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Concrete Buildings Scheme Design Manual

A handbook for the IStructE chartered membership examination

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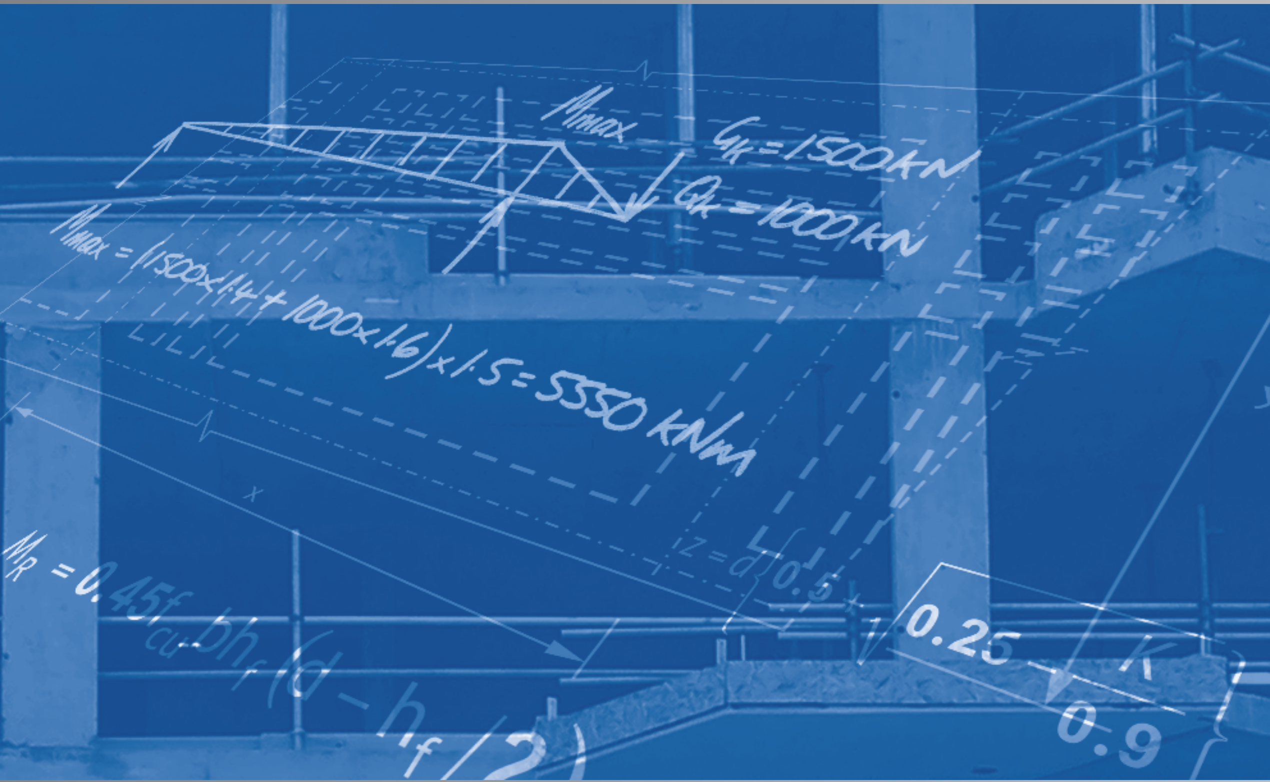

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Concrete buildings scheme design manual

Contents

Symbols used in this handbook

Symbol	Definition
A_b	Area of the pile base
A_c	Area of concrete
A_s	Area of tension reinforcement
A_s	Surface area of the pile
$A_{s,min}$	Minimum area of reinforcement
$A_{s,prov}$	Provided area of reinforcement
$A_{s,req}$	Required area of reinforcement
A_{sc}	Area of vertical reinforcement in a column
A_{sv}	Total cross-sectional area of shear reinforcement
a	Drape of tendon measured at centre of profile between points of inflection
b	Width or effective width of the section or flange in the compression zone
b_e	Breadth of the effective moment transfer strip
b_v	Breadth of section (for a flanged beam width, below the flange)
c	Cover
c	Average undrained shear strength over the length of the pile
c_u	Undisturbed shear strength at the base of the pile
D	Pile diameter
d	Effective depth of the tension reinforcement
E_f	Group efficiency ratio
e	Eccentricity of prestressing force
e	Eccentricity of lateral load and shear centre
F	Total design ultimate load ($1.4\ G_k + 1.6\ Q_k$)
F_c	Ultimate load capacity of wall
f_{cu}	Characteristic strength of concrete
f_s	Estimated design service stress in the tension reinforcement
f_y	Characteristic cube strength of reinforcement
f_{yv}	Characteristic strength of shear reinforcement
G_k	Characteristic dead load
h	Overall slab depth
h	Depth of column section
h_f	Thickness of the flange
I	Second moment of area of shear walls
K	A measure of the relative compressive stress in a member in flexure