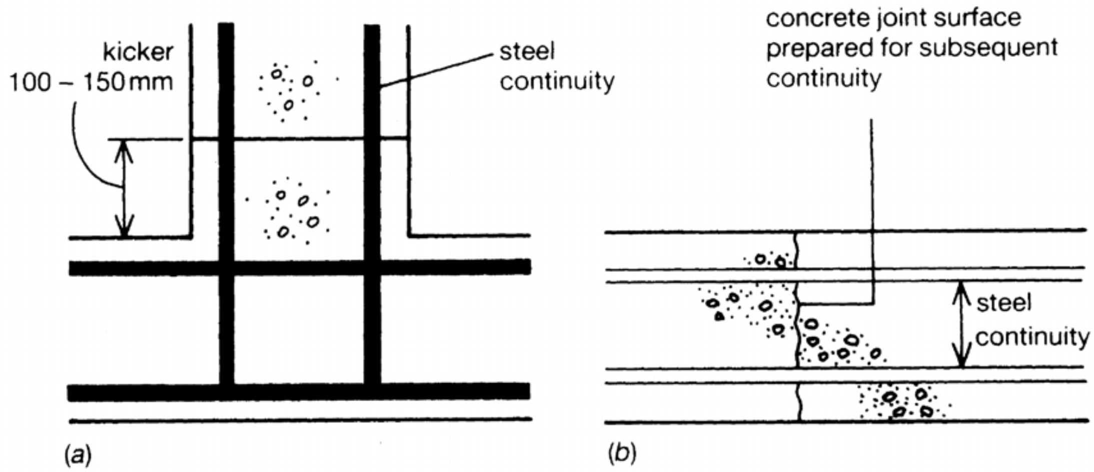


Fig. 5.9 Construction joints
(*a*) Horizontal joint between base slab and wall
(*b*) Vertical joint

Construction joints are not intended to accommodate movement across the joint but, due to the discontinuity of the concrete, some slight shrinkage may occur. This is reduced by proper preparation of the face of the first-placed section of concrete to encourage adhesion between the two concrete faces. Joint preparation consists in removing the surface laitance from the concrete without disturbing the particles of aggregate. It is preferable to carry out this treatment when the concrete is at least five days old, either by sandblasting or by scabbling with a small air tool. The use of retarders painted on the formwork is not recommended, because of the possibility of contamination of the reinforcement passing through the end formwork. The face of a construction joint is flat and should not be constructed with a rebate. It is found that

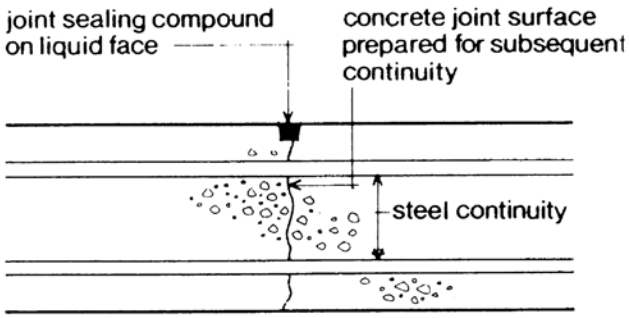


Fig. 5.10 Construction joint sealed on the liquid face

the shoulders of a rebate are difficult to fill with compacted concrete, and are also liable to be cracked when the formwork is removed. Any shear forces can be transmitted across the joint through the reinforcement. If a construction joint has been properly prepared and constructed, it will retain liquid without a waterstop. Extra protection may be provided by sealing the surface as shown in Figure 5.10.

Designers are under some pressure to use waterstops in construction joints for obvious commercial reasons and also because it is thought that there is less responsibility thrown onto the designer if a waterstop is specified than if it is omitted. The author knows of instances where waterstops have been used and leaks have been widespread. In other cases, both waterstops and an external membrane have been specified and again with completely unsatisfactory results. These examples suggest that workmanship is critical and that whatever specification is used this point is valid.

The designer must try to convince the contractor's site operatives that the work they will be executing is of the greatest importance for the correct functioning of the completed structure. Where the contractor has taken the job at a particularly low price, this may be difficult.

It is perhaps also worth stating the obvious, that it is much cheaper to spend a little more time initially to make a satisfactory job than to have to make repairs later.

5.4.2 *Movement joints*

Movement joints are designed to provide a break in the continuity of a slab, so that relative movement may occur across the joint in the longitudinal direction. The joints may provide for the two faces to move apart (contraction joints) or, if an initial gap is created, the joint faces are able to move towards each other (expansion joints). Contraction joints are further divided into *complete contraction joints* and *partial contraction joints*.

Other types of movement joints are needed at the junction of a wall and roof slab (Fig. 5.11), and, where a free joint is required, to allow sliding to take place at the foot of the wall of a circular prestressed tank (Fig. 5.12).

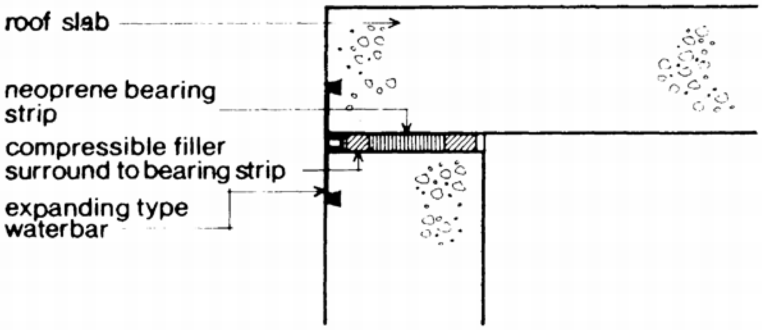
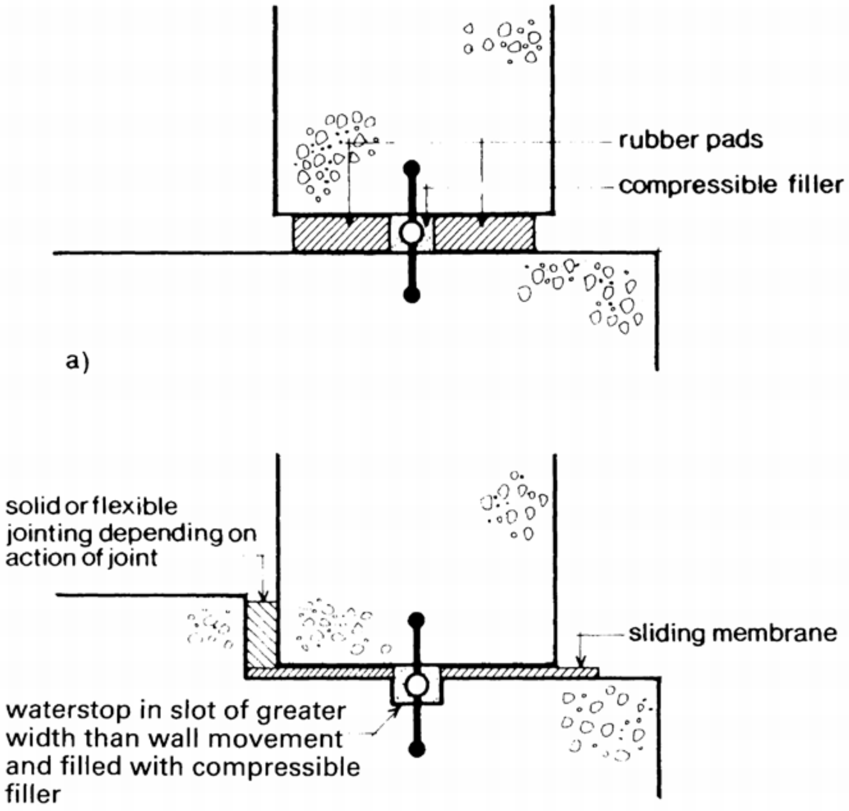


Fig. 5.11 Detail for movement joint between wall and roof slab



b)

Fig. 5.12 Movement joints between base slab and wall of prestressed concrete tank
(*a*) Rubber pad
(*b*) Sliding membrane

Contraction joints. Complete contraction joints have discontinuity of both steel and concrete across the joint, but partial contraction joints have some continuity of reinforcement. Figures 5.13 and 5.14 illustrate the main types. In partial contraction joints, the reinforcement may all continue across the joint, or only 50% of the steel may continue across the joint, the remaining

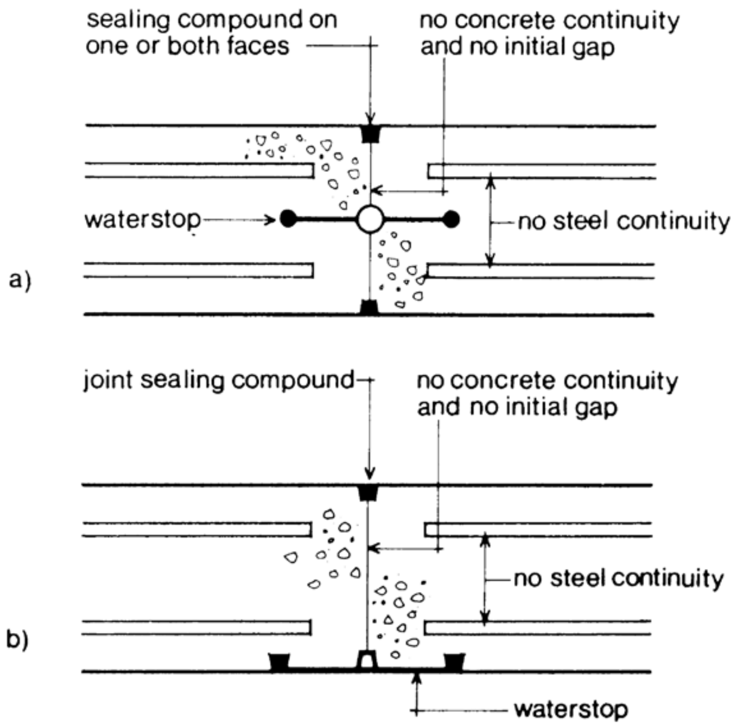


Fig. 5.13 Complete contraction joints
(*a*) Wall joint
(*b*) Floor joint

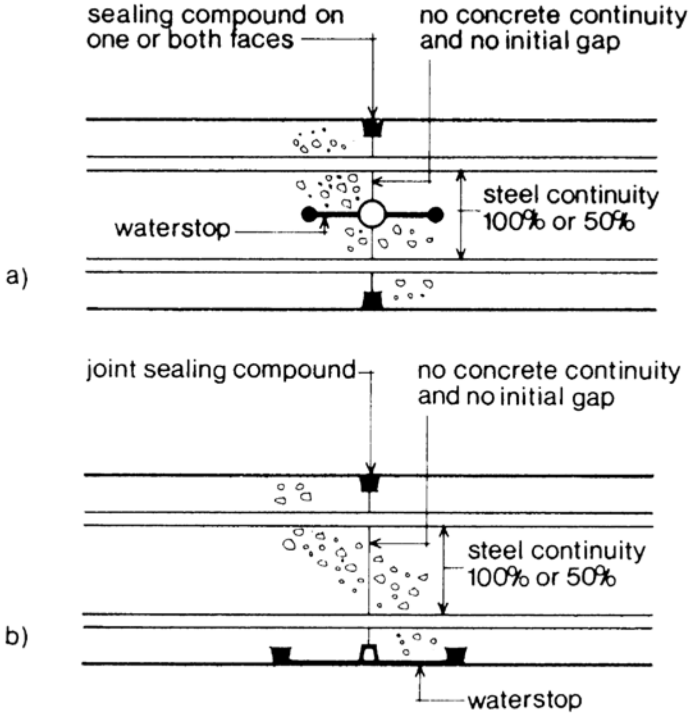


Fig. 5.14 Partial contraction joints
(*a*) Wall joint
(*b*) Floor joint

50% being stopped short of the joint plane. The purpose of the three types of contraction joints is described in Section 5.4.2.

Contraction joints may be constructed as such, or may be induced by providing a plane of weakness which causes a crack to form on a preferred line. In this case, the concrete is placed continuously across the joint position, and the action of a device which is inserted across the section, to reduce the depth of concrete locally, causes a crack to form. The formation of the crack releases the stresses in the adjacent concrete, and the joint then acts as a normal contraction joint. A typical detail is shown in Figure 5.15. Great care is necessary to position the crack inducers on the same line, as otherwise the crack may form away from the intended position. Similar details may be used in walls with a circular-section rubber tube placed vertically on the joint-line on the wall centre-line, causing the crack to form.

Expansion joints. Expansion joints are formed with a compressible layer of material between the faces of the joint. The material must be chosen to be durable in wet conditions, non-toxic (for potable water construction), and have the necessary properties to be able to compress by the required amount and to subsequently recover its original thickness. An expansion joint always needs sealing to prevent leakage of liquid. In a wall, a water-bar is necessary containing a bulb near to the centre which will allow movement to take place without tearing (Fig. 5.16). The joint also requires surface sealing to prevent the ingress of solid particles. By definition, it is not possible to transmit longitudinal structural forces across an expansion joint, but the designer may wish to provide for shear forces to be carried across the joint, or to prevent the slabs on each side of the joint moving independently in a lateral direction. If a reservoir wall and footing is founded on ground that is somewhat plastic, the sections of wall on either side of an expansion joint may rotate under load by differing amounts. This action creates an objectionable appearance and may tear the jointing materials (Fig. 5.17). The slabs on either side of an expansion joint may be prevented from relative lateral movement by provid-

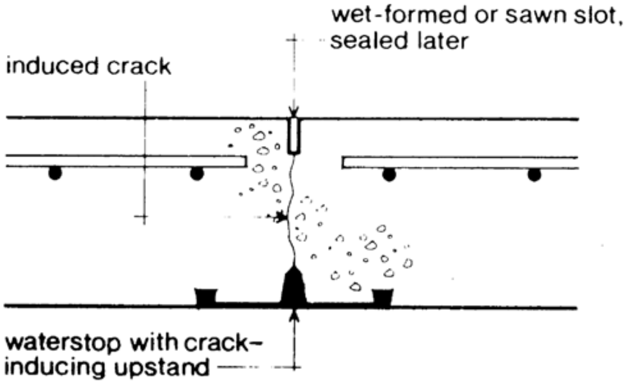


Fig. 5.15 Induced contraction joint in floor

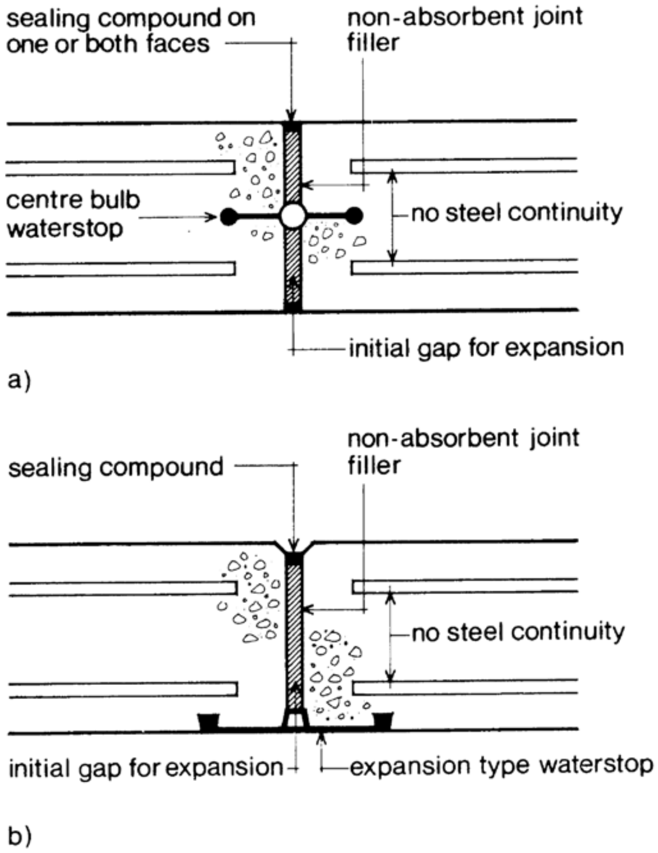


Fig. 5.16 Expansion joints
(*a*) Floor
(*b*) Wall

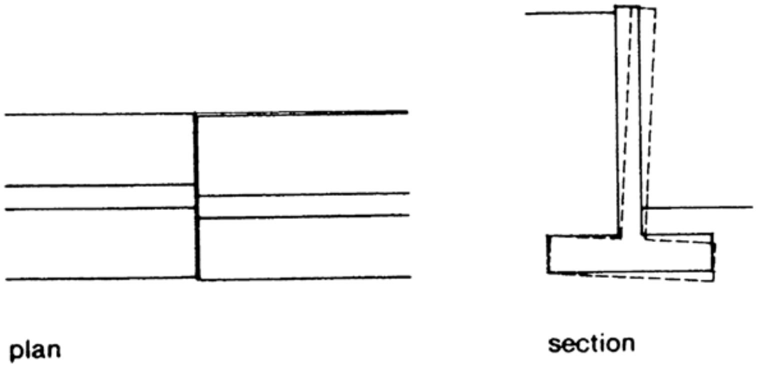


Fig. 5.17 Lateral movement at unrestrained expansion joint

ing dowel bars with provision for longitudinal movement (a similar arrangement to a road slab). The dowel bars must be located accurately in line (otherwise the joint will not move freely), be provided with an end cap to allow movement, and be coated on one side of the joint with a de-bonding compound to allow longitudinal movement to take place (Fig. 5.18).

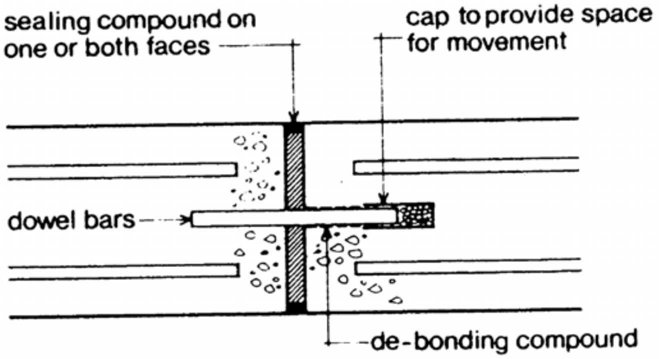


Fig. 5.18 Expansion joint including dowel bars to prevent lateral movement

5.5 Typical calculations for distribution steel

5.5.1 *Codes of Practice requirements*

The theory outlined in this chapter has been adopted by British Standard Code of Practice (BS 8007), which requires the designer to assess the required reinforcement to control shrinkage and thermal movement. A critical minimum steel ratio is specified.

The ACI Code(11) specifies a minimum percentage of shrinkage and thermal reinforcement for walls up to 300 mm thick and a fixed minimum quantity of steel for thicker sections. The Australian Code (AS 3735–1991) specifies an absolute minimum of 0.4% and a larger value for fully restrained concrete which varies between 0.48% and 1.28% depending on bar size.

5.5.2 *Calculation of minimum reinforcement*

Assume construction with movement joints at 15 m centres.

Design for crack width of 0.2 mm

$$\text{Slab thickess} = 400\text{ mm}$$

$$T_1 = 30°\text{C}$$

$$T_2 = 0$$

$$\alpha_c = 12$$

$$\text{Restraint factor } R = 0.5$$

Net effective contraction strain $= 0.5 \times 12 \times 30 = 180$ microstrain

Maximum allowable crack width = 0.2 mm

$$\therefore\ s_{max} = (0.2/180) \times 10^6 = 1110\text{ mm}$$

Also $$s_{max} = \frac{f_{ct}}{f_b} \times \frac{\phi}{2\rho}$$

Assume bar size $\phi = 12$ mm

$$\therefore \quad s_{max} = \frac{2}{3} \times \frac{12}{2\rho} = \frac{4}{\rho} \text{ mm}$$

$$\therefore \quad \rho = (4/1110) \times 100 = 0.36\%$$

Critical steel ratio $$\rho_{crit} = \frac{1.6}{460} \times 100 = 0.35\%$$

Use <u>T12 at 150 EF</u> (1510) (EF = each face)

$$\left(\text{Actual ratio} = \frac{1510}{10^3 \times 400} \times 100 = 0.38\%\right)$$

6

Design calculations

This chapter comprises three design calculations, each forming a complete example and dealing with a particular structure. The handwritten sheets illustrate the graphical method employed in the engineering design process, and each calculation conforms to practical requirements.

6.1 Design of pumphouse

A pumphouse is to be built as part of a sewerage scheme to house three underground electric pumps. The layout is shown in Figure 6.1. Design the underground concrete structure. A soil investigation shows dense sand and no ground water. For safety considerations, stairs and a ladder would be provided in the dry well and two manholes in the roof of the wet well. The manholes would enable air to be blown into the well before inspections were made. To simplify the design problem these elements may be disregarded.

Design assumptions

The floor and walls must be designed against external soil pressures due to soil and surcharge from vehicles which may park near to the structure. Although no ground water has been found during the site investigation it is quite possible that during the life of the building some ground water may be present on the outside of the walls. Building the structure creates a sump in the original ground which tends to collect water. It is therefore prudent to design the floor and walls to exclude any ground water which may be present. For these reasons, a nominal head of ground water of 1.0 m will be assumed in the structural design. The pump well is designed to hold the effluent as a liquid-retaining structure. Although the normal working level is about mid-height of the walls, it is possible for the effluent to fill the well completely, and for design purposes this condition will be assumed.

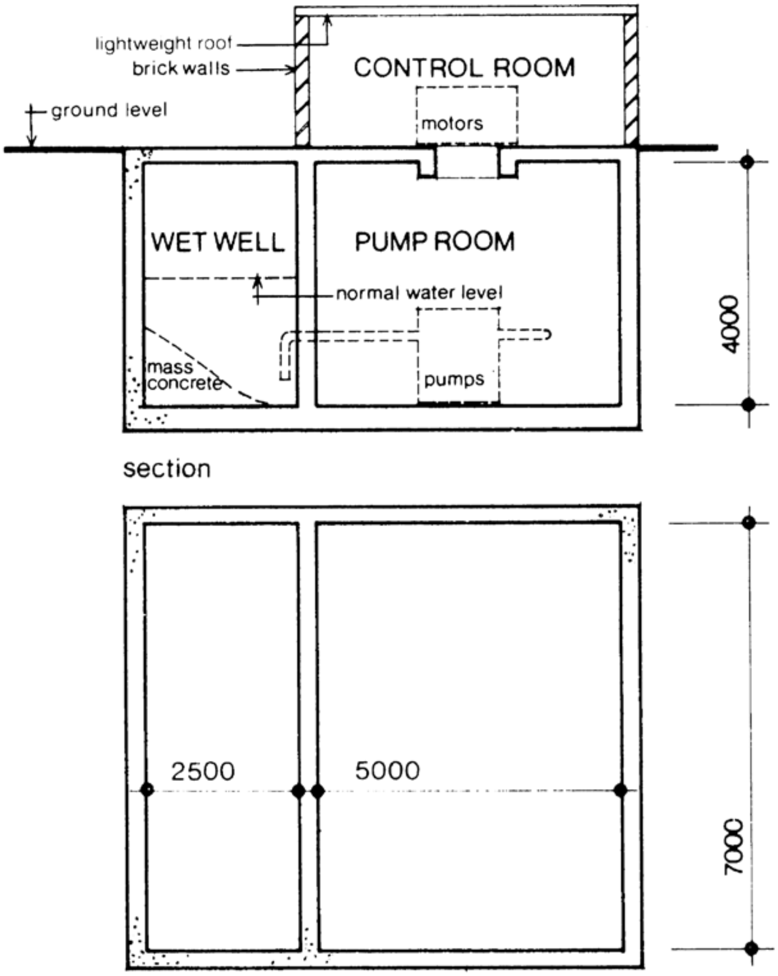


Fig. 6.1 Layout of pumphouse

ref. | calculations EXAMPLE 6.1 SHEET 1 | output

DESIGN OF PUMPHOUSE

Design the reinforced concrete underground pumphouse shown in FIG 6.1

SOIL PROPERTIES

Granular soil : density = 18 KN/m³
Angle of repose = 30°
Surcharge due to loaded vehicles on surrounding ground = 10 KN/m²

Water in the wet well is normally at a level 2.0 m above the floor, but the design should allow for overflow conditions with the compartment full. The pumphouse is dry.

DESIGN ASSUMPTIONS

(a) Loads

$$\text{Soil pressure} = \frac{1 - \sin \phi}{1 + \sin \phi} \times 18 = \tfrac{1}{3} \times 18$$

$$= 6 \text{ KN/m}^2$$

Although no water is said to be present in the ground, the construction of the structure will create conditions which will allow ground water to collect. BS8102 recommends that design should be based on a ground water head of 0.75 of the depth, i.e. 1 m below the surface.
Assume density of water = 10 KN/m³

(b) Design to BS8007 and BS8110

BS 8110 — Severe exposure

BS 8007 — Design crack width = 0.2 mm

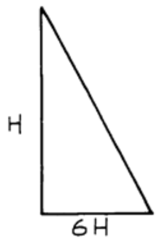
ref.	calculations	output
	(c) Materials	
BS 8007	Concrete grade 35A with a minimum cement content of $325\ Kg/m^3$ of finished concrete	CONCRETE GRADE 35A
	Reinforcement steel - ribbed high yield bars grade 460	
BS 8007 2.7.6.	(d) Cover to outer layer of steel = 40 mm	COVER = 40
	(e) Design Design all floor, wall and roof slab panels as continuous and 2-way spanning	
	(f) Joints In view of the size of the structure, no movement joints are desirable as they are potential sources of leakage. The structure will therefore be designed as a monolithic structure, and construction joints	
BS 8007 5.4	will be shown on the drawings. No waterbar will be necessary at these joints, but the joint surface will be scabbled.	

ref. | calculations EXAMPLE 6 1 SHEET 3 | output

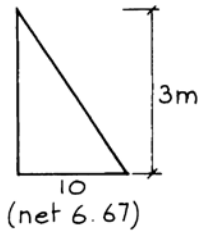
Loading Cases

(a) Wet well empty

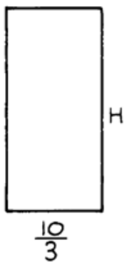
External soil and water pressures :-



Soil



Ground water



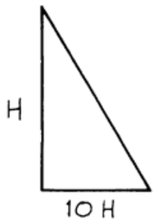
Surcharge

Where ground water is present, the effective density of the soil is reduced due to the buoyancy effect. The effective extra load due to the presence of ground water is therefore :—

$$10\ (1-\tfrac{1}{3})H = 6.67H$$

(b) Wet Well Full

Pressure due to water in well



BS 8007 2.3 — No allowance will be made for the passive resistance of the external soil

(c) Partial Safety Factor for Loads = 1.4

ref.	calculations EXAMPLE 6.1 SHEET 4	output
	Thickness of Sections	
BS 8110 TABLE 3.9	For ease of construction of a wall 4.0 m high, the minimum thickness should be 300 mm. The allowable ultimate shear strength of 35 grade concrete with an assumed 0.5% of reinforcement is 0.60 N/mm^2 The maximum ultimate shear force at the foot of the walls due to the maximum external loading is :—	
FACTOR $\frac{5}{8}$ IS FOR APPROX PROPPED CANTILEVER $\gamma f = 1.4$	$\frac{5}{8} \times 1.4 \left(\underset{\text{soil}}{\tfrac{1}{2} \times 6 \times 4^2} + \underset{\text{water}}{\tfrac{1}{2} \times 6.67 \times 3.0^2} + \underset{\text{surcharge}}{3.33 \times 4} \right)$ $= 80 \text{ kN/m}$	
	The minimum effective depth of wall required for shear is :— $d = \frac{80 \times 10^3}{0.6 \times 10^3} = 133 \text{ mm}$	
	Overall thickness = 133 + 40 + 16 + 8 = 197 mm	
	Use walls 300 thick and floor 400 thick	WALLS h = 300 FLOOR h = 400
	The roof to the wet well has to carry the surcharge pressure of 10 kN/m^2	
	Use a slab 250 thick	ROOF h = 250
	Effective depth d = approx. 250 − 40 − 10 = 200	
BS 8110 3.4.6.3	Span/effective depth = $\frac{2500}{200} = 12.5$ Satisfactory	

ref.	calculations EXAMPLE 6.1 SHEET 5	output

Calculation of Minimum Reinforcement

Maximum length of continuous construction =

$2500 + 5000 + 3 \times 300 = 8400$ mm

$T_1 = 30°C$

$T_2 = 0°C$

$\alpha_e = 12$

Net effective contraction strain

$= 0.5 \times 12 \times 30 = 180$ microstrain

Maximum allowable crack width = 0.2 mm

$\therefore S_{max} = \frac{0.2}{180 \times 10^{-6}} = 1111$ mm

Also $S_{max} = \frac{f_{ct}}{f_b} \times \frac{\phi}{2\rho}$

Assume $\phi = 12$ mm

$S_{max} = 0.67 \times \frac{12}{2\rho} = \frac{4}{\rho}$

$\therefore \rho = (\frac{4}{1111}) \times 100 = 0.36\%$

Also

$\rho_{crit} = \frac{f_{ct}}{f_y} = \frac{1.6}{460} \times 100 = 0.35\%$

$\therefore$ Minimum area of reinforcement

$= 0.36\% \times 1000 \times 300 = 1080$

Use T12 at 200 each face (1130)

T12 at 200
EF

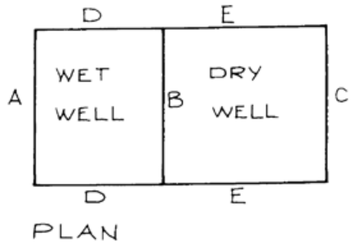
ref.

calculations

EXAMPLE 6 | SHEET 6

output

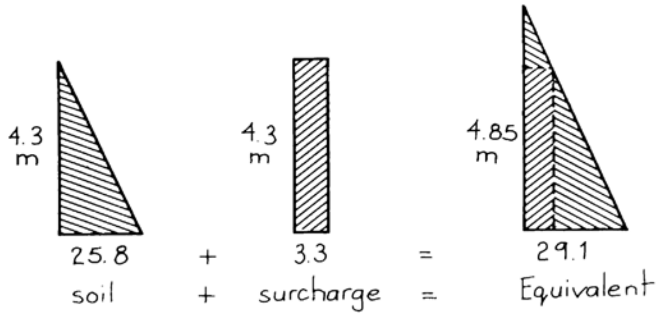
Forces on walls



Effective height of walls, centre/centre of roof slab/floor slab = 4000 + 300 = 4300

The wall panels are loaded triangularly due to water and soil pressures, and rectangularly due to surcharge pressure

It is convenient to replace the surcharge pressure by the equivalent soil pressure



$$h \text{ equivalent} = h + \frac{3.3}{6.0} = 4.85$$

Loading

Case 1

External soil pressure (including surcharge) = 29.1

Ground water = 6.7

$\underline{35.8}$

Case 2

Internal water pressure = 10 × 4.3 = 43.0

ref.	calculations EXAMPLE 6.1 SHEET 7	output

Wall A

Case

height of wall = 4300 }
length of wall = 7300 } ratio l_x / l_z = 1·7

M_V = vertical span
M_H = horizontal span
$+M$ = tension on unloaded face
$-M$ = tension on loaded face

APPENDIX B
CASE 2
FIG B.1

Soil + water
$-M_V$ = ·045 x 35.8 x 4·85 x 4·3 = 33·6
$+M_V$ = ·030 x 35.8 x 4·85 x 4·3 = 22·4
$-M_H$ = ·010 x 35.8 x $7{\cdot}3^2$ = 19·1
$+M_H$ = ·005 x 35.8 x $7{\cdot}3^2$ = 9·6

Case 2.

Internal water:
$-M_V$ = ·045 x 43 x $4{\cdot}3^2$ = 35·8
$+M_V$ = ·030 x 43 x $4{\cdot}3^2$ = 23.8
$-M_H$ = ·010 x 43 x 7.3^2 = 22·9
$+M_H$ = ·005 x 43 x 7.3^2 = 11·5

Wall B

As Case 2 wall A on each face

Wall C

As Case 1 wall A

ref.	calculations EXAMPLE 6.1 SHEET 8	output
	Wall D	
	Case 1	
	height of wall = 4300 } ratio l_x / l_z	
	length of wall = 2800 } = 0·65	
	Symbols and conventions as wall A	
APP. B. FIG B-1 CASE 2.	Soil + water $-M_V = \cdot 01 \times 35\cdot 8 \times 4\cdot 85 \times 4\cdot 3 = 7\cdot 5$	
	$+M_V = \cdot 006 \times 35\cdot 8 \times 4\cdot 85 \times 4\cdot 3 = 4\cdot 5$	
	$-M_H = \cdot 04 \times 35\cdot 8 \times 2\cdot 8^2 = 11\cdot 2$	
	$+M_H = \cdot 025 \times 35.8 \times 2.8^2 = 7\cdot 0$	
	Case 2	
	Internal water	
	$-M_V = \cdot 01 \times 43 \times 4\cdot 3^2 = 8\cdot 0$	
	$+M_V = \cdot 006 \times 43 \times 4\cdot 3^2 = 4\cdot 8$	
	$-M_H = \cdot 04 \quad 43 \times 2\cdot 8^2 = 13\cdot 5$	
	$+M_H = \cdot 025 \quad 43 \times 2\cdot 8^2 = 8\cdot 4$	
	Wall E	
	height of wall = 4300 } ratio l_x / l_z	
	length of wall = 5300 } = 1·2	
	Symbols and conventions as Wall A	
APP. B FIG B.1 CASE 2.	Soil $-M_V = \cdot 03 \times 35\cdot 8 \times 4\cdot 85 \times 4\cdot 3 = 22\cdot 4$	
	$M_V = \cdot 02 \times 35\cdot 8 \times 4\cdot 85 \times 4.3 = 14\cdot 9$	
	$M_H = \cdot 02 \times 35\cdot 8 \times 5\cdot 3^2 = 20\cdot 1$	
	$M_H = \cdot 01 \times 35\cdot 8 \times 5\cdot 3^2 = 10\cdot 1$	

ref.	calculations EXAMPLE 6·1 SHEET 9.	output

Direct Forces

All external loads cause compressive horizontal forces in the walls which are resisted by the concrete in compression. The value of the compressive stress is low and may be ignored. The load due to the water in the pumpwell causes tension in the walls which is evaluated below.

The maximum tension is in Wall D due to pressure on Walls A and B.

Maximum water pressure $= 10 \times 4{\cdot}3 = 43\ \text{kN/m}^2$

Average water pressure over lowest 1 m height of wall

$$= \frac{10\,(4{\cdot}3 + 3{\cdot}3)}{2}$$

$$= 38\ \text{kN/m}^2$$

Total force $= 38 \times 7{\cdot}0\ \text{m} = 266\ \text{kN/m}$ height

Force per metre height on each of walls, D

$$= \frac{266}{2} = 133\ \text{kN}$$

This calculation neglects the effect of the floor but is conservative

Area of steel required each face at $f_s = 240$

$$\frac{133 \times 10^3}{2 \times 240} = 277\ \text{mm}^2$$

ref. calculations EXAMPLE 6·1 SHEET 10 output

Floor Slab

The soil pressure under the floor slab is due to the imposed weight of the structure

The weight of the pumphouse super-structure (roof, walls and floor slab) has been calculated separately and amounts to 2200 kN

Assuming a uniform distribution over the floor area, the soil pressure is:-

$$\frac{2200}{5{\cdot}6 \times 7{\cdot}6} = 52 \text{ kN/m}^2$$

(wet wall area is neglected as load is largely over pump well)

Floor slab spans 2 ways

$$l_y/l_x = \frac{7300}{5300} \quad 1{\cdot}4$$

Assume simply supported and allow for fixing moments later

BS 8110
TABLE 3·15
CASE 9

$$+M_x = {\cdot}087 \times 52 \times 5{\cdot}3^2 = 127$$

$$+M_y = {\cdot}056 \times 52 \times 5{\cdot}3^2 = 82$$

Minimum fixing moments from walls (neglecting surcharge)

Ex. 6·1
(8)

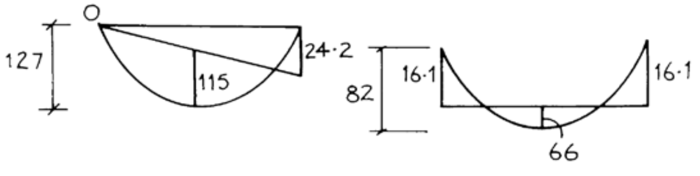
Walls E $M_F = -16{\cdot}1$

Walls C $M_F = -24{\cdot}2$

(Wall B – internal)

$\text{Max} + M_x = 127 - 24{\cdot}2/2 = 115$ kNm

$\text{Max} + M_y = 82 - 16{\cdot}1 = 66$ kNm



M_x M_y

See sheet 12 for reinforcement

ref.	calculations EXAMPLE 6·1 SHEET 11.	output
	<u>Wall reinforcement</u> Wall thickness = 300 Cover = 40 Effective depth to inner layer of reinforcement = 300 − 40 − 12 − 6 = 242 Minimum reinforcement calculated on sheet 6·1 (5) is T12 at 200 each face Referring to Appendix A Table A2·5, for T12 at 200, $M = 35{\cdot}4$	

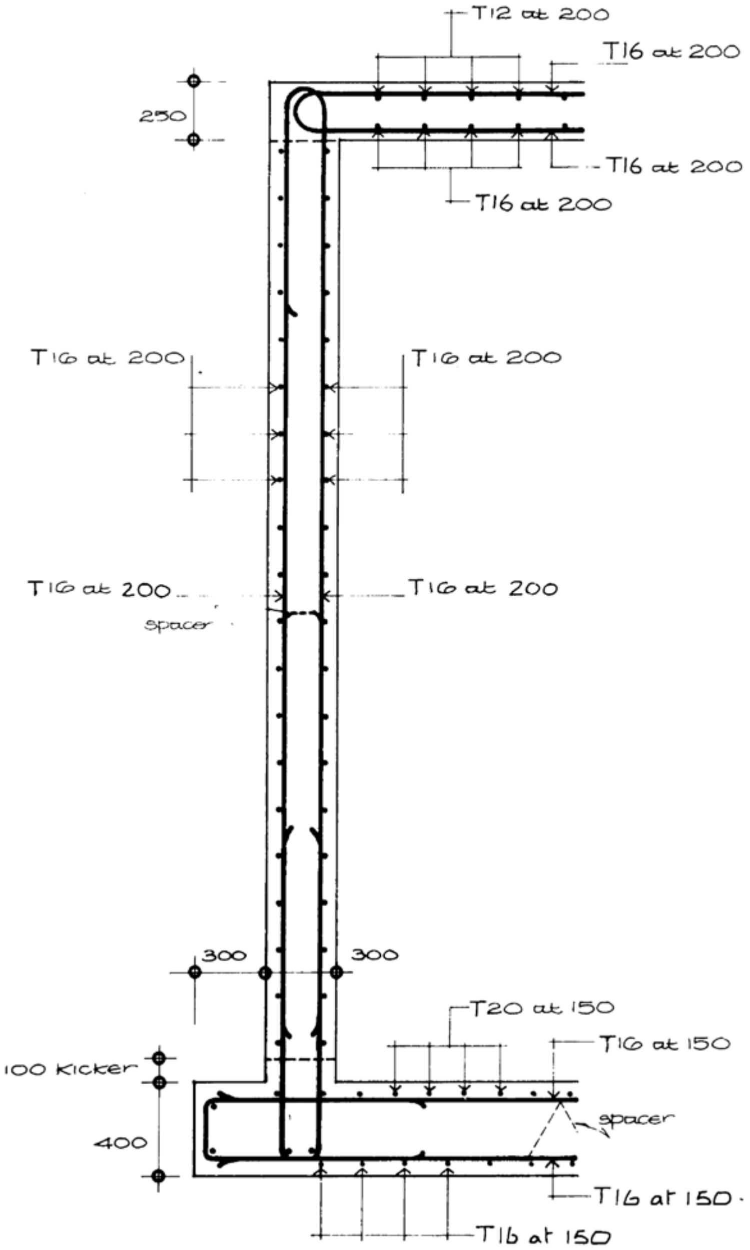
ref.	calculations EXAMPLE 6·1 SHEET 12	output
	Wall A	
Ex 6·1. (11).	Horizontal steel, all moments are less than 35·4 kNm/m therefore, for bending, use T12 at 200 A_s = 566. Add for direct tension A_s required = 566 + 277 = 843 USE T16 at 200 E.F. (1010)	WALL A. T16 at 200 EF
	Vertical steel M = 35·8 (this is maximum value : use same steel each face) Table A2·5 Try T16 at 200 E.F. Allowable M = 46 Satisfactory	WALL A. T16 at 200 EF
	Wall B As wall A	
	Wall C As wall A	
	Wall D T12 at 200 EF, EW	WALL D T12 at 200 EW, EF
	Wall E T12 at 200 EF, EW (Note EF = each face, EW = each way)	WALL E T12 at 200 EW, EF
	Floor reinforcement. Floor thickness = 400 Cover = 40 Effective depth of inner layer = 400−40−16−8 = 336 Negative moments as wall steel	
Ex 6·1. (10)	$+M_x$ = 115 Table A2·7 USE T20 at 150 TF (2090)	FLOOR T20 at 150 TF

ref.	calculations EXAMPLE 6·1 SHEET 13	output
	$+M_y = 66$ USE T16 at 150 TF (1340)	T16 at 150 TF
	Minimum steel in floor. $S_{max} = \frac{0{\cdot}2}{180 \times 10^{-6}} = 1111\text{mm}$ $\phi = 20$ $S_{max} = 0{\cdot}67 \times \frac{20}{2\rho} = \frac{6{\cdot}7}{\rho}$ $\therefore \rho = 0{\cdot}6\%$ $\rho_{crit} = 0{\cdot}35\%$	
	$\therefore A_s = 0{\cdot}6 \times 1000 \times 400 = 2400$ Use T16 at 150 EF (2680)	T16 at 150 EF
	Final steel arrangement Top face : T20 at 150 short span T16 at 150 long span Bottom face : T16 at 150 EW	

ref. **calculations** EXAMPLE 6·1 SHEET 14 **output**

REINFORCEMENT DETAILS

WALL A



6.2 Design of reservoir

Design a roofed reservoir to contain 4 000 000 litres of water.

Due to site conditions, the plan size will be taken as 21 m × 32 m and the maximum water height as 6.5 m. The normal height of the stored water is 6.0 m (Fig. 6.2). A division wall is required to divide the reservoir into two equal sections. The site is underlain by a granular soil, and there is no ground water present.

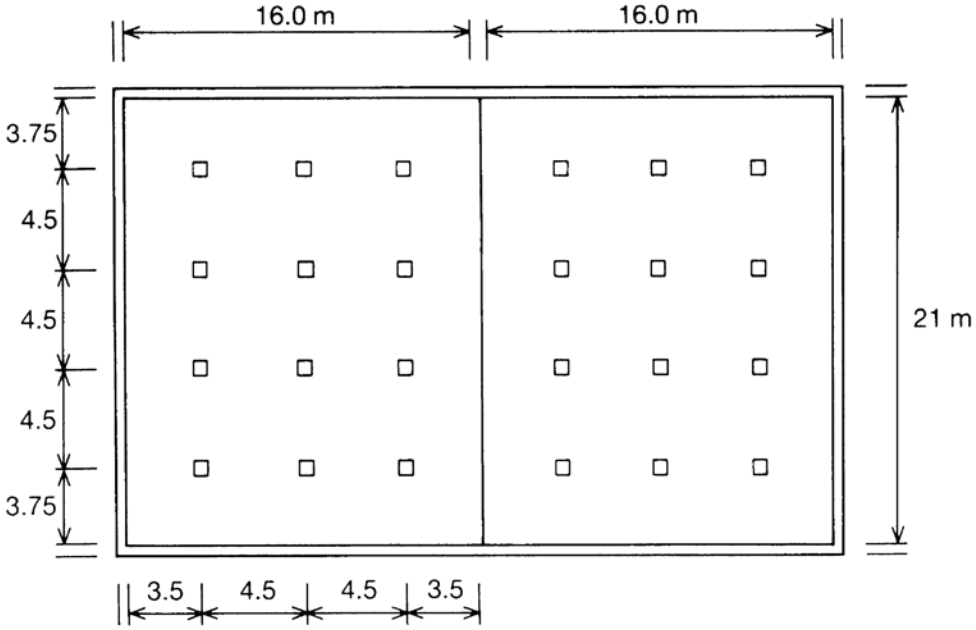
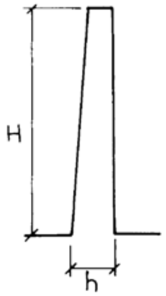
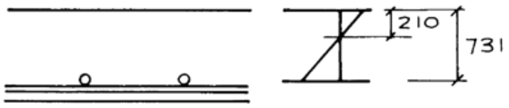


Fig. 6.2 Layout of reservoir

ref.	calculations EXAMPLE 6·2 SHEET 1	output
	DESIGN OF RESERVOIR	
	Layout of structure	
	Overall size in proportions of 1 : 1·5 with central wall.	
	Columns to support roof at equal spacings except that end spans of flat slab roof are about 0·8 of internal spans to equal end support bending moments with internal moments.	
	The size of the structure is less than the length which would require an expansion joint (say 70m)	
	Contraction joints will be provided at approx. 5.0m centres in the wall and floor panels	
	Provide a sliding joint between the top of the wall and the roof slab.	
	The cantilever wall is designed for water pressure in detail on pages ex. 6·2/2 to 4. The design to resist external soil pressure is prepared from the design charts.	
	Access manholes and stairs are required for safety during cleaning operations. These have not been detailed to simplify the design.	
	Exposure Conditions	
BS 8110	Severe exposure	
BS 8007	Design crack width = 0·2 mm	
	Materials	
	Concrete grade 35A	
	Steel grade 460	
	Partial Factors of Safety	
	Materials	
	Concrete 1·5	
	Steel 1·15	
	Loads	
	Water 1·4	
	Soil 1·4	

ref.	calculations EXAMPLE 6·2 SHEET 2	output
	LIMIT STATE DESIGN	
	Cantilever Wall	
	Consider 1m length of wall	
		
	Design a cantilever wall to withstand water pressure	
	Height H = 6·5m	
	Normal height of water = 6·0m	
	Water Wg = 10 kN/m^3	
	Partial safety factors:-	
	Ultimate limit state - γ_f = 1·4	
	Service limit state - γ_f = 1·0	
	Service moment at root of Cantilever = 1/6 × 10 × 6·0^3	
	= 360 kN m/m	
	Ultimate moment = 1/6 × 10 × 6·5^3 × 1·4 = 641 kN m/m	
TABLE 3.1 TABLE A2·12	From design charts the estimated root thickness	
	h = 800 mm and the tension reinforcement is	h = 800 T25 at 175
	T25 at 175 (2810)	
	Effective depth d = 800-40-16-12·5 = 731 mm	
	Ultimate limit state	
	For ultimate limit state, calculate using the maximum possible height of liquid	
	Applied moment $M = \frac{1}{6} \times (1{\cdot}4 \times 10) \times 6{\cdot}5^3$ = 641 kNm/m	
	The ultimate moment of resistance based on the concrete is:- $M = 0{\cdot}157 f_{cu} bd^2$	
SECTION 3.5 (6)	$= 0{\cdot}157 \times 35 \times 10^3 \times 731^2 \times 10^{-6}$	
	= 2936 kNm/m Satisfactory	
	The ultimate moment of resistence based on the steel is	
SECTION 3·5 (5)	given by $M_u = (0{\cdot}87 f_y) A_s z$	
	Depth of neutral axis $x = (1 - \sqrt{1 - 0{\cdot}7\frac{M}{M_u}})/0{\cdot}9)\,d$	
	$= 0{\cdot}088d$	
SECTION 3·5 (7)	where the lever arm $z = d - 0{\cdot}45x = 0{\cdot}96d$	
	But maximum value is 0·95d	
	$z = 694$	
	M_u = 780 kNm/m	
	The section is therefore satisfactory as the applied ultimate moment is less than the moment of resistance of the section	ULTIMATE MOMENT SATISFACTORY

ref.	calculations EXAMPLE 6·2 SHEET 3	output
	Ultimate shear force at root of wall	
	$V = \frac{1}{2} \times (1{\cdot}4 \times 10) \times 6{\cdot}5^2$	
	$= 296$ kN/m	
	∴ shear stress	
	$v = \frac{296 \times 10^3}{1000 \times 731} = 0{\cdot}40$	
	$\frac{100A_s}{bd} = \frac{100 \times 2810}{1000 \times 731} = 0{\cdot}38$	
BS 8110 Table 3·9	∴ $v_c = 0{\cdot}45 \times 1{\cdot}1$ (by interpolation)	
	$= 0{\cdot}50$	ULTIMATE SHEAR SATISFACTORY
	This is satisfactory and no shear steel is necessary	
	<u>Note</u>: As shear stress is satisfactory at root of wall, it is not necessary to consider the critical section at a height 2d above root.	
	<u>Limit State of Cracking</u>	
	Service moment $M_s = \frac{1}{6} \times 10 \times 6{\cdot}0^3$	
	$= 360$ kNm	
	Depth of neutral axis (elastic no-tension theory)	
SECTION 3·4	$x_1 = \alpha_e \rho \left(\sqrt{1 + \frac{2}{\alpha_e \rho}} - 1\right)$	
	modular ratio $\alpha_e = 15$	
	$\frac{A_s}{bd} = \rho = \frac{2810}{1000 \times 731} = 0{\cdot}00384$	
	∴ $\alpha_e \rho = 15 \times 0{\cdot}00384 = 0{\cdot}0577$	
	($\alpha_e = 15$ is sufficiently accurate)	
	∴ $x = 731 \times 0{\cdot}287 = 210$	
		
	Moment of resistance of section	
	$M_r = A_s f_s \times (d - \frac{x}{3})$	
	∴ $360 = 2810 \times f_s \times (731 - \frac{1}{3} \times 210)$	$f_s = 194$
	∴ $f_s = 194$ N/mm^2	

ref. | calculations EXAMPLE 6·2 SHEET 4. | output

BS 8007
APP B
B2

Check steel and concrete service stresses

$$f_s = 194 \qquad 0{\cdot}8\,f_y = 0{\cdot}8 \times 460 = 368$$

$$f_{cb} = \frac{2M_s}{z\,b\,x} = \frac{2 \times 360 \times 10^6}{661 \times 10^3 \times 210} = 5{\cdot}19$$

$$0{\cdot}45\,f_{cu} = 0{\cdot}45 \times 35 = 15{\cdot}7$$

Satisfactory

Elastic strain at surface

$$\varepsilon_1 = \frac{h-x}{d-x} \times \frac{f_s}{E_s}$$

$$= \frac{800-210}{731-210} \times \frac{194}{200} \times 10^{-3}$$

$$= 1{\cdot}098 \times 10^{-3}$$

Correction factor $\varepsilon_2 = \dfrac{b(h-x)(h-x)}{3E_s A_s\,(d-x)}$

$$= \frac{10^3 \times 590 \times 590}{3 \times 200 \times 10^3 \times 2810 \times 521}$$

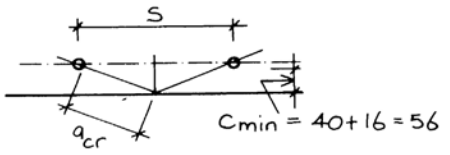
$$= 0{\cdot}4 \times 10^{-3}$$

Average surface strain

$$\varepsilon_m = (1{\cdot}098 - 0{\cdot}4)\,10^{-3}$$

$$= 0{\cdot}698 \times 10^{-3}$$

Crack width $w = \dfrac{3a_{cr}\varepsilon_m}{1 + 2\left(\dfrac{a_{cr} - c_{min}}{h-x}\right)}$



bar spacing s = 175
$c_{min} = 56$
$c_a = 56 + 12{\cdot}5 = 68{\cdot}5$

bar size ø = 25

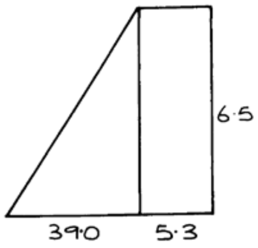
$$a_{cr} = \sqrt{\left(\frac{s}{2}\right)^2 + c_a^2} - \frac{ø}{2} = 98{\cdot}6$$

∴ surface crack width

$$w = \frac{3{\cdot}0 \times 98{\cdot}6 \times 0{\cdot}698 \times 10^{-3}}{1 + 2\left(\dfrac{98{\cdot}6 - 56}{800 - 210}\right)}$$

$$= 0{\cdot}18\,\text{mm}$$

Allowable w = 0·2 mm Satisfactory | w = 0·18

ref.	calculations EXAMPLE 6·2 SHEET 5	output
	Cantilever Wall (reservoir empty, soil pressure only)	
	Consider 1 m length of wall	
		
	soil density = $18 kN/m^3$	
	angle of respose = 30°	
	∴ soil pressure = 6h = $39 kN/m^2$	
	surcharge due to soil and imposed	
	load = 0·6m × 18 + 5 = $15·8 kN/m^2$	
	Surcharge pressure	
	= 15·8/3 = $5·3 kN/m^2$	
	Total service moment at base of wall	
	$= \frac{1}{6} \times 39 \times 6·5^2 + \frac{1}{2} \times 5·3 \times 6·5^2$	
	= 387 kNm	
	Ultimate moment	
	= 1·4 × 387	
	= 541 kNm	
	From previous calculation wall thickness h = 800	
	cover = 40 + 16 = 56	
	effective depth = 800 − 40 − 16 − 12·5	
	= 731	
	The calculations follow the previous pages and, as	
	an alternative, the results can be read from Table A2·12	
	For a design crack width of 0·2 mm	
	Provide T25 at 175 (2810)	
	Note: The slight under-design is not critical,	T25
	and the steel arrangements will be symmetrical	at 175
	in each face.	
Table	M_r = 384·0	
A2:12	f_s = 207	
	Ultimate shear force at (say) 1·5m above base	
	$= (\frac{1}{2} \times 39 \times 5 + 5·3 \times 5) \times 1·4$ = 174 kN/m	
	By inspection, and previous calculation – satisfactory	

ref. | calculations EXAMPLE 6·2 SHEET 6 | output

Cantilever Wall - distribution steel

Provide movement joints at 6·5 m centres

Critical steel ratio $\rho_{crit} = \frac{f_{ct}}{f_y}$

For grade 35 concrete the direct tensile strength at 3 days $f_{ct} = 1 \cdot 6$ N/mm²

For grade 460 steel $f_y = 460$ N/mm²

$$\rho_{crit} = \frac{1 \cdot 6}{460} = 0 \cdot 35\%$$

For close joint spacings, minimum steel ratio

horizontally $= \frac{2}{3}\rho_{crit} = 0 \cdot 23\%$

$\rho_{min} =$ 0.23%

Maximum permissible design crack width = 0·2 mm

$$S_{max} = \left(\frac{f_{ct}}{f_b}\right)\frac{\phi}{2\rho_c} = \frac{2}{3} \cdot \frac{\phi}{2\rho_c} = \frac{\phi}{3\rho_c}$$

Also $W_{max} = S_{max} \times \frac{\alpha}{2} T$

$\alpha = 12$ microstrain/°C

T is assumed to be 40°C (for a wall over 500 thick)

$$\therefore 0 \cdot 2 = S_{max} \times \frac{12}{2} \times 40 \times 10^{-6}$$

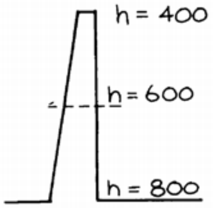
$$\therefore S_{max} = \frac{0 \cdot 2}{240} \times 10^{6}$$

$$\therefore \frac{\rho_c}{\phi} = \frac{1}{3} \cdot \frac{240}{0 \cdot 2} \times 10^{-6} = 400 \times 10^{-6}$$

For $\phi = 16$

$$\rho_c = 6 \cdot 4 \times 10^{-3} = 0 \cdot 64\%$$

or $\phi = 25$, $\rho_c = 1 \cdot 0\%$



For walls over 500 thick, only 500 need be considered in calculating the distribution reinforcement

At base level $\rho_c = 0 \cdot 64\%$ ($\phi = 16$)

$$A_s = 0 \cdot 64\% \times 500 \times 1000 = 3200 \text{ mm}^2/\text{m}$$

Use T20 at 175 each face (3600)

T20 at 175 EF

As the reinforcement remains the same until the thickness reduces below 500, the same distribution reinforcement will be used for the full height of the wall vertically.

ref.	calculations EXAMPLE 6·2 SHEET 7	output

Roof Slab

Design as flat slab with continuous construction, (ie no movement joints)

Provide movement joint at junction of slab and walls

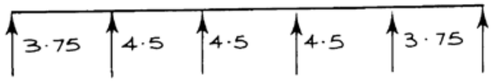
Loads

Imposed due to light construction traffic	5·0
600 soil cover	10·8
	15·8 kN/m²

With a careful construction specification, the imposed load can be taken as either not present, or present on all spans simultaneously

The critical span is adjacent to the external walls but the span has been made equal to 0·8 times the internal spans to compensate.

Due to the assumed incidence of loading, there will be only a small transfer of moment to the columns which will be ignored in the design of the slabs



Assume slab thickness = 450

Dead load	= 24 x 0·450 =	10·8
Imposed load		15·8
		26·6 kN/m²

Maximum ± service M for each span

BS 8110 TABLE 3.1

$$M = 0{\cdot}086x(26{\cdot}6 \times 4{\cdot}5) \times 4{\cdot}5^2$$

= 208 kNm/ full bay width

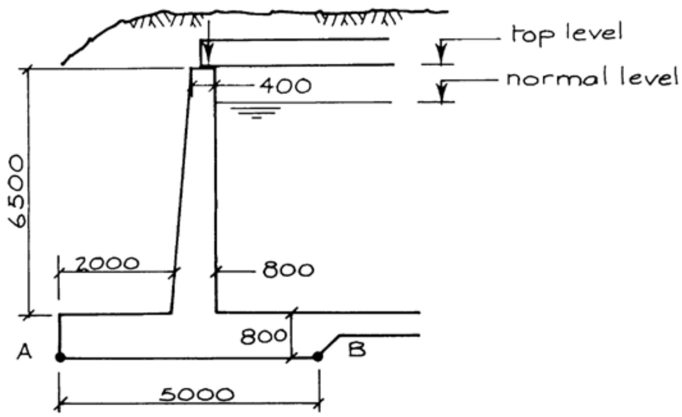
ref.	calculations EXAMPLE 6·2 SHEET 8	output
	Divide slab into column strips and middle strips, each 2·25 m wide	
BS 8110 Table 3·20 75% ON COLUMN STRIP	+ Moment/metre width on column strip $= 208 \times \frac{0\cdot75}{2\cdot25}$ $= 69$ kN m/m	
	Design for 0·2 mm crack width Table A2·8 $h = 450$ Use T20 at 200 (1570)* $M_s = 109$ ($f_s = 200$) Cover to lowest layer = 40 Cover to second layer = 60 ∴ Results from table are satisfactory (* See below for distribution steel calculation)	
	Roof slab - minimum reinforcement As before $\rho_{crit} = \frac{1\cdot6}{460} = 0\cdot35\%$ Assume :- $T = 30°C$ $\alpha = 12 \times 10^{-6}$ $\omega = 0\cdot2$ mm $S_{max} = \frac{2}{3} \times \frac{\phi}{2\rho}$ also $S_{max} = \frac{\omega}{\varepsilon} = \frac{0\cdot2}{\frac{1}{2} \times 12 \times 10^{-6} \times 30} = 1110$ mm ∴ $\frac{\phi}{\rho} = 1111 \times 3 = 3333$ mm ∴ ρ_c is given by	
	ø ρ_c% A_s (each face) 12 0·36 810 16 0·48 1080 20 0·60 1350	
	Provide T20 at 200 (1570). Use this arrangement of steel both faces and in both directions to provide both main and shrinkage steel.	T20 at 200 EW EF

ref.	calculations EXAMPLE 6·2 SHEET 9	output
	Shear at Column Head	
	Maximum column load	
	$= 26{\cdot}6 \times 4{\cdot}5 \times 4{\cdot}5 \times 1{\cdot}15$	
	$= 619$ kN	
	Assume that $\gamma_f = 1{\cdot}4$ for all applied loading, as imposed load (after construction) will be unusual	
	Ultimate shear force	
	$= 1{\cdot}4 \times 619$	
	$= 867$ kN	
BS 8110 3·7·6·2	Allow for unequal distribution	
	Ultimate shear force	
	$= 1{\cdot}25 \times 867$	
	$= 1084$ kN	
BS 8110 3·7·7	Assume column size $= 350 \times 350$	
	Maximum shear stress	
	$= \frac{1084 \times 10^3}{4 \times 350 \times 390} = 1{\cdot}99$ N/mm^2	
	Permissible $= 0{\cdot}8\sqrt{f_{cu}} = 4{\cdot}7$ N/mm^2	
	Satisfactory	
	Slab thickness $h = 450$	
	Critical perimeter	
	$= 4 \times 350 + 12 \times 390$	
	$= 6080$	
	Steel ratio $\rho = \frac{100A_s}{bd} = \frac{100 \times 1570}{1000 \times 390} = 0{\cdot}4\%$	
BS 8110 Table 3·9	Concrete grade 35	
	Permissible shear stress $= v_c = 0{\cdot}52$	
	$\therefore$ permissible shear force	
	$= 0{\cdot}52 \times 6080 \times 390 \times 10^{-3}$	
	$= 1233$ kN	NO SHEAR STEEL
	Satisfactory	
	Columns	
	Maximum ultimate column load	
	$= 867$ kN	
	Column height $= 6{\cdot}5$ m	
	Effective height $l_e = 1{\cdot}5 \times 6{\cdot}5$ (unbraced)	
	$= 9{\cdot}75$ m	

ref.	calculations EXAMPLE 6·2 SHEET 10	output
	Assume column size 350 × 350	
	Slenderness ratio $= \frac{le}{h}$	
	$= \frac{9750}{350} = 27{\cdot}9$	
	∴ column is slender	
	Additional moments are:	
BS8110 3·8·3	(a) $M_1 = 0{\cdot}05 \times Nh$	
	$= 15{\cdot}1$ kN m	
	(b) $M_2 = kNh\left(\frac{le}{b}\right)^2/2000$	
	$= 1 \times 867 \times 350\left(\frac{9{\cdot}75}{0{\cdot}35}\right)^2/2000$	
	$= 118$	
	Total M = 15·1 + 118 = 133 kNm	
BS8110 PART 3 CHART 34	$\frac{N}{bh} = \frac{867 \times 10^3}{350^2} = 7{\cdot}07$	
	$\frac{M}{bh^2} = \frac{133 \times 10^6}{350^3} = 3{\cdot}10$	
	$\frac{100A_{sc}}{bh} = 0{\cdot}7$	
	Minimum steel = 0·4%	
	$A_{sc} = 350^2 \times 0{\cdot}007 = 858\ \text{mm}^2$	
	Use 4-T20 (1260)	4-T20
	Links R8 at 300	R8 at 300
	Note: Vertical bars being 6m. long need to be reasonably stiff to avoid reinforcement cage being too flexible	
	Column Bases	
	Service load = 619	
	S.W. column = 19	
	S.W. base* = 30 * extra over soil	
	668 kN	
	Use base 2250 × 2250 × 1000	
	Moment due to slender column = (say) 50	
	Soil pressure under base	
	$= \frac{668}{2{\cdot}25^2} \pm \frac{50 \times 6}{2{\cdot}25^3}$	
	$= 132 \pm 26$	MAX. SOIL PRESSURE
	$= 158$ or 106 (kN/m²) Satisfactory	158 kN/m²
	Note: total pressure	
	= 158 + pressure due to contained water	
	= 158 + 10 × 6·5	
	= 223 kN/m²	

ref.	calculations EXAMPLE 6·2 SHEET 11	output

Wall Footings



Water pressure = 10h

Soil pressure = 6h

Surcharge = 15·8 kN/m^2

Roof slab load = $26{\cdot}6 \times \frac{3{\cdot}75}{2}$ = 50 kN/m

D.L. only = $10{\cdot}8 \times \frac{3{\cdot}75}{2}$ = 20 kN/m

Case 1

Tank full - no soil

Take moments about A

Restoring forces

water	= 10 x 6·5 x 2·2	= 143	x 3·9	= 558
Wall	= 24 x 6·5 x 0·6	= 94	x 2·4	= 226
Base	= 24 x 5·0 x 0·8	= 96	x 2·5	= 240
Roof (min. load)		20	x 2·4	= 48
		353		1072

Overturning moment

$= \frac{1}{6} \times 10 \times 6{\cdot}5^3$ = 457

615

Factor of safety against overturning

$= \frac{1072}{457}$

= 2·35 Satisfactory
(> 2·0)

ref. | calculations EXAMPLE 6·2 SHEET 12 | output

Case 2

Tank empty – soil + surcharge

Take moments about B

Restoring forces

Soil	18 × 6·5 × 2·0	=	234	× 4·0	=	936
Surcharge	15·8 × 2·0	=	32	× 4·0	=	126
wall	24 × 6·5 × 0·6	=	94	× 2·6	=	243
Base	24 × 5·0 × 0·8	=	96	× 2·5	=	240
Roof		=	20	× 2·4	=	48
			476			1593

Overturning moment

$$= \frac{1}{6} \times 6 \times 6\cdot5^3 + \frac{1}{2} \times \frac{15\cdot8}{3} \times 6\cdot5^2 = 387$$

$$\underline{1206}$$

(soil) (surcharge)

Factor of safety against overturning

$$= \frac{1593}{387} = 4\cdot1$$

Satisfactory (> 2·0)

Case 3

Tank full + soil + surcharge

Take moments about B

Restoring forces

As case 2	476			1593
Water	143	× 1·1	=	157
Roof load (imposed)	30	× 2·4	=	72
	649			1822

Net overturning moment about B

= 387 – 457 = –70

1892

By inspection – not critical

ref. calculations EXAMPLE 6·2 SHEET 13 output

Soil pressure under base

Calculate moment about centre line of base

Case 1

$$M_c = 353\left(\frac{615}{353} - 2\cdot5\right) = -267$$

Soil pressure

$$f = \frac{353}{5\cdot0} \pm \frac{267 \times 6}{5\cdot0^2}$$

$$= 71 \pm 64$$

$$= 135 \text{ or } 7$$

Case 2

$$M_c = 476\left(\frac{1206}{476} - 2\cdot5\right) = -6$$

Soil pressure

$$\rho = \frac{476}{5\cdot0} \pm \frac{16}{5\cdot0^2} \times 6$$

$$= 95 \pm 1$$

$$= 96 \text{ or } 94$$

Case 3

$$M_c = 649\left(\frac{1898}{649} - 2\cdot5\right) = 270$$

Soil pressure

$$f = \frac{649}{5\cdot0} \pm \frac{270 \times 6}{5\cdot0^2}$$

$$= 130 \pm 65$$

$$= 195 \text{ or } 65$$

Maximum soil pressure = 195 kN/m^2

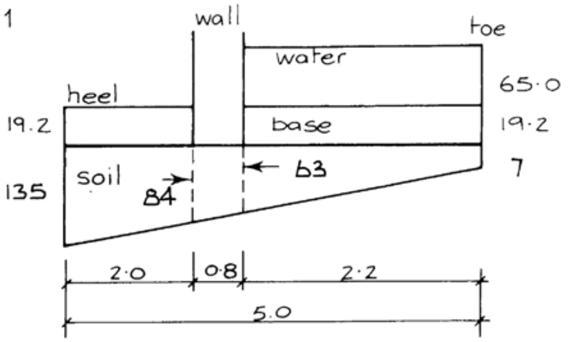
Satisfactory

Note: Depending on soil conditions, the base could be designed more economically. It may also be an advantage in wet soil conditions to design the base with no outward projection.

ref. | calculations EXAMPLE 6·2 SHEET 14 | output

Footing Reinforcement

Case 1



Moment at root of toe

$$= -(65{\cdot}0 + 19{\cdot}2) \times 2{\cdot}2^2 \times 0{\cdot}5$$

$$+ 0{\cdot}5 \times 63 \times 2{\cdot}2^2 \times \tfrac{1}{3} + 0{\cdot}5 \times 7 \times 2{\cdot}2^2 \times \tfrac{2}{3}$$

$$= -142 \text{ kNm/m}$$

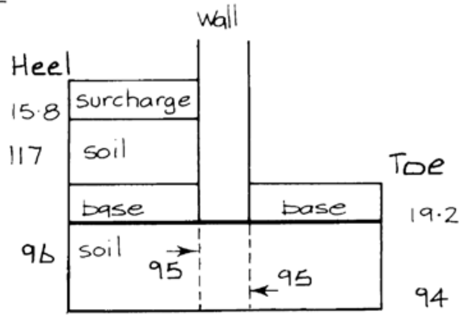
Moment at root of heel

$$= 0{\cdot}5 \times 135 \times 2{\cdot}0^2 \times \tfrac{2}{3} + 0{\cdot}5 \times 84 \times 2{\cdot}0^2 \times \tfrac{1}{3} -$$

$$0{\cdot}5 \times 19{\cdot}2 \times 2{\cdot}0^2$$

$$= 198 \text{ kN m/m}$$

Case 2



Moment at root of toe

$$= 0{\cdot}5 \times 95 \times 2\,2^2 \times \tfrac{1}{3} + 0{\cdot}5 \times 94 \times 2{\cdot}2^2 \times \tfrac{2}{3}$$

$$- 0{\cdot}5 \times 19{\cdot}2 \times 2{\cdot}2^2$$

$$= 182 \text{ kN m/m}$$

ref. calculations EXAMPLE 6.2 SHEET 15 output

Moment at root of heel

$$= -0.5 \times (15.8 + 117 + 19.2) \times 2.0^2$$
$$+ 0.5 \times 96 \times 2.0^2 \times \frac{2}{3} + 0.5 \times 95 \times 2.0^2 \times \frac{1}{3}$$

$$= -113 \text{ kNm/m}$$

Case 3

By inspection, not critical

Heel reinforement

Top reinforcement

$M = 113$

$M_u = 113 \times 1.4 = 158$

$h = 800$

$d = 735$

$$x_1 = \frac{\left(1 - \sqrt{1 - 0.7 \times \frac{158}{2120}}\right)}{0.9}$$

$= 0.03$

$z_1 = 1 - 0.01 = 0.99$

but maximum value $= 0.95$

$$\therefore A_{st} = \frac{113 \times 10^6}{0.95 \times 735 \times 0.87 \times 460} = 404$$

T16 at 200

Use T16 at 200 top (1005)

Bottom reinforcement

$M = 198$

$M_u = 198 \times 1.4 = 277$

$x_1 = 0.05$

$z_1 = 0.095$

$$A_{st} = \frac{277 \times 10^6}{0.95 \times 735 \times 0.87 \times 460} = 991$$

T16 at 200

Use T16 at 200 bottom (1005)

ref. | calculations | EXAMPLE 6·2 SHEET 16 | output

Toe reinforcement

Top reinforcement

M = 142

Cracking not significant

Use T20 at 175 (1800)

T20 at 175

Bottom reinforcement

M = 182

$M_u = 182 \times 1{\cdot}4 = 255$

$$x_1 = \frac{\left(1 - \sqrt{1 - 0{\cdot}7 \times \frac{255}{2120}}\right)}{0{\cdot}9} = 0{\cdot}05$$

$z_1 = 0{\cdot}95$

$$A_s = \frac{255 \times 10^6}{0{\cdot}95 \times 735 \times 0{\cdot}87 \times 460} = 913$$

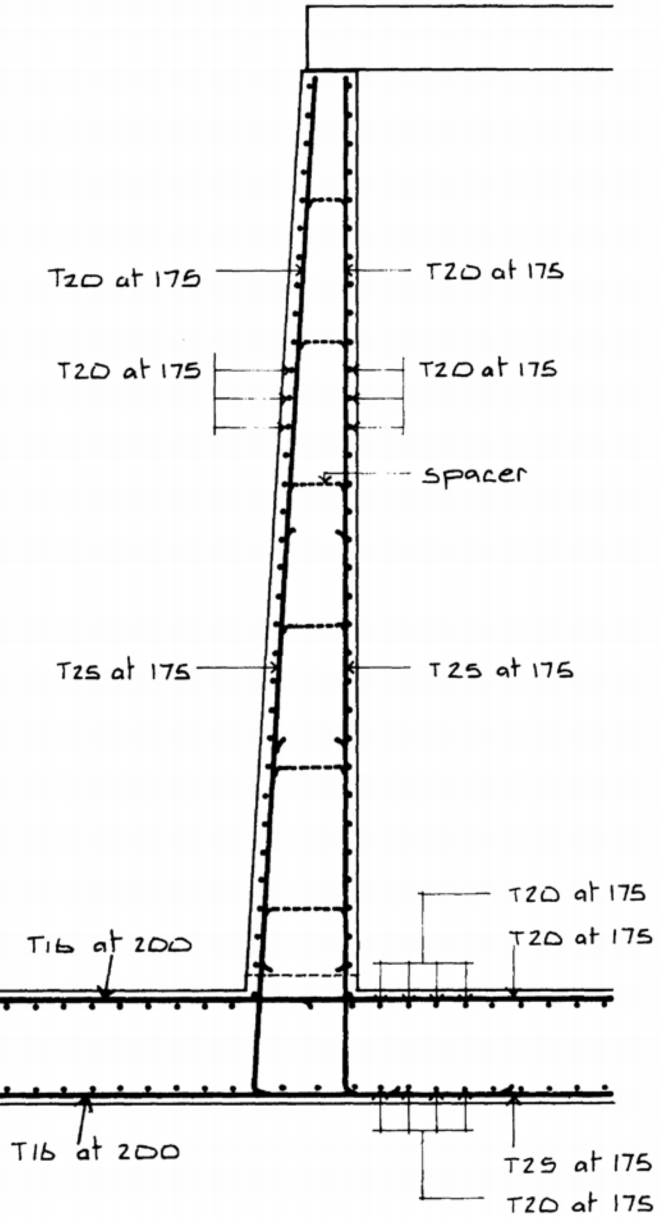
Use T25 at 175 to suit wall reinforcement (2810)

T25 at 175

ref.	calculations EXAMPLE 6·2 SHEET 17	output
	Reservoir Floor Slab	
	Floor is divided into 4·5 m panels by movement joints Assuming uniform ground conditions, the floor slab is uniformly loaded and has no transverse bending stresses	
	To provide a reasonable thickness of concrete, and allowing for possible construction tolerances, use a slab 200 mm thick	
	$\rho_{crit} = \frac{f_{ct}}{f_y} = \frac{1\cdot6}{460} = 0\cdot35\%$	
	Provide top reinforcement only based on a surface zone 100mm deep (i.e. one half of slab depth)	
BS 8007 A.3 and Fig A·2	$A_s = 0\cdot35\% \times 100 \times 1000$ $= 350 \text{ mm}^2/\text{m}$	
	Use a welded mesh fabric with 10 mm wires in each direction at 200 mm spacings (393)	MESH A393
	Bottom zone does not require reinforcement	

ref. **calculations** EXAMPLE 6·2 SHEET 18. **output**

REINFORCEMENT DETAIL.



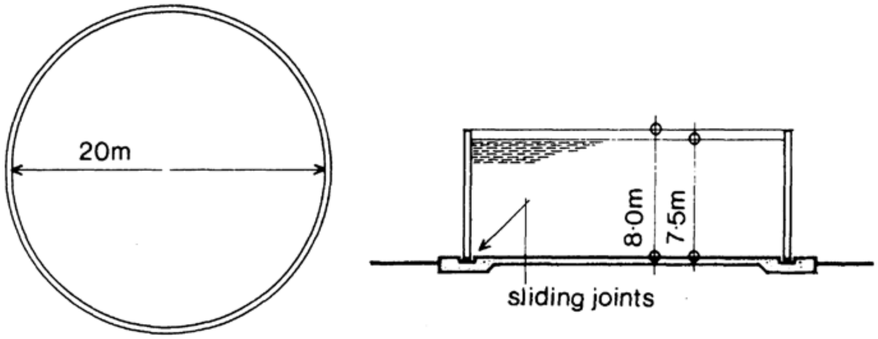
6.3 Design of a circular prestressed concrete tank

Design a circular prestressed concrete tank to contain water for fire-fighting purposes (Fig. 6.3).

Tank diameter = 20.0 m
Height of water = 7.5 m

The tank is to be constructed entirely above ground level.

This example is intended to show the application of prestressed concrete design to liquid-retaining structures.

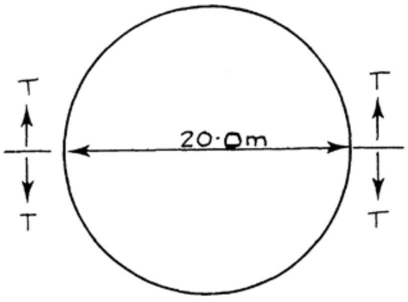


plan section

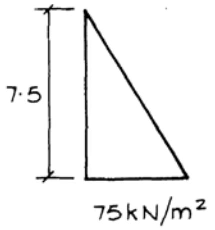
Fig. 6.3 Layout of circular prestressed concrete tank

ref.	calculations EXAMPLE 6·3 SHEET 1.	output
	DESIGN OF PRESTRESSED CIRCULAR TANK	
	Internal diameter of tank = 20·0 m	
	Maximum depth of water = 7·5 m	
	Allow freeboard of 0·5 m	
	Tank is constructed above ground	
	Provide a sliding joint at the foot of the wall (Fig. 6·3)	
	Materials	
	For prestressed concrete construction a high strength concrete is required	
	Use grade 40 with a minimum cement content of 300 kg/m³	40 GRADE CONCRETE
	Reinforcement	
	Use high strength low relaxation prestressing strands to BS 5896 : 1980, and grade 460	7 WIRE STANDARD STRAND
	high yield deformed reinforcement to BS 4449	GRADE 460 STEEL
	Exposure Conditions	
BS 8007 2·7·3	Severe exposure	
BS 8007 4·3	The basic requirement is to ensure that there is a circumferential compression in the concrete when the tank is full of water.	
	The prestressing cables will be placed outside the walls and protected with sprayed concrete	
	Concrete cover to normal reinforcement = 40	COVER = 40
	Wall Thickness	
	To enable the concrete to be placed with 4 layers of normal reinforcement and to prevent local percolation, use a wall thickness of 225	h = 225

ref. calculations EXAMPLE 6·3 SHEET 2 output



PLAN



WATER PRESSURE

Maximum water pressure $= 10 \times 7{\cdot}5$

$= 75\ kN/m^2$

Maximum circumferential tension due to water pressure

$$T = \frac{75 \times 20}{2} = 750\ kN/m$$

The extra force required to provide a net circumferential compressive stress in the concrete of 1 N/mm² is

$$T_1 = \text{Area} \times \text{stress}$$
$$= 225 \times 1000 \times 1{\cdot}0 \times 10^{-3}$$
$$= 225\ kN/m$$

Total force required

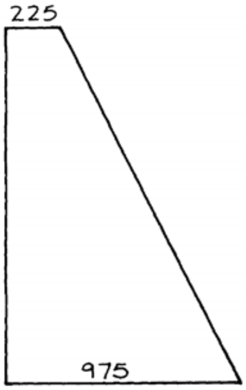
$$= T + T_1$$
$$= 750 + 225$$
$$= 975\ kN/m$$

At top of tank water pressure = zero and $T = 0$

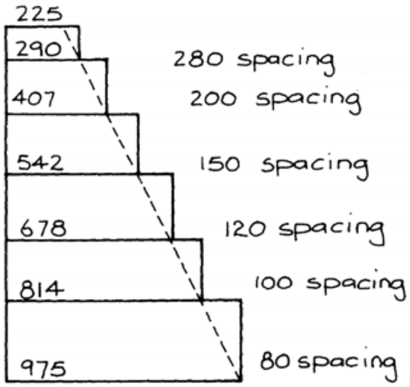
Total force required is

$$T_1 = 225\ kN/m.$$

ref. | calculations EXAMPLE 6·3 SHEET 3 | output



theoretical minimum circumferential force required



actual forces provided by tensions at varying spacings up wall height (Details are calculated later on page 6·3 (8))

Prestressing Cables

BS 5896 — Use 12·5 mm 7 wire low relaxation strand.

Area = 93 mm²

Characteristic load = 164 kN

$$\text{Characteristic strength} = \frac{164 \times 10^3}{93}$$

$$= 1770 \text{ N/mm}^2$$

Initial jacking force (say) = 0·7 × 164

= 115 kN

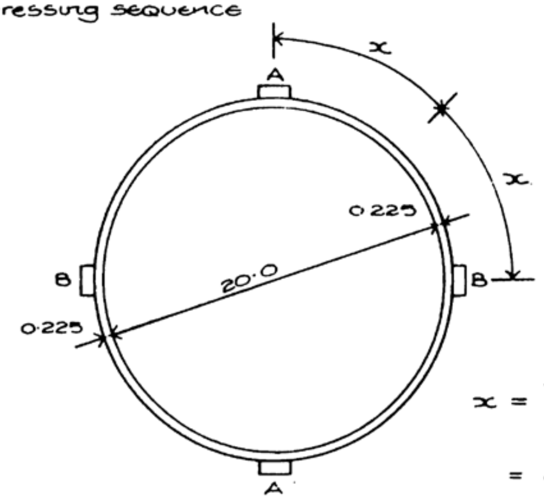
P_o = 115

Initial stress = 0·7 × 1770

= 1240 N/mm²

ref. **calculations** EXAMPLE 6·3 SHEET 4 **output**

Stressing sequence



$x = \frac{\pi \times 20{\cdot}45}{8}$

$= 8{\cdot}0\text{m}$

$x = 8{\cdot}0\text{m.}$

The prestressing strands are anchored on concrete projections (pillasters) at the quarter points of the circumference

Each cable extends half-way round the circumference and is stressed from both ends. Both cables at each level are stressed simultaneously and four jacks are required

The jacking points for alternate cables (in elevation) are at A and then B etc.

The maximum friction loss is half-way between jacking points for each cable and by averaging for two consecutive cables AA and BB, the point of average maximum loss will be at a point defined by $x = 8{\cdot}0\text{m}$.

ref. | **calculations** EXAMPLE 6·3 SHEET 5. | **output**

Loss of prestressing force due to friction.

Assume constant for friction due to irregularities in ducts :–

$K = 33 \times 10^{-4}$ /m

Coefficient of friction between tendon and duct

$\mu = 0 \cdot 30$

Radius of curvature of tendons

$$R = \frac{20 \cdot 45}{2}$$

$$= 10 \cdot 2 \text{ m}$$

Point of average maximum loss of stress

Factor $\left(Kx + \frac{\mu x}{R}\right)$ $\qquad x = 8 \cdot 0$ m

$$= 8 \cdot 0 \left(33 \times 10^{-4} + \frac{0 \cdot 3}{10 \cdot 2}\right)$$

$$= 0 \cdot 262$$

If the initial prestressing force $= P_o$, the force after friction losses at x is

$$P_x = P_o \times e^{-0 \cdot 262}$$

$$= 0 \cdot 77 P_o$$

$\therefore \qquad P_x = 0 \cdot 77 \times 115$

$$= 88 \cdot 5 \text{ kN}$$

and stress in strand after friction losses

$$= 0 \cdot 77 \times 1240$$

$$= 955 \text{ N/mm}^2$$

ref.	calculations EXAMPLE 6·3 SHEET 6	output
	Prestressing Losses	
	(1) Loss due to creep of low relaxation strand = 2% (from manufacturers' catalogue)	2·0%
	(2) Loss due to elastic contraction of concrete.	
	Modulus of elasticity of concrete $= 31\,\text{kN/mm}^2$ Modulus of elasticity of steel $= 195\,\text{kN/mm}^2$ Modular ratio $\alpha_e = \frac{195}{31} = 6{\cdot}3$	
	Maximum elastic stress in concrete with tank empty after losses $= \frac{975 \times 10^3}{225 \times 10^3} = 4{\cdot}3\,\text{N/mm}^2$	
	Assuming 10% total losses, concrete stress at transfer $= \frac{4{\cdot}3}{0{\cdot}9} = 4{\cdot}8\,\text{N/mm}^2$	
	∴ elastic strain in concrete $= \frac{4{\cdot}8}{31} \times 10^{-3}$ which is equal to loss of strain in steel ∴ loss of stress in steel $= \frac{4{\cdot}8 \times 195}{31}$ or $4{\cdot}8 \times 6{\cdot}3 = 30{\cdot}2\,\text{N/mm}^2$	
	As the tank will be post-tensioned, the final strands will be tensioned after nearly all the elastic shortening in the concrete has taken place, therefore the average loss may be taken as half the value calculated above. i.e. loss $= \frac{1}{2} \times 30{\cdot}2 = 15{\cdot}1\,\text{N/mm}^2$ Initial stress in strand $= 955\,\text{N/mm}^2$ (after friction losses)	
	∴ % loss $= \frac{15{\cdot}1 \times 100}{955} = 1{\cdot}6\%$	1·6%

ref.	calculations EXAMPLE 6·3 SHEET 7.	output
	(3). Loss due to shrinkage of concrete Shrinkage strain in concrete = 200×10^{-6} ∴ loss of strain in strands = 200×10^{-6} ∴ loss of stress in strands = $(\text{strain}) \times \alpha_{es} = 200 \times 10^{-6} \times 195 \times 10^{3}$ $= 39\ \text{N/mm}^2$	
	$\%\ \text{loss} = \frac{39 \times 100}{955} = 4 \cdot 1\%$	4·1%
	(4). Loss of stress due to creep of concrete Stress in concrete at transfer = $4 \cdot 8\ \text{N/mm}^2$ Proportion of cube strength $= \frac{4 \cdot 8}{40}$ $= 0 \cdot 12$ As this is less than $\frac{1}{3}$, the creep values need not be increased Creep strain = 36×10^{-6} Loss of stress in strand = $(\text{strain}) \times \alpha_{es} = 36 \times 10^{-6}\ 195 \times 10^{3}$ $= 7.0$	
	$\%\ \text{loss} = 7 \cdot 0 \times \frac{100}{955} = 0 \cdot 7\%$	0·7%
	Total losses = $(2 \cdot 0 + 1 \cdot 6 + 4 \cdot 1 + 0 \cdot 7)\%$ $= 8 \cdot 4\%$	TOTAL LOSSES 8·4%
	This is less than the value of 10% which was assumed on page 6 and is satisfactory Stress in strand after friction losses = $955\ \text{N/mm}^2$ ∴ final effective stress $= 955\left(\frac{100 - 8 \cdot 4}{100}\right) = 875\ \text{N/mm}^2$	
	and force/tendon $= 875 \times 93 \times 10^{-3} = 81 \cdot 4\ \text{kN}$ (area)	EFFECTIVE TENDON FORCE 81·4KN.

ref. calculations EXAMPLE 6·3 SHEET 8 output

Number of tendons per metre at bottom of tank

$$= \frac{\text{force required}}{\text{force / tendon}} = \frac{975}{81{\cdot}4} = 12$$

$\therefore$ spacing $= \frac{1000}{12}$ say $= 80$ mm

Number of tendons per metre at top of tank

$$= \frac{225}{81{\cdot}4} = 2{\cdot}76$$

$\therefore$ spacing $- \frac{1000}{2{\cdot}76}$ say $= 280$ mm

Intermediate spacings can be calculated from the diagram on page 6·3 (3)

Vertical Design

Tank empty

When the tank is empty, moments will be induced in the vertical direction by the larger prestressing forces near to the foot of the wall, as compared with the smaller prestressing forces near to the top of the wall

The maximum moment induced may be assessed as being numerically equal to one half of the moment induced by a pinned base condition.

ACI Tables

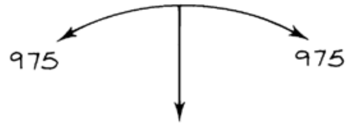
Table A3·7

$$\frac{h^2}{dt} = \frac{7{\cdot}5^2}{20 \times 0{\cdot}225} = 12{\cdot}5$$

Maximum value of coefficient = 0·0037

Radial pressure due to prestressing

$$= \frac{\text{ring force}}{\text{radius}}$$



ref. | calculations EXAMPLE 6·3 SHEET 9 | output

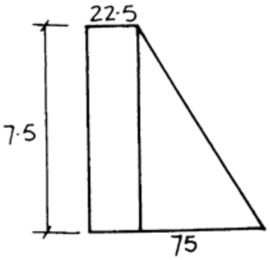
At top of tank, ring force $= 225 \text{ kN/m}$

At bottom of tank, ring force $= 975 \text{ kN/m}$

$\therefore$ Radial pressure at top $= \frac{225}{10} = 22\cdot5$

Radial pressure at bottom $= \frac{975}{10} = 97\cdot5$

$\therefore$ Assume uniform load of $22\cdot5 \text{ kN/m}^2$ with a triangular load of 75 kN/m^2



$q = 22\cdot5 \text{ kN/m}^2$

$w_g = \frac{75}{7\cdot5} = 10 \text{ kN/m}^3$

$M = K\,(w_g h^3 + q h^2)$

$\therefore$ Moment (hinged condition)

$= 0\cdot0037\,(10 \times 7\cdot5^3 + 22\cdot5 \times 7\cdot5^2)$

$= 20\cdot3 \text{ kNm/m}$

The design moment to be used is one half of $20\cdot3 = 10\cdot2 \text{ kNm/m}$ (tension on the outside)

Partially prestressed condition:

During the prestressing operations the tank will be compressed non-uniformly at each level
The vertical stress produced may be estimated as numerically equal to $0\cdot3 \times$ the ring compressive stress at transfer

i.e. $0\cdot3 \times 4\cdot8 = 1\cdot44 \text{ N/mm}$*

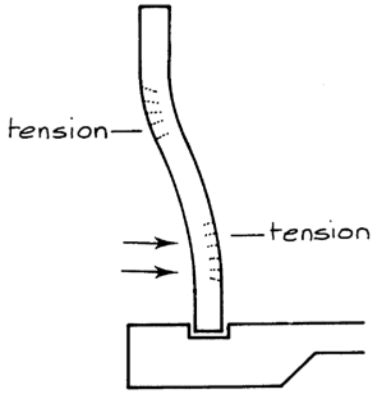
$$\text{Moment} = f_z = \frac{1\cdot44 \times 10^3 \times 225^2}{6 \times 10^6}$$

$$= 12\cdot1 \text{ kNm/m}$$

*BS 8007 restricts this value to 1·0. The tank may therefore have to be stressed in two operations.

ref. | calculations EXAMPLE 6·3 SHEET 10 | output

This moment can be assumed to act in causing tension on both faces

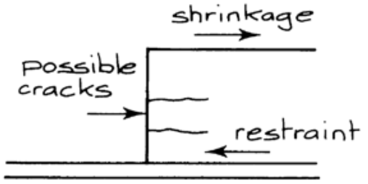


Section through tank when partially prestressed

Design of Reinforcement

Use normal reinforcement (ie. not prestressing)

Minimum reinforcement may be assumed to be 0·35%, as there is no restraint in the vertical direction except for friction at the base which causes some tension in the partially cast condition.



Equally horizontal reinforcement is necessary to control cracking before prestressing, and is used to support the vertical steel.

Horizontal reinforcement

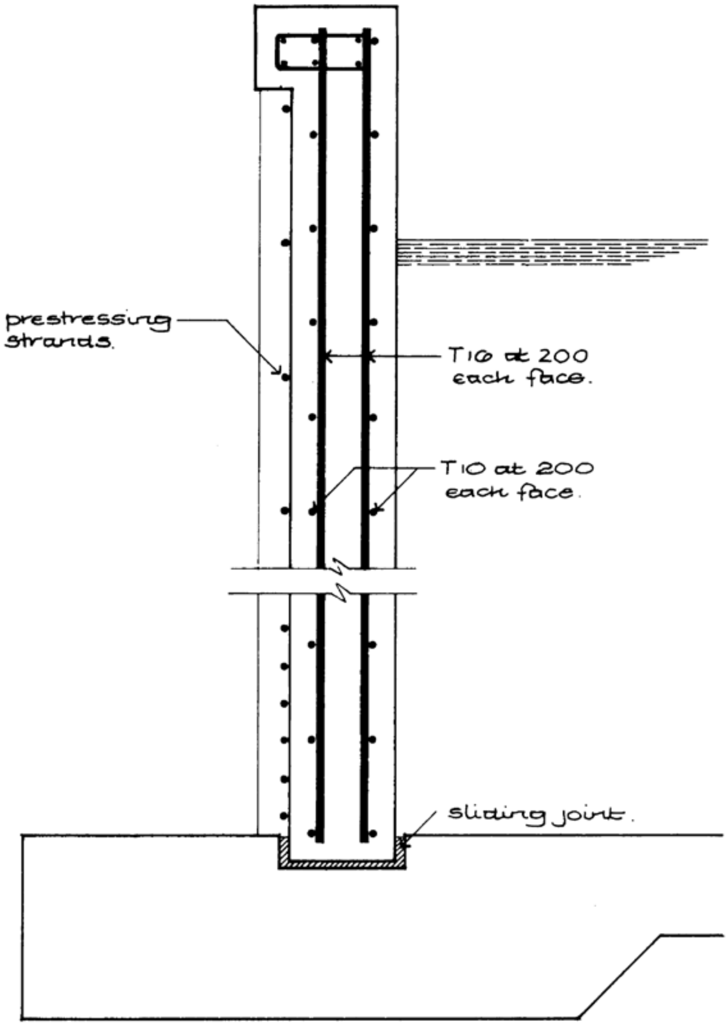
$0{\cdot}35\% \times 225 \times 1000 = 787\ \text{mm}^2$

Use T10 at 200 each face (785)

T10 at 200

ref.	calculations EXAMPLE 6·3 SHEET 11	output
	Vertical reinforcement Maximum moment to be resisted = 12·1 kNm/m	
TABLE A2·2	Design crack width = 0·2 mm Table A2·2 $h = 225$ T12 at 200 $M = 20\cdot 7$ $f_s = 241$	
	For $M = 12\cdot 1$ $f_s = \frac{12\cdot 1}{20\cdot 7} \times 241 = 141$	
	This is satisfactory Use T16 at 200 vertically each face (T12 vertically would be too flexible)	T16 at 200
	If there are a number of tanks to construct, it may be economical to use welded wire fabric, which can be specially fabricated to provide the steel arrangements and sheet sizes required.	

ref.	calculations. EXAMPLE 6·3 SHEET 12.	output
	REINFORCEMENT DETAILS.	



7

Testing and rectification

7.1 Testing for watertightness

The design and construction of liquid-retaining structures require close attention to detail by both the designer and contractor but, in spite of the best intentions of both parties, errors and omissions can occur. Equally, although the design theory outlined in this book has been used successfully for many structures, random occurrences and unfavourable statistical conjunctions can result in a structure which is less than completely liquid-tight. It is therefore necessary to test the structure after completion, to ensure that it is satisfactory and that it complies with the specification.

The method of test depends on the visibility and position of the elements of the structure. The walls of overground structures can be inspected for leaks on the outer face and, if the walls are finally to be backfilled with soil, the inspection can be made before the fill is placed. The walls of underground structures can be inspected if there is sufficient working space available. The floor slabs of all structures built on soil cannot be inspected for leaks, and other methods of test have to be used (42). The floor of an elevated reservoir (or water tower) can be inspected in the same way as walls, as can the underside of a flat reservoir roof. If the structure is designed to exclude rather than retain water, it is possible to inspect the inside faces of the walls and floors but rarely possible to ensure that liquid is available to make a test, and hardly ever possible to take remedial measures from the outside. An example of this situation is that of a basement of a building which is designed to exclude ground water. Detailed methods of testing are described in the following sections.

7.2 Definition of watertightness

The term 'watertightness' although descriptive, is not sufficiently precise for the purposes of a contract specification. Essentially, a watertight structure is built to contain (or exclude) a liquid, but some loss of liquid is inevitable due to evaporation or slow diffusion through the concrete. Also, actual leaks may occur through fine cracks in the concrete. These may heal autogenously (i.e.

without any treatment). This occurs as water percolates through the crack and dissolves calcium salts from the cement. As the process continues, the crack is slowly filled and eventually the water penetration ceases. The process may take up to about one week with cracks of 0.1 mm width, but up to three weeks for cracks of up to 0.2 mm width. Cracks over 0.2 mm thick may not self-seal at all. The result of autogenous healing is a white excrescence along the line of the crack, but no further loss of liquid. This may be acceptable as a permanent feature in some types of structure such as underground tanks, but could not be allowed in the walls of an elevated water tower. A further form of leakage consists of damp patches on the surface of a wall. The liquid flow is very small, but the appearance may not be acceptable.

It is essential that the required standard of watertightness is clearly described in the contract specification so that there is no misunderstanding of the quality of result required from the contractor.

7.3 Water tests

A completed structure may be tested by filling with water and measuring the level over a period of time. The concrete in the structure must be allowed to attain its design strength before testing commences, and all outlets must be sealed to prevent loss of water through pipes, overflows and other connections. The structure should also be cleaned.

The structure is slowly filled to its normal maximum operating level. If the structure is filled too quickly, the sudden increase in pressure is likely to cause cracking. As a guide, a swimming pool or relatively small tank could be filled over a period of three days, but a large reservoir will take much longer to fill because of the volume of water required. BS 8007 limits the rate of filling to a uniform rate of not greater than 2 m in 24 hours.

To allow the concrete to become completely saturated with water, a stabilizing period is allowed after filling has been completed. The length of the stabilizing period depends on the design surface crack width and hence the time required to complete any autogenous healing which may be necessary. For a design crack width of 0.1 mm, a period of one week may be required, but for a design crack width of 0.2 mm, a period of up to three weeks is necessary. These times may be adjusted as appropriate. If it is obvious that there is no leakage through cracks after some days, it may be possible to commence the record test somewhat earlier. At the commencement of the test, the level of the water is recorded, and subsequently each day for a further period of seven days. The difference in level over the period of seven days is then used to assess the result of the test. The levels may be measured by fixing scales to the walls, or by making marks on the walls above water line and measuring down to water level with a moveable scale or other device. The level should be recorded at four positions, but with a large reservoir at eight to twelve positions, to guard against errors in reading and local settlements.

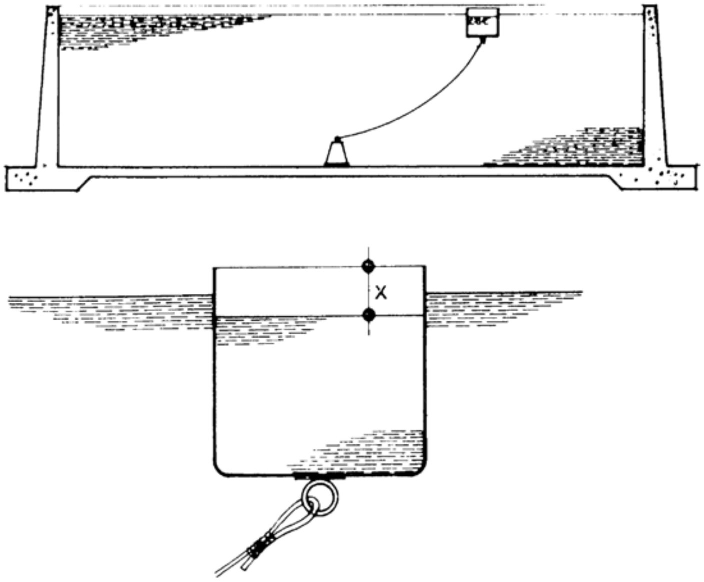


Fig. 7.1 Arrangements for water test

An open structure (or a closed structure where the air above the water is affected by wind movements) may lose moisture by evaporation, or may gain water due to rainfall. In assessing the results of the water level readings during the test, allowance must be made for these variations. A simple method of achieving this is to moor watertight containers 80% filled with water at points on the water surface. The water surface inside the container is subject to the same gains and losses as the water in the main reservoir. By taking measurements (x) of the water level in the container from the top edge of the container, the gains or losses due to rainfall and evaporation in the main reservoir may be assessed (Fig. 7.1).

It will be apparent that a degree of honesty and care is necessary when carrying out tests of this nature, and the daily measurement of water levels during the test will assist in detecting any unusual occurrences. The author has had experience of a test where the water level was appreciably higher at the end of the test than at the beginning.

7.4 Acceptance

A water test will enable a net loss of water to be measured due to leakage and further absorption into the concrete structure. The acceptable fall in water level should be stipulated by the designer before the test is commenced. For many structures, the maximum acceptable limit may be taken as 1/1000 of the average depth of the water. BS 8007 recommends a value of 1/500 of the average water depth or 10 mm or other specified amount. The Australian standard adopts similar values. It is not possible to set a limit less than about

3 mm due to the difficulty of making a sufficiently accurate measurement.

If the test is judged to be unsatisfactory after seven days, and if the daily readings indicate that the rate of loss of water is reducing, the designer may decide to extend the test period by a further seven days. If the net loss of water is then no greater than the specified value during the second period of seven days, the test may be considered satisfactory. If the test is judged to be unsatisfactory, then it should be repeated after measures have been taken to locate and deal with leakage.

A successful water level test is a necessary but not sufficient criterion for accepting the structure as satisfactory. If seepage can be observed from the 'dry' side of the walls or if damp patches are present, then remedial work will be required. The condition of the surface of elements of the structure should be assessed by reference to the contract specification. The structure should not be accepted as satisfactory until the specification has been satisfied in each particular.

7.5 Remedial treatment

It is particularly difficult to isolate defects in floor slabs, but joints and areas where the concrete surface is irregular or honeycombed should be inspected very thoroughly. When water has been drained out of a structure, and the surface is drying, areas containing defects may be the last parts to remain wet or damp, due to water being trapped in the defective area.

Small leaks and damp patches are usually self-healing after two to three weeks. After the healing is complete, accretions on the outside of the leak may be scraped off the surface. More persistent leaks require treatment with proprietary products, preferably from the water face. Chemicals are available which are applied to cracks as a slurry and are drawn into the crack by the water flow. Fine crystals are formed which close the crack. A similar effect occurs when the slurry is applied to a porous area. Areas of severe honeycombing or wide cracks may be repaired with pressure grouting techniques or, if there is severe leakage, a whole section may need to be cut out and replaced.

8

Vapour Exclusion

8.1 The problem

In recent years there has been a trend towards ever larger buildings in city centres, and because of a shortage of land, there are frequently one or more basements below ground level. The basements may be used for car parking, storage, or as office or shop accommodation. A further recent phenomenom has been the slow rise in ground water levels in many cities in the UK following a reduction in pumping for industrial uses.

The previous chapters in this book have addressed the design of structures to retain aqueous liquids, and by using the same principles, it is possible to design structures to exclude ground water — as in a basement situation. When a structure is designed to BS 8007 to exclude liquid, it is accepted that there may be a damp patch or two on the walls, but there should be no leakage of water. A basement in this condition will be acceptable for use as a car park, but for use as a retail sales floor, passage of water vapour (or damp) must be prevented. For reasons of health, the UK Building Regulations require all habitable rooms to be designed so that vapour may not pass through the external enclosure below ground level.

Properly compacted concrete will prevent the passage of water, but will still allow water vapour to migrate through the structural elements, particularly if the basement is heated and/or ventilated. If water vapour is to be excluded, then additional measures are necessary.

The following sections deal with the ways in which a vapour excluding concrete structure may be achieved. It is not possible in this book to deal with all the details applicable to particular materials, but guidance may be obtained from manufacturers' information sheets, and also from BS 8102: Code of Practice for Protection of structures against water from the ground. To stay within the limits of the subject of this book, it is assumed that the underground structure is of new *in situ* reinforced concrete construction.

8.2 Design requirements

Although it may seem that a vapour excluding basement is sufficiently well-defined by its description, this is not so. Before design commences, discussion is necessary between the designer and the building owner to decide on the level of protection required. A balance must be struck between satisfactory performance of the structure in use and the cost of providing the protection. The consequences of failure and the anticipated life of the building (or contents) are also part of the considerations.

BS 8102 defines the levels of protection required for various specified uses as types A, B and C. These are reproduced in Table 8.1 together with the grades, usages, and corresponding performance levels. To satisfy each of these criteria, a structure may be designed in several ways, each being designated by the type letter as follows:

Type A — Tanked protection
The structure itself is not water excluding and protection is provided by a membrane system applied either externally or internally. The tanking may either be water and vapour excluding or only water excluding.

Type B — Structurally integral protection
The structure is designed to BS 8007 to be water excluding but will not be vapour excluding unless an external or internal membrane system is applied.

Table 8.1 Guide of level of protection to suit basement use

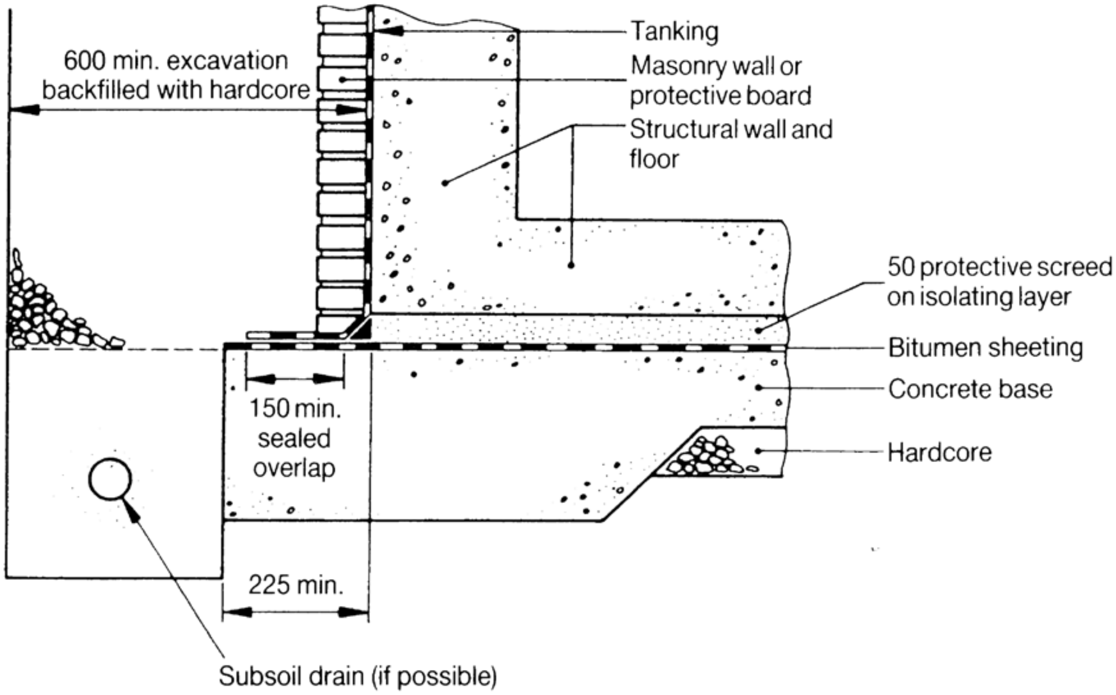
Grade	Basement usage	Performance level	Form of construction
1	Car parking; plant rooms (excluding electrical equipment); workshops	Some seepage and damp patches tolerable	Type B. Reinforced concrete design in accordance with BS 8110
2	Workshops and plant rooms requiring drier environment; retail storage areas	No water penetration but moisture vapour tolerable	Type A. Type B. Reinforced concrete design in accordance with BS 8007.
3	Ventilated residential and working areas including offices, restaurants etc., leisure centres	Dry environment	Type A. Type B. With reinforced concrete design to BS 8007. Type C. With wall and floor cavity and DPM
4	Archives and stores requiring controlled environment	Totally dry environment	Type A. Type B. With reinforced concrete design to BS 8007 plus a vapour proof membrane. Type C. With ventilated wall cavity with vapour barrier to inner skin and floor cavity with DPM

Type C — Drained protection
A drained and possibly ventilated cavity wall construction is provided together with drained cavity floor construction. The floor construction also includes a damp excluding membrane.

Figures 8.1–8.4 show examples of these types of construction, and Table 8.1 includes recommendations of their application for the various grades of protection required. It should be noted that more than one type of construction is available for each level of protection. As stated above, it is not possible to 'design' reinforced concrete to prevent the passage of vapour and hence an additional barrier of an appropriate material is necessary. The essential feature of the barrier is that it should be continuous, with particular attention being given to the junction between floor and walls and to the effective sealing of any pipes or services which pass through the walls or floor (see Figure 8.6).

8.3 Assessment of site conditions

The water and water vapour that are to be excluded from a basement come from ground water, local surface water, or fractured water supply or drainage pipes. It is important to provide protection from rain falling on the surfaces



All dimensions are in millimetres

Fig. 8.1 External membrane protection

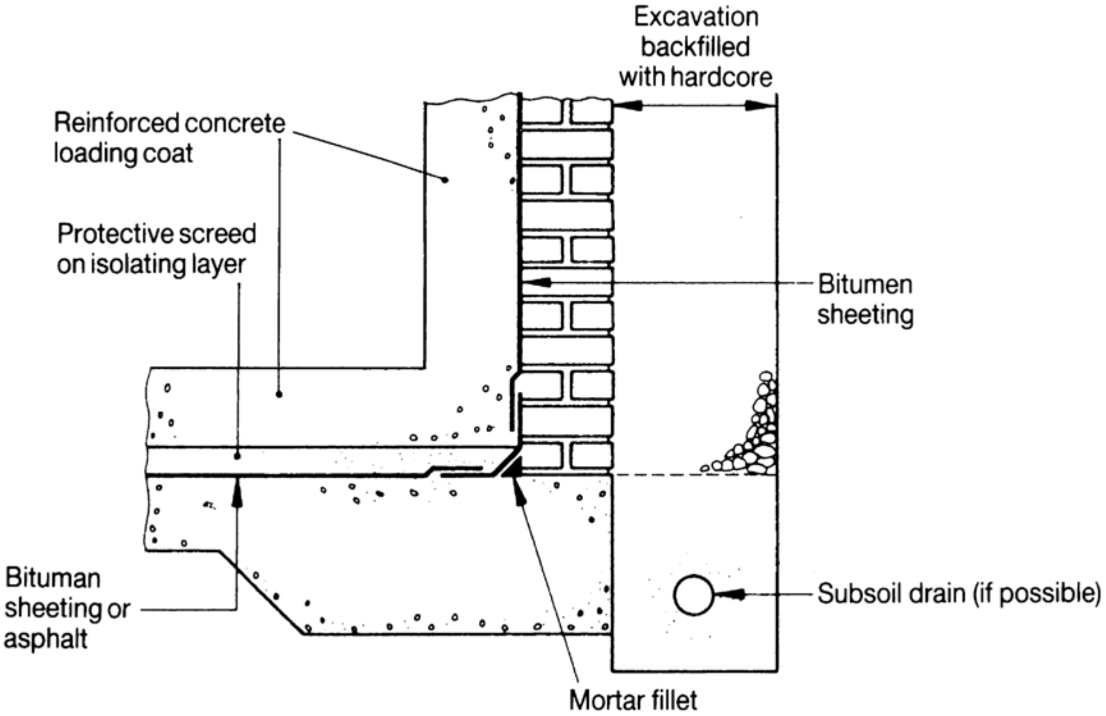


Fig. 8.2 Internal membrane protection

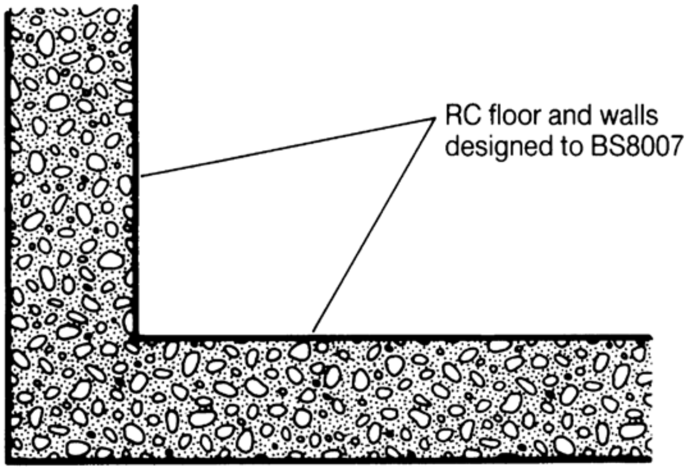


Fig. 8.3 Structurally integral protection

adjacent to the building, and paved areas should be provided around the structure which will allow surface water to be drained away.

BS 8102 recommends that in the design of basements not exceeding 4.0 metres deep, the design head of ground water should be assumed to be three-quarters of the full depth of the basement below ground level (but not less than 1 metre). This may sometimes seem to be a very conservative approach, but it is important to remember that if a basement is excavated in

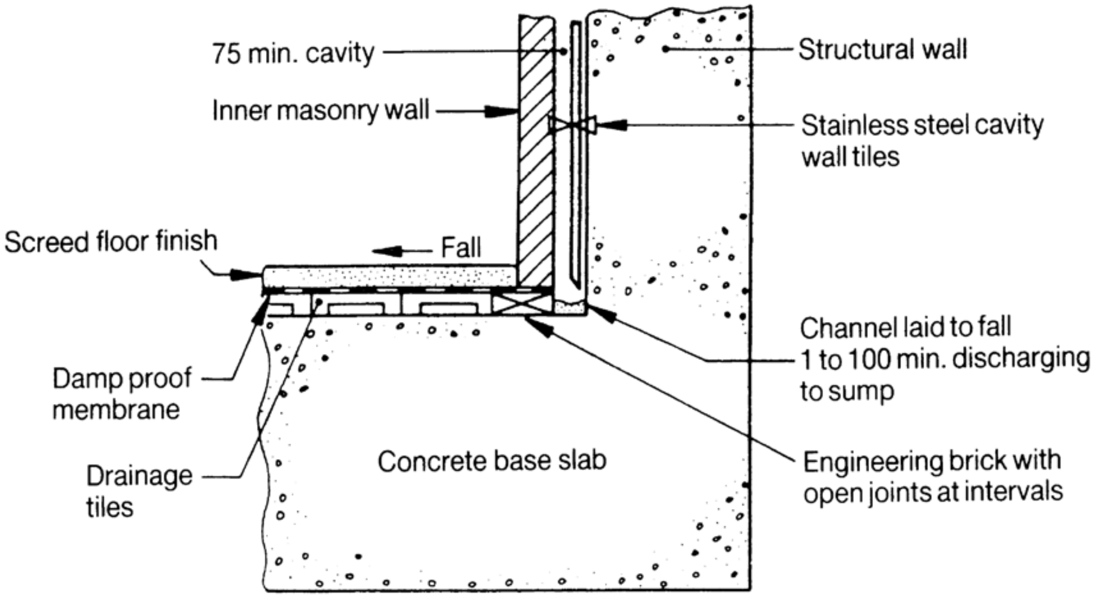


Fig. 8.4 Drained cavity construction

clay soil and backfill is placed around the completed structure, then a sump has been created which will tend to attract any surface water in the vicinity.

A comprehensive soils investigation is necessary for all but very small jobs and, in the case of basement construction, it is important to obtain detailed information concerning any ground water table together with an indication of the likely variation of that table both seasonally and over the anticipated life of the building. The results of the investigation can be compared with the recommendations in BS 8102, to arrive at a design decision. The investigation should also provide information about the quality of the soils and ground water in terms of pH value and any dissolved chemicals. Sulphate content is particularly important, together with any other chemicals present from previous uses of the ground. The design decisions concerning the use of an external or internal membrane may well be influenced by the results obtained from the soil investigation (see Figure 8.1 and 8.2).

8.4 Barrier materials

The essential properties for a barrier material are that it should be inherently vapour excluding and that it should be of a form that can be conveniently applied to the main structure. This includes the ability to negotiate corners and changes of level and to remain stable in a vertical application to a wall. The structure onto which the barrier material is placed should not contain uncontrolled cracks which might rupture the material. Hence, design of concrete to BS 8007 is to be preferred. Details for any movement joints should be prepared to preserve the exclusion of vapour, and also at any change in backing material (e.g. brick to concrete). It should be noted that a vapour excluding barrier will also prevent water penetration, assuming that the barrier material is not forced away from the structure by water pressure. The main materials in use are described below. To specify each material in detail it is necessary to consult BS 8102 and other appropriate British Standards and manufacturers' literature.

Protection of the material is generally required after it has been placed. This applies both on the outside of a structure before backfilling and on the inside by providing a loading material to prevent vapour pressure blowing the material away from the structure.

8.4.1 *Mastic asphalt*

Mastic asphalt is a material which has been used widely for many years. It is applied hot and worked into position by hand. It is normally applied in two or three coats of 10 mm per coat. In vertical work, it may require support at intervals due to the weight of the material. The joints in each layer are staggered to avoid possible paths for leakage. Where asphalt is applied to the exterior of the structure, it requires protection before backfill is placed.

8.4.2 *Bitumen sheet*

This material consists of a sheeting material coated with bitumen. It is supplied in rolls of various weights and widths, and applied by being stuck to the structure with a priming coat of liquid bitumen. The surfaces onto which the material is applied should be smooth and free from rough edges. At least two layers are required, with the lines of the joints being staggered in position.

8.4.3 *Cement-based renders*

These are mixed on site from sand, cement and a waterproofing admixture or a polymer resin. Water is added and the mixture applied in two coats with staggered joints. These renders are not necessarily entirely vapour excluding. It is important to ensure that the backing materials are in a satisfactory condition to receive the render and that the backing is stable and uncracked. Rendering over a change in materials is not likely to be satisfactory as cracks will form in the render over the lines of change. No protection to the render is normally necessary.

8.4.4 *Polyurethane resins*

Various products are available which are supplied as a liquid or semi-liquid and are applied by roller, trowel or other means specified by the manufacturer. The resin cures after a period of one to two days forming a jointless vapour excluding sheet.

8.5 Structural problems

8.5.1 *Construction methods*

During construction, it is almost always necessary to support the ground outside basement walls and this has an effect on the construction sequence and the positioning of joints. If ground water is present at a relatively high level, then sheet piling, diaphragm walling, or a system of well-points may be required. The design must take account of any restrictions that the construction method imposes.

8.5.2 *Layout*

The layout of the basement structure will be influenced by the method of construction and, in particular, by the means used to support the ground at the sides of the excavation. If temporary sheet piling is used, it is more economic if the junction of the floor and the wall has no heel projecting beyond the outside face of the wall. However, this may conflict with the need for an overlap of the barrier material at the wall/floor junction.

8.5.3 *Piled construction*

For vapour excluding structures, construction on piles requires a complete separation between the pile caps with their stabilizing beams and the wall and floor structure (Fig. 8.5). It occasionally happens that tension piles are required to hold down the basement structure against uplift forces due to ground water. This creates a particular problem as the tension reinforcement in the piles must be properly anchored in the main basement structure, and yet any membrane must be continuous. The possible solutions are either to devise a special local joint around the tension bars, or to use cavity construction.

8.5.4 *Diaphragm and piled walls*

The use of diaphragm walls or contiguous piled walls is extremely convenient when an excavation has to be carried out alongside an existing building, but the nature of these systems is such that they cannot be relied upon to be water excluding. The simplest solution to this problem is to use cavity construction (Fig. 8.4). This is a system where it is accepted that there will be some penetration of the main structure by vapour and possibly water. A system of lining walls is provided, positioned to form a cavity which separates the main structure from the inner lining. Similarly, a secondary floor is provided, which allows for an air space between the main structural floor and the secondary floor. The floor is provided with a vapour excluding layer. Arrangements are

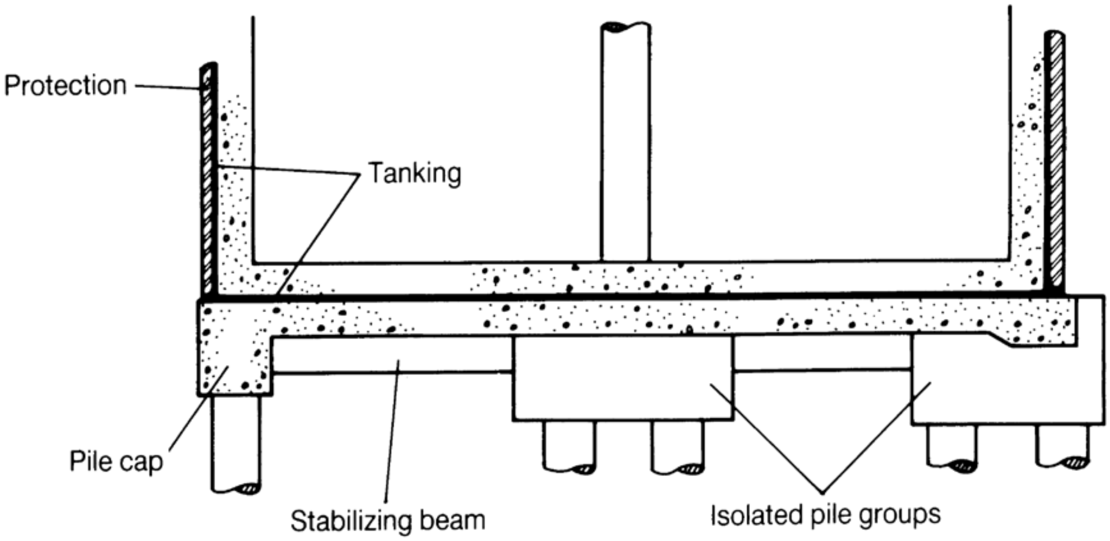


Fig. 8.5 Piled construction

made so that any water which collects in the cavities can be drained away to a sump and pumped out. Vapour may be removed by ventilating the cavity. The degree of protection required will be determined by the particular use of the building (Table 8.1).

8.6 Site considerations

8.6.1 *Workmanship*

Although the quality of workmanship is important in all building operations, the construction of vapour excluding structures demands workmanship of the highest quality. The reasons for this are as follows.

(*a*) Moisture can easily migrate from a defect behind a membrane to emerge on the opposite face in an entirely different position. The source of any leakage of water or transmission of vapour is difficult to locate.

(*b*) When an external membrane is used, it is virtually impossible to gain access to the underside of the floor slab or the outer faces of the external walls without enormous cost and disruption.

(*c*) In general, it is not possible to check that a structure is vapour excluding during the construction phase when there is a great deal of moisture present. Some defects may not be revealed before the heating is activated.

The work involved in the application of membranes to a concrete wall is straightforward, but it requires dedication and detailed care. In adverse weather conditions, work may have to be halted. If the construction sequence requires a section of work to remain in a part-finished state for some time, then the exposed temporary edge may need protection, and the joint between the old and new will require careful treatment by cleaning the previous work before bonding on the new.

8.6.2 *Failure*

The author has inspected a basement which was to be used as a retail trading floor, and the structure was subjected to ground water pressure from a level of 800 mm below the surface. The structure was quite correctly designed to BS 8007, with the addition of an externally applied membrane. In spite of these features, the structure leaked profusely. The workmanship on the application of the membrane was very poor, and the waterstops which had been inserted in the construction joints were ineffective — again due to faulty workmanship.

It is not sufficient for a contractor to hire the next man on the list from the labour exchange and put a brush in his hand. The operatives must be properly trained and preferably have relevant experience. Supervision is also important and needs to be nearly continuous. To execute a design correctly costs money, but the cost of satisfactory repairs will be many times greater.

8.6.3 *Services*

It is frequently required to pass pipes or services through a water and/or vapour excluding wall. It is preferable to cast service pipes ducts etc into the wall rather than leave a hole to be made good later. A puddle flange should be provided around pipes etc at the centre of thickness of the wall. Puddle flanges can be provided on both cast iron and plastic pipes, but with plastic pipes a further problem occurs due to the flexibility of the material. There is a possible lack of adhesion between the surface of the pipe and concrete (leading to leakage). A convenient method of improving the adhesion between plastic pipes and concrete is to paint the outside surface of the pipe with epoxy adhesive and scatter dry sand onto the surface. This technique produces a surface similar to glass paper, and reduces the possibility of any leakage (Fig. 8.6).

8.6.4 *Fixings*

When a basement is used for storage, retail activity or other similar purposes, there will always be a requirement to fix signs, shelves, services and other items to the walls. If the vapour excluding barrier is placed on the inside of the structural walls, the fixings will penetrate the barrier and destroy its effectiveness. It may be possible to design local details to overcome this problem but, in general, the original designers or developers of a building will

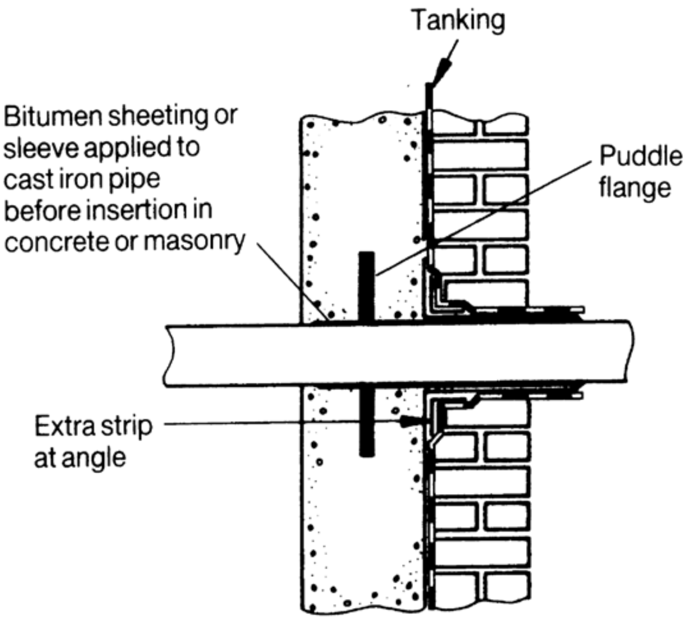


Fig. 8.6 Typical detail of service entry

not have control over the activities of the occupants, and eventually the vapour barrier will be compromised. This problem arises irrespective of the material used for the barrier. There is less problem when services are required in a floor, as they can be embedded in a screed above the vapour barrier. If any drainage goods are specified in the floor, they should be made of cast iron rather than ceramic or plastic as it is otherwise not possible to make a satisfactory vapour seal around pipes and gullies.

Appendix A

Design tables

A1.1 Limit state design

The two limit states that have a predominant effect on the thickness of concrete required and the quantity of reinforcement are:

(1) The limit state of cracking
(2) The ultimate limit state for flexure.

Generally (1) is the over-riding criterion. The first assessment of thickness of concrete section and reinforcement quantities for a preliminary design, in order to proceed to check that the two limit states are satisfied, is not easily possible by direct methods. Tables have therefore been prepared which enable the designer to choose sections that will resist a stated bending moment, in the knowledge that the chosen section will meet the requirements of both the limit states given above. An outline of the derivation of the Tables and the method of use follows.

A1.2 Layout of the design tables

The tables may be used to arrive at the overall thickness and the amount of reinforcement required to satisfy the limit state of cracking and the ultimate limit state. Values are tabulated for two limiting crack widths.

Tables A1.1–13 limiting crack width = 0.1 mm
Tables A2.1–13 limiting crack width = 0.2 mm

The layout of each separate table is similar. All the tables have been prepared for material properties and other conditions which are described in paragraph A1.5. Each is constructed for a particular overall section thickness h, varying between 200 mm and 1000 mm. Values of service bending moment, reinforcement stress and ultimate shear capacity are given for standard bar sizes placed at standard spacings.

A1.3 Tabulated values

For a given reinforcement arrangement, each entry in a table consists of three values aligned vertically. In descending order they are:

Service moment of resistance (kNm/m)

Service tensile stress in the steel (N/mm^2)
Ultimate shear capacity of the section (kN/m)

The values of bending moments are the lesser of:

(1) The ultimate moment of resistance of the section divided by 1.4.
(2) The elastic moment of resistance of the section at the steel stress indicated, which ensures that the crack width is within the limiting value for the particular class of exposure.

Where (1) is critical, the tabulated values are printed in italics and the steel stress will be 285 N/mm^2. The formulae which have been used and the assumptions which have been made are given in paragraph A1.5.

A1.4 Method of using the tables

The tables should be used as follows:

(1) Decide on the minimum thickness of section by considering the ease of construction, the relation of the element to the remainder of the structure, and the necessity to limit deflection (see Section 3.2).
(2) Calculate the applied service moment, i.e. the bending moment on the section due to the design loads (with a partial safety factor $\gamma_f = 1.0$).
(3) Decide on the exposure conditions and refer to the set of tables for the appropriate crack width.
(4) Examine the tables for values of moment of resistance which are at least as great as the applied design moment, and decide on the thickness of the section. The precise size and spacing of reinforcement can be read, together with the design (service) stress in the reinforcement.
(5) There are several possible values of h and reinforcement arrangement for any given value of applied moment, and the designer should be guided in his final choice by adopting a thickness h which shows the value of required design bending moment associated with the design tensile reinforcement stress desired. When deciding on an appropriate value for the thickness h of a section, it is also necessary to consider shear stresses. If no shear reinforcement is to be used, the ultimate shear stress must be limited to that permissible on the concrete alone.
(6) The design tensile stress in the reinforcement should be as high as possible for due economy, but relatively low steel ratios at high stresses are not very forgiving if the site workmanship is less than perfect. The author therefore generally uses stresses of the order of 220 to 250 N/mm^2.

A1.5 Derivation of tables

A1.5.1 *Calculation method*

The values of service bending moment and reinforcement stress have been calculated for limiting design crack widths of 0.1 mm and 0.2 mm. The value of the factor for the stiffening effect of the concrete depends on the particular limiting crack width. The tables in this appendix have been calculated by assuming values for the section details, material properties and also the required limiting crack width. From these values, it is possible to calculate the permissible stress in the tensile reinforcement and then the service moment of resistance. A check calculation of the ultimate moment is also made, and where this is critical the value is printed in the tables in italics, with a stress of 285 N/mm^2.

The third value is the ultimate shear capacity of the section in kN.

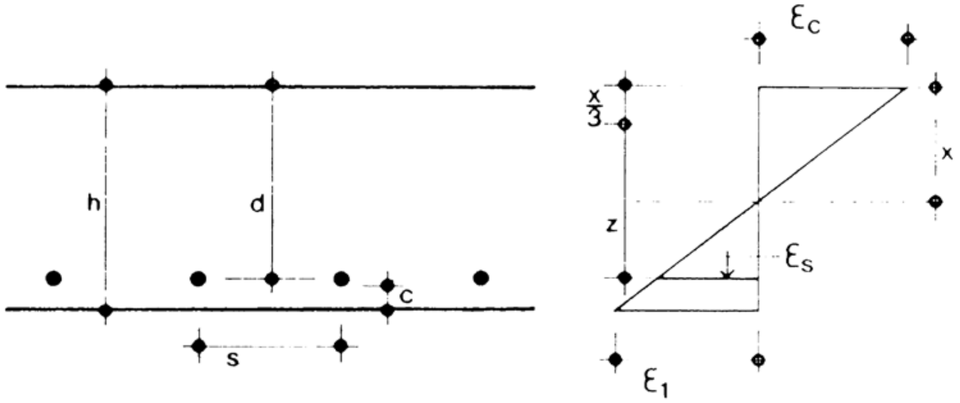


Fig. A1.1 Section with strain diagram

A1.5.2 *Formulae—elastic calculation* (*see Section 3.4*)

The calculations are made by normal elastic theory for a unit length of slab reinforced in one face. Figure A1.1 illustrates a section through the slab. The following formulae are used in sequence to evaluate the properties of the section. The symbols are defined in Chapter 3.

$$\text{Depth of neutral axis} = \frac{x}{d} = \alpha_e\rho\left(\sqrt{1+\frac{2}{\alpha_e\rho}}-1\right) \text{ where } \rho = \frac{A_s}{bd} \tag{A1.1}$$

$$\text{Lever arm } z = d - \tfrac{1}{3}x \tag{A1.2}$$

Note: $\frac{z}{d} \nless 0.75$ and $\ngtr 0.95$

$$\text{Applied elastic moment } M = A_s f_{st} z \times 10^{-6} \tag{A1.3}$$

Value of strain at surface ignoring stiffening effect of concrete

$$\varepsilon_1 = \frac{f_{st}}{E_s} \cdot \frac{h-x}{d-x} \times 10^{-6} \tag{A1.4}$$

The stiffening effect of the concrete between cracks may be assessed using the following empirical formulae. These formulae apply only to the particular crack widths stated as a value of strain is implicit in the formulae.

For a limiting design surface crack width of 0.2 mm:

$$\varepsilon_2 = \frac{b_t(h-x)(a'-x)}{3E_sA_s(d-x)} \tag{A1.5}$$

For a limiting design surface crack width of 0.1 mm:

$$\varepsilon_2 = \frac{1.5b_t(h-x)(a'-x)}{3E_sA_s(d-x)} \tag{A1.5}$$

and, allowing for the uncracked concrete,

$$\varepsilon_m = \varepsilon_1 - \varepsilon_2 \tag{A1.6}$$

For cracking in a slab the factor

$$\frac{a'-x}{h-x}$$

in the formula is equal to unity.

The design surface crack width

$$w = \frac{3a_{cr}\varepsilon_m}{1+2\left(\dfrac{a_{cr}-c_{min}}{h-x}\right)} \tag{A1.7}$$

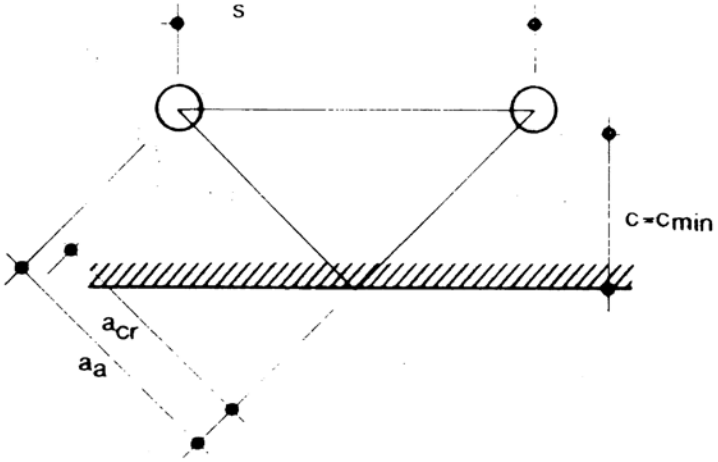


Fig. A1.2 Geometry of concrete cover

Value of strain at surface ignoring stiffening effect of concrete

$$\varepsilon_1 = \frac{f_{st}}{E_s} \cdot \frac{h-x}{d-x} \times 10^{-6} \tag{A1.4}$$

The stiffening effect of the concrete between cracks may be assessed using the following empirical formulae. These formulae apply only to the particular crack widths stated as a value of strain is implicit in the formulae.

For a limiting design surface crack width of 0.2 mm:

$$\varepsilon_2 = \frac{b_t(h-x)(a'-x)}{3E_sA_s(d-x)} \tag{A1.5}$$

For a limiting design surface crack width of 0.1 mm:

$$\varepsilon_2 = \frac{1.5b_t(h-x)(a'-x)}{3E_sA_s(d-x)} \tag{A1.5}$$

and, allowing for the uncracked concrete,

$$\varepsilon_m = \varepsilon_1 - \varepsilon_2 \tag{A1.6}$$

For cracking in a slab the factor

$$\frac{a'-x}{h-x}$$

in the formula is equal to unity.

The design surface crack width

$$w = \frac{3a_{cr}\varepsilon_m}{1+2\left(\dfrac{a_{cr}-c_{min}}{h-x}\right)} \tag{A1.7}$$

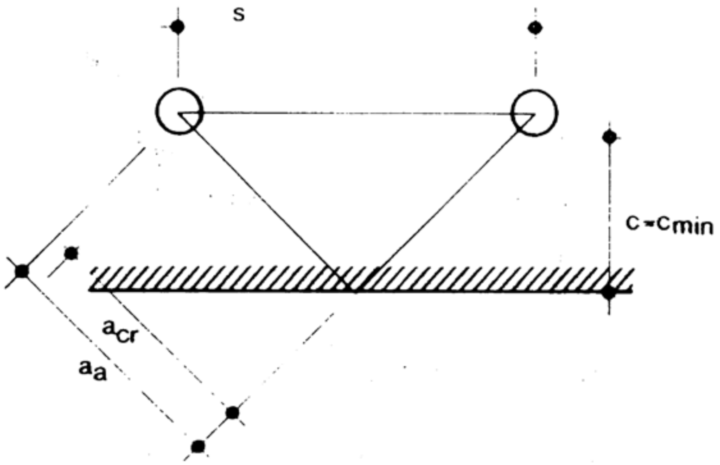


Fig. A1.2 Geometry of concrete cover

From Figure A1.2 it can be seen that

$$a_{cr} = a_a - \tfrac{1}{2}\phi \tag{A1.8}$$

$$a_a^2 = (\tfrac{1}{2}s)^2 + (c + \tfrac{1}{2}\phi)^2 \tag{A1.9}$$

Hence a_{cr} can be calculated and used in formula A1.7.

A1.5.3 *Formulae—limit-state calculation* (*see Section 3.5*)

The ultimate lever arm is calculated as

$$z = \left(1 - \frac{0.97A_s f_y}{f_{cu}bd}\right)d \tag{A1.10}$$

with a minimum value of $0.75d$ and a maximum value of $0.95d$

and the ultimate moment of resistance as

$$M_u = (0.87f_y)A_s z \tag{A1.11}$$

The factored value is $M_u/1.4$ which is compared with the service moment.

A1.5.4 *Formulae—shear calculation*

The permissible ultimate concrete shear stress is taken from BS 8110 Table 3.9

$$v_c = 0.79(f_{cu}/25)^{1/3}(100A_s/bd)^{1/3}(400/d)^{1/4}/\gamma_m$$

where d is limited to 400 mm and the steel ratio is limited to 3.0.

The ultimate permissible shear force on the concrete alone is then:

$$V = v_c bd$$

A1.5.5 *Constants and material properties*

The following values have been used in the computer program from which the tables have been prepared:

Concrete: $f_{cu} = 35$ N/mm^2

Reinforcement: high yield $f_y = 460$ N/mm^2

Modular ratio $\alpha_e = 15$

Cover to tensile reinforcement $c = 52$ mm ($h = 200$ to 300)
56 mm ($h = 350$ to 800)
60 mm ($h = 1000$)

(It is assumed that the cover to the secondary reinforcement in the outer layer will be 40 mm.)

The area of tensile reinforcement is calculated from the specified bar sizes and spacings.

The area of compression reinforcement is assumed to be zero.

A1.5.6 *Restrictions on tabulated values*

1 The bar spacings for a given section are limited to the value of the section thickness with a maximum value of 300 mm (as recommended by BS 8007).

2 If the arrangement of a particular bar size and spacing amounts to a steel ratio of less than 0.175% per face, no values of moment or stress are shown in the tables. The steel ratio is calculated using a maximum value of $h = 500$ as recommended in BS 8007. This steel ratio is a practical minimum. Where values are tabulated, it does not follow that the particular steel ratio is necessarily sufficient to control early thermal movement, and this point must always be checked.

3 If a particular bar size and spacing, associated with a particular section thickness, results in a design concrete compressive stress of more than $0.45f_{cu}$, no values are printed.

4 As reinforcement stresses become larger with smaller ratios of steel, a point is reached where the ultimate moment of resistance becomes critical. This point is at a steel stress of about $460/(1.15 \times 1.4) = 285\ \text{N/mm}^2$. In this case the tabulated values are shown in italic type. Due to differing assumptions in respect of the separate calculations for cracking and strength, an anomaly sometimes occurs when the service stress in the cracking calculation is greater than $285\ \text{N/mm}^2$ although the section has a more than adequate ultimate strength.

5 It should be noted that satisfactory designs may be prepared with values of moment and stress that are less than those shown in the tables. The tabulated values are absolute maxima derived from the formulae stated above. The judgement of the designer should be exercised in determining the service reinforcement stress to be used.

A1.6 Example of use of Tables A1 and A2

Design a cantilever wall 6.5 metres high to support a load due to water pressure. Design crack width = 0.2 mm.

From Table 3.1, for $H = 6.5$ m and a possible thickness $h = 750$ mm.

The values shown in Tables A1 and A2 for the service moment of resistance of the section and the service stress in the reinforcement may be adjusted by applying the ratio to the steel stress of actual moment/tabulated moment.

This is obviously only possible when the tabulated moment being considered is greater than the applied service moment (service conditions are with $\gamma_f = 1.0$).

Consider a unit width of slab of 1 metre:

Service moment $M_s = (1/6) \times 10 \times 6.5^3 = 458$ kN m/m

From Table A2.12 for $h = 800$ mm consider T25 at 125 (3925 mm^2/m).

$$\text{Tabulated values: } M_s = 527 \text{ kNm/m}$$

$$f_s = 206 \text{ N/mm}^2$$

$$\text{For } M_s = 458, \quad f_s = 206 \times \frac{458}{527} = 179 \text{ N/mm}^2$$

which is satisfactory.

$\therefore$ Use a slab with $\underline{h = 800\text{ mm}}$ and reinforced with $\underline{\text{T25 at 125}}$.

Using the values obtained from the design tables, a check on the accuracy of the original assumptions must now be made.

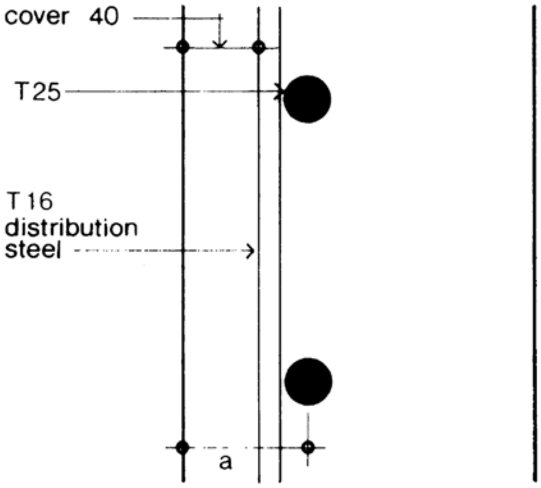


Fig. A1.3 Reinforcement arrangement

$$\text{Actual axial distance} = 40 + 16 + 12.5 = 68.5 \text{ mm}$$

$$d = 800 - 69 = 731 \text{ mm}$$

Check ultimate shear capacity.

From Table A2.12 $\qquad V = 420$ kN

Applied ultimate shear at root of cantilever

$= \frac{1}{2} \times 10 \times 6.5^2 \times 1.4$

$= 296$ kN

Satisfactory.

Tables A1 Limiting moments (kN m), service steel stresses (N/mm²) and ultimate shear capacity (kN/m) for 0.1 mm crack widths

Table A1.1 $h = 200$ Cover to main bars = 52 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	19.0 136 121	17.4 153 112	16.3 171 105	15.5 189 100	15.0 207 96		
16					17.1 138 115		
20							
25							
32							

Table A1.2 $h = 225$ Cover to main bars = 52 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	24.6 148 129	22.5 167 120	21.1 186 113	20.1 206 107	19.4 226 102		
16				23.2 139 129	22.0 150 123		
20							
25							
32							

Table A1.3 $h = 250$ Cover to main bars = 52 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	30.7 159 137	28.1 180 127	26.4 202 119	25.2 223 113	24.4 246 109	23.3 291 101	
16			30.9 138 144	28.8 149 137	27.3 160 131	25.3 184 121	
20						28.2 136 140	
25							
32							

Table A1.4 $h = 275$ Cover to main bars = 52 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	37.3 170 144	34.2 193 134	32.2 216 126	30.8 240 119	29.9 265 114		
16		40.9 135 161	37.4 147 152	34.9 159 144	33.1 171 138	30.8 197 128	
20						34.0 144 148	
25							
32							

Table A1.5 $h = 300$ Cover to main bars = 52 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	44.4	40.8	38.5	36.9	35.8		
	181	206	231	257	284		
	151	140	132	125	119		
16	54.8	48.5	44.3	41.5	39.4	36.8	35.2
	131	143	155	168	181	210	239
	182	169	159	151	144	134	126
20					44.6	40.4	37.8
					135	151	169
					167	155	146
25							
32							

Table A1.6 $h = 350$ Cover to main bars = 56 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	57.8	53.8	51.2	49.5			
	197	226	257	288			
	162	150	141	134			
16	69.6	62.3	57.6	54.3	52.0	49.1	47.4
	138	153	168	183	199	233	268
	196	182	171	162	155	144	136
20				61.3	57.6	52.8	49.9
				136	145	165	185
				188	180	167	157
25							54.7
							134
							181
32							

Table A1.7 $h = 400$ Cover to main bars = 56 Crack width = 0.1

	Bar spacing (mm)						
Bar size (mm)	100	125	150	175	200	250	300
12	75.5	70.5	67.4				
	217	251	286				
	173	161	151				
16	89.7	80.7	74.8	70.9	68.1	64.6	
	150	167	184	202	220	259	
	209	194	183	174	166	154	
20			85.3	79.0	74.5	68.7	65.3
			138	148	159	181	205
			211	201	192	178	168
25						76.2	70.6
						132	146
						206	194
32							

Table A1.8 $h = 450$ Cover to main bars = 56 Crack width = 0.1

	Bar spacing (mm)						
Bar size (mm)	100	125	150	175	200	250	300
12	95.2	89.4					
	237	276					
	183	170					
16	111.8	101.0	94.1	89.4	86.2	82.2	
	162	181	200	221	242	286	
	222	206	194	184	176	163	
20		117.2	106.2	98.7	93.4	86.6	82.7
		138	148	160	172	197	224
		238	224	213	204	189	178
25						95.0	88.4
						142	157
						219	206
32							

Table A1.9 $h = 500$ Cover to main bars = 56 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	117.1 257 197	*150.6* *285* *183*					
16	136.0 173 238	123.5 194 221	115.5 216 208	110.1 239 198	106.4 263 189		
20	161.5 135 276	141.9 147 256	129.1 159 241	120.4 171 229	114.3 185 219	106.5 213 203	102.2 244 191
25					128.3 136 253	115.6 152 235	108.2 169 221
32							

Table A1.10 $h = 600$ Cover to main bars = 56 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*231.2* *285* *226*	*185.0* *285* *210*					
16	190.7 195 273	174.7 222 254	164.6 249 239	158.0 277 227	*204.8* *285* *217*		
20	222.8 150 316	197.4 164 294	181.1 179 276	170.0 195 263	162.3 212 251	152.8 247 233	147.5 284 219
25			209.1 136 320	191.4 144 304	178.9 153 290	163.0 172 270	153.8 194 254
32							168.5 133 298

Table A1.11 *h* = 700 Cover to main bars = 56 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*274.2* *285* *253*	*219.4* *285* *235*					
16	253.7 218 306	234.5 249 284	222.4 282 268	*277.7* *285* *254*	*243.0* *285* *243*		
20	292.1 164 355	261.1 181 329	241.3 200 310	228.0 219 294	218.8 239 282	207.6 281 261	*252.3* *285* *246*
25	354.3 131 411	305.8 140 381	274.2 149 359	252.7 159 341	237.6 170 326	218.6 193 302	207.8 219 285
32						242.8 135 355	223.7 148 334

Table A1.12 *h* = 800 Cover to main bars = 56 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*317.2* *285* *279*	*253.7* *285* *259*					
16	325.3 240 338	302.9 277 313	*374.9* *285* *295*	*321.3* *285* *280*	*281.2* *285* *268*		
20	369.7 178 391	333.1 199 363	309.9 220 342	294.5 243 325	284.0 266 310	*350.5* *285* *288*	*292.1* *285* *271*
25	442.7 140 453	384.9 150 420	347.5 162 396	322.2 174 376	304.5 186 359	282.6 214 334	270.4 244 314
32					347.4 133 422	309.1 147 392	287.0 162 369

Table A1.13 *h* = 1000 Cover to main bars = 60 Crack width = 0.1

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*401.4* *285* *327*	*321.1* *285* *303*					
16	485.2 280 395	*569.7* *285* *367*	*474.7* *285* *345*	*406.9* *285* *328*	*356.0* *285* *314*		
20	536.3 202 458	493.3 230 425	466.2 259 400	448.3 289 380	*555.1* *285* *363*	*444.1* *285* *337*	*370.1* *285* *318*
25	622.9 154 530	553.6 169 492	508.9 185 463	478.9 201 440	458.0 219 421	432.3 256 391	*576.7* *285* *368*
32			594.1 135 545	542.8 143 518	506.5 152 495	460.4 171 460	434.3 192 432

Tables A2 Limiting moments (kN m), service steel stresses (N/mm^2) and ultimate shear capacity (kN/m) for 0.2 mm crack widths

Table A2.1 *h* = 200 Cover to main bars = 52 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	26.0 186 121	22.2 196 112	19.7 206 105	17.8 216 100	16.4 227 96		
16	36.8 155 145	31.1 162 135	27.1 167 127	24.2 173 120	22.0 178 115		
20		40.7 142 155	35.4 146 146	31.4 150 139	28.4 153 133		
25					36.9 134 153		
32							

Table A2.2 *h* = 225 Cover to main bars = 52 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	33.0 199 129	28.1 209 120	24.8 219 113	22.4 230 107	20.7 241 102		
16	47.1 167 155	39.5 173 144	34.2 178 136	30.4 183 129	27.6 188 123		
20	62.6 148 179	52.3 153 167	45.1 156 157	39.8 159 149	35.7 162 142		
25				52.5 140 172	47.0 142 164		
32							

Table A2.3 *h* = 250 Cover to main bars = 52 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	40.4 210 137	34.3 220 127	30.2 231 119	27.4 243 113	25.3 255 109	22.5 280 101	
16	58.0 177 165	48.3 182 153	41.7 186 144	36.9 191 137	33.4 196 131	28.6 208 121	
20	77.7 158 190	64.4 161 177	55.1 164 166	48.4 166 158	43.3 168 151	36.2 174 140	
25			73.6 146 192	64.4 148 182	57.3 149 174	47.3 152 162	
32						64.5 133 189	

Table A2.4 $h = 275$ Cover to main bars = 52 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	48.2	40.9	36.0	32.6	30.2		
	220	231	242	255	267		
	144	134	126	119	114		
16	69.2	57.4	49.4	43.7	39.5	33.8	
	185	190	194	199	204	216	
	174	161	152	144	138	128	
20	93.4	76.8	65.4	57.2	51.1	42.6	
	166	168	170	172	174	180	
	201	186	175	167	159	148	
25		103.6	88.0	76.5	67.8	55.7	
		152	153	153	154	156	
		215	202	192	184	171	
32					93.8	76.5	
					137	137	
					215	200	

Table A2.5 $h = 300$ Cover to main bars = 52 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	56.3	47.8	42.1	38.2	35.4		
	230	241	253	266	280		
	151	140	132	125	119		
16	80.9	66.8	57.4	50.8	45.9	39.4	35.3
	193	197	201	206	211	224	240
	182	169	159	151	144	134	126
20	109.5	89.6	76.0	66.3	59.1	49.3	43.1
	173	175	176	177	179	185	192
	210	195	184	174	167	155	146
25		121.6	102.7	88.9	78.5	64.2	55.0
		158	158	158	158	160	162
		225	212	201	193	179	168
32					109.4	88.6	74.8
					141	141	141
					226	210	197

Table A2.6 $h = 350$ Cover to main bars = 56 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	69.4 236 162	59.6 251 150	53.1 266 141	48.6 282 134			
16	98.1 194 196	81.6 200 182	70.6 205 171	62.8 212 162	57.2 219 155	49.6 235 144	44.9 254 136
20	132.2 173 226	108.4 175 210	92.3 177 198	80.8 179 188	72.3 182 180	60.8 190 167	53.6 199 157
25	180.1 157 262	147.0 158 243	124.0 158 228	107.4 158 217	95.0 158 208	77.9 161 193	67.1 164 181
32			174.1 142 268	150.2 141 255	132.0 140 243	106.6 140 226	90.1 140 213

Table A2.7 $h = 400$ Cover to main bars = 56 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	87.7 252 173	75.5 269 161	67.5 287 151				
16	122.9 205 209	102.3 211 194	88.5 218 183	78.9 225 174	72.0 233 166	62.8 252 154	
20	165.6 183 242	135.4 184 225	115.0 186 211	100.6 188 201	90.0 191 192	75.9 200 178	67.2 211 168
25	227.1 166 280	183.9 166 260	154.3 165 245	133.2 165 232	117.5 165 222	96.3 167 206	83.0 171 194
32		261.1 150 305	218.3 149 287	187.0 147 273	163.6 146 261	131.2 144 242	110.5 144 228

Table A2.8 $h = 450$ Cover to main bars = 56 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	107.4	92.9					
	267	287					
	183	170					
16	149.2	124.3	107.8	96.4	88.2	77.4	
	216	222	229	238	247	269	
	222	206	194	184	176	163	
20	200.6	163.6	138.9	121.6	109.0	92.3	82.0
	191	192	194	197	200	210	222
	257	238	224	213	204	189	178
25	275.6	222.1	185.9	160.1	141.2	115.8	100.0
	174	173	171	171	171	173	178
	297	276	259	246	236	219	206
32		317.2	263.6	224.8	196.0	156.7	131.8
		157	154	152	150	148	148
		324	305	289	277	257	242

Table A2.9 $h = 500$ Cover to main bars = 56 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	128.5	*150.6*					
	282	*285*					
	197	*183*					
16	176.8	147.7	128.5	115.2	105.7		
	225	232	241	250	261		
	238	221	208	198	189		
20	236.9	193.2	164.2	143.9	129.3	109.9	98.2
	198	200	202	205	209	220	234
	276	256	241	229	219	203	191
25	325.6	261.7	218.6	188.2	166.0	136.4	118.2
	180	178	177	176	177	179	185
	318	296	278	264	253	235	221
32	469.0	374.7	310.0	263.7	229.4	183.0	154.0
	165	162	159	156	154	152	152
	373	347	326	310	296	275	259

Table A2.10 $h = 600$ Cover to main bars = 56 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*231.2* *285* *226*	*185.0* *285* *210*					
16	236.4 242 273	198.7 252 254	174.1 263 239	157.1 276 227	145.1 290 217		
20	313.7 211 316	256.5 213 294	218.8 216 276	192.6 221 263	173.8 227 251	149.4 241 233	134.7 260 219
25	429.8 190 366	344.7 189 340	288.0 187 320	248.2 187 304	219.4 188 290	181.5 192 270	158.5 200 254
32	622.1 175 430	493.6 171 399	406.5 167 375	344.7 164 356	299.4 161 341	239.0 159 316	201.8 160 298

Table A2.11 $h = 700$ Cover to main bars = 56 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*274.2* *285* *253*	*219.4* *285* *235*					
16	301.8 259 306	255.4 271 284	*225.4* *285* *268*	*277.7* *285* *254*	*243.0* *285* *243*		
20	396.1 222 355	325.3 226 329	278.8 231 310	246.9 237 294	224.0 244 282	194.4 263 261	*176.9* *285* *246*
25	539.5 199 411	433.1 198 381	362.5 197 359	313.4 197 341	278.0 198 326	231.8 205 302	204.1 215 285
32	781.0 182 482	617.6 178 448	507.8 174 421	430.5 170 400	374.2 168 383	299.8 166 355	254.5 168 334

Table A2.12 *h* = 800 Cover to main bars = 56 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*317.2* *285* *279*	*253.7* *285* *259*					
16	372.8 275 338	318.0 290 313	*374.9* *285* *295*	*321.3* *285* *280*	*281.2* *285* *268*		
20	484.1 233 391	399.7 238 363	344.6 245 342	306.7 253 325	279.8 262 310	*245.2* *285* *288*	*292.1* *285* *271*
25	654.7 207 453	526.8 206 420	442.5 206 396	384.0 207 376	342.1 209 359	287.7 218 334	255.3 231 314
32	945.4 189 532	746.8 184 494	614.2 180 465	521.3 176 442	454.0 174 422	365.7 174 392	312.4 177 369

Table A2.13 *h* = 1000 Cover to main bars = 60 Crack width = 0.2

Bar size (mm)	Bar spacing (mm)						
	100	125	150	175	200	250	300
12	*401.4* *285* *327*	*321.1* *285* *303*					
16	*712.1* *285* *395*	*569.7* *285* *367*	*474.7* *285* *345*	*406.9* *285* *328*	*356.0* *285* *314*		
20	651.2 245 458	548.4 256 425	481.2 267 400	435.1 281 380	*555.1* *285* *363*	*444.1* *285* *337*	*370.1* *285* *318*
25	861.9 212 530	704.5 215 492	600.6 218 463	528.7 222 440	477.0 228 421	410.1 243 391	370.5 262 368
32	1226.1 190 624	978.3 187 579	813.0 185 545	697.3 184 518	613.4 184 495	503.5 187 460	437.5 193 432

Appendix B

Two-way slabs

Rectangular slab panels: 2-way spans

Figure B.1 enables slab panels to be designed when loaded with triangularly distributed loads. An additional surcharge pressure with rectangular distribution can be designed by reference to Table 3.15 in BS 8110 or by considering the wall to be extended by a height equal to (surcharge/density of liquid).

An example of the use of Figure B.1 is given in Example 6.1.

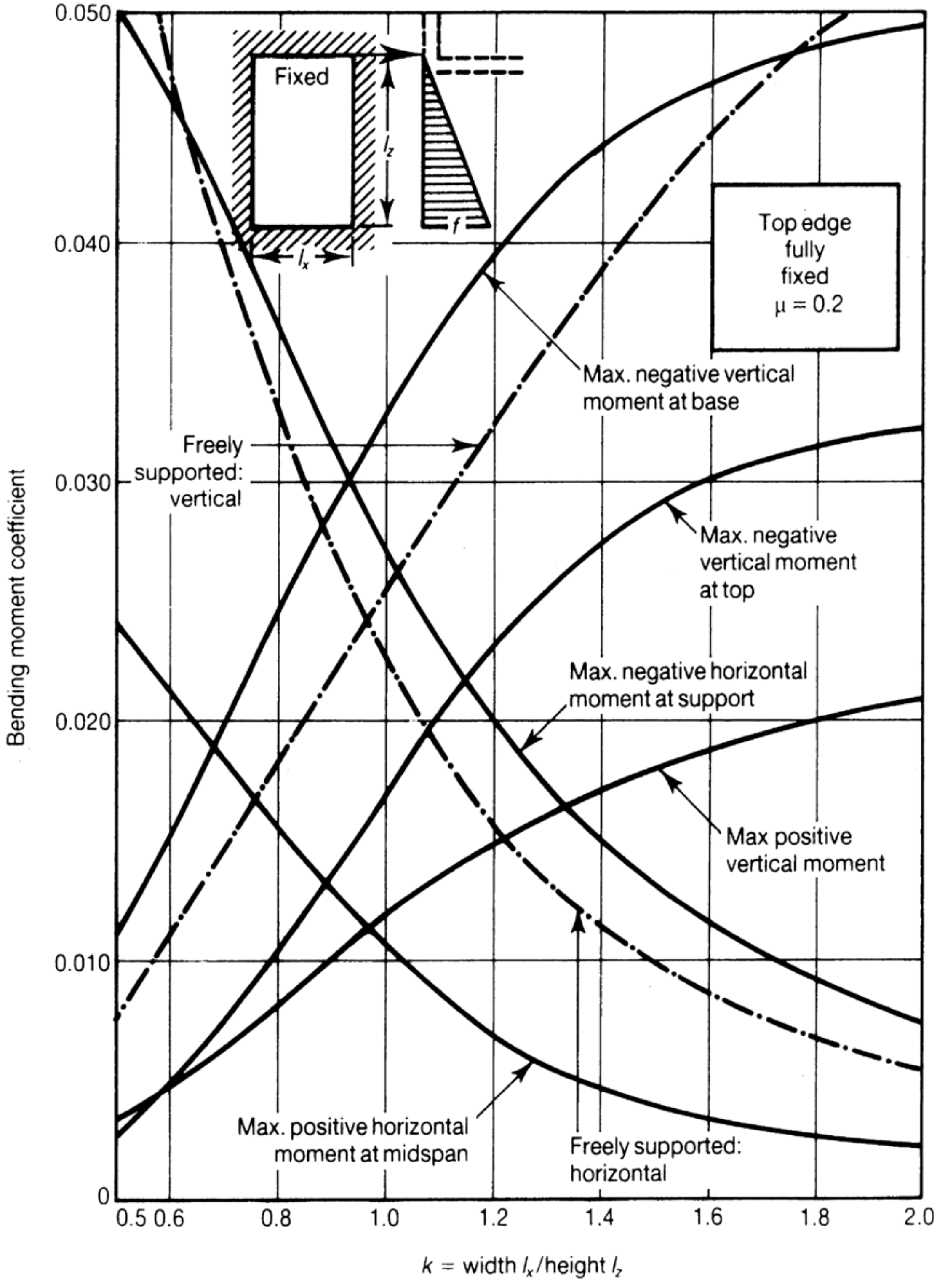
Fig. B.1 Two-way slabs: rectangular panels, triangularly-distributed loads

Panels fixed or continuous along bottom edge and both vertical sides; condition along top edge as indicated.

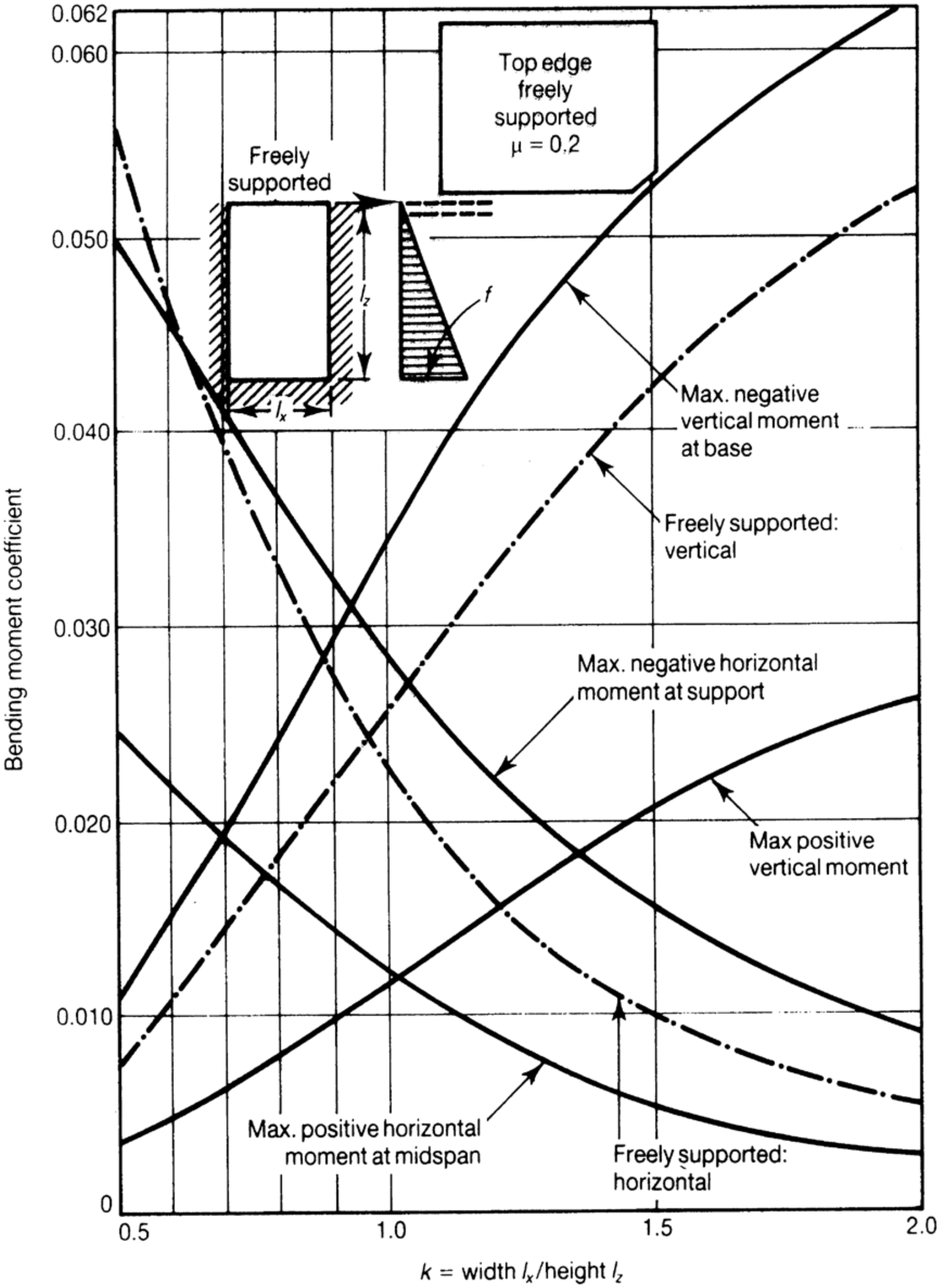
Vertical span: bending moment = (coefficient) $f^2_{l_z}$

Horizontal span: bending moment = (coefficient) $f^2{}_{l_x}$

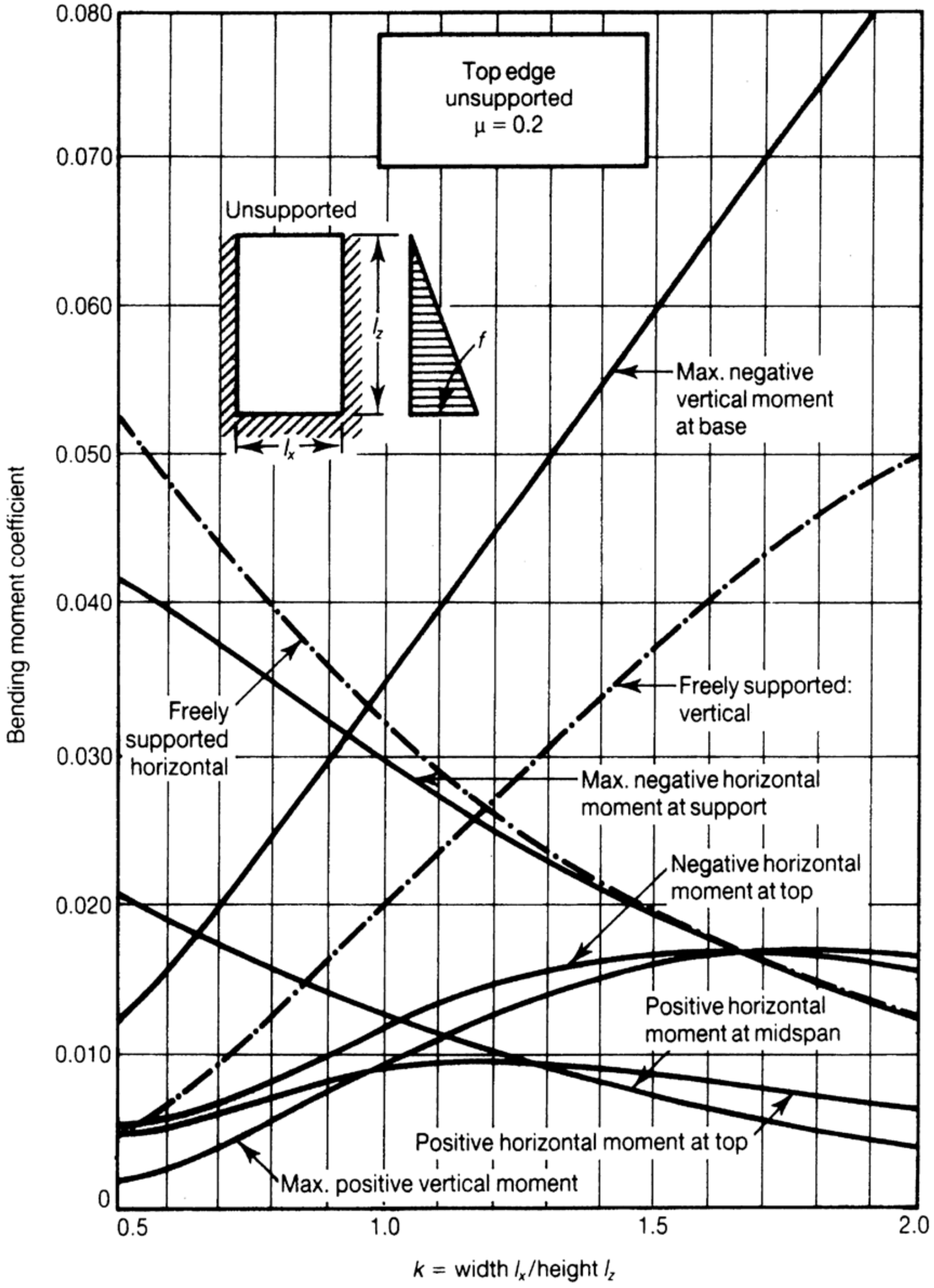
Scale on right-hand side is for values of λ_1, λ_2 and λ_3. Ratio of spans = $k = l_x / l_z$.



(a) Fixed at top



(b) Freely supported at top



(c) Unsupported at top

Appendix C

Bar tables

Bar reinforcement

The tables of bar areas in Table C.1 are included for convenience. The spacings and bar sizes are the preferred values in UK.

Abbreviations: (UK practice)
T = High yield deformed bars
R = Mild steel plain bars

Table C.1 Sectional areas of groups of bars (mm^2)

Bar size (mm)	Number of bars 1	2	3	4	5	6	7	8	9	10
8	50.3	101	151	201	252	302	352	402	453	503
10	78.5	157	236	314	393	471	550	628	707	785
12	113	226	339	452	566	679	792	905	1020	1130
16	201	402	603	804	1010	1210	1410	1610	1810	2010
20	314	628	943	1260	1570	1890	2200	2510	2830	3140
25	491	982	1470	1960	2450	2950	3440	3930	4420	4910
32	804	1610	2410	3220	4020	4830	5630	6430	7240	8040
40	1260	2510	3770	5030	6280	7540	8800	10100	11300	12600

Sectional areas per metre width for various bar spacings (mm^2)

Bar size (mm)	Spacing of bars (millimetres) 50	75	100	125	150	175	200	250	300
8	1010	671	503	402	335	287	252	201	168
10	1570	1050	785	628	523	449	393	314	262
12	2260	1510	1130	905	754	646	566	452	377
16	4020	2680	2010	1610	1340	1150	1010	804	670
20	6280	4190	3140	2510	2090	1800	1570	1260	1050
25	9820	6550	4910	3930	3270	2810	2450	1960	1640
32	16100	10700	8040	6430	5360	4600	4020	3220	2680
40	25100	16800	1·2600	10100	8380	7180	6280	5030	4190

Note: The tables have been calculated to three significant figures according to BSI recommendations.

Bibliography

British Standards

The British Standard publications referred to in this book, or of relevance to the subject, are listed below.

BS 12	Specification for Portland cements
BS 882	Aggregates from natural resources for concrete
BS 1370	Low heat Portland cement
BS 2499	Hot applied joint sealants for concrete pavements
BS 3892	Pulverised fuel ash
BS 4027	Sulphate resisting Portland cement
BS 4246	Low heat Portland blast furnace cement
BS 4254	Two-part polysulphide-based sealants
BS 4449	Carbon steel bars for the reinforcement of concrete
BS 4483	Steel fabric for the reinforcement of concrete
BS 5328	Concrete (Parts 1–4)
BS 5896	High tensile steel wire and strand for the prestressing of concrete
BS 6213	Guide to the selection of constructional sealers
BS 6699	Ground granulated blast furnace slag for use with Portland cement
BS 7295	Fusion bonded epoxy-coated carbon steel bars for the reinforcement of concrete
BS 8000	Workmanship on building sites
Pt. 2	Code of practice for concrete work
Pt. 4	Code of practice for waterproofing
BS 8004	Foundations
BS 8007	Design of concrete structures for retaining aqueous liquids
BS 8102	Code of practice for the protection of structures against water from the ground
BS 8110	Structural use of concrete (Parts 1–3)

Australian Standard

AS 3735–1991	Concrete structures for retaining aqueous liquids

American Standards and Codes of Practice

American Society for Testing and Materials

A82–88	Specification for steel wire, plain, for concrete reinforcement
A185–85	Specification for steel welded wire fabric, plain, for concrete reinforcement
A416–86	Specification for uncoated seven-wire stress-relieved strand for prestressed concrete
A421–80	(1985) Specification for uncoated stress-relieved wire for prestressed concrete
A496–85	Specification for steel wire, deformed, for concrete reinforcement
A497–89	Specification for welded deformed steel wire fabric for concrete reinforcement
C33–90	Specification for concrete aggregates
C39–86	Test method for compressive strength of cylindrical concrete specimens
C150–89	Specification for Portland cement
C494–86	Specification for chemical admixtures for concrete
D994–71	(1982) Specification for preformed expansion joint filler for concrete (Bituminous type)
D1190–74	(1980) Specification for concrete joint filler, hot-poured elastic type

American Concrete Institute

ACI 301–89	Specifications for structural concrete for buildings
ACI 308–81	Standard practice for curing concrete (revised 1986)
ACI 309–87	Guide for consolidation of concrete
ACI 318–89	Building code requirements for reinforced concrete
ACI 318R–89	Commentary on ACI 318–89
ACI 318M–89	Metric version of ACI 318–89
ACI 318RM–89	Metric version of ACI 318R–89
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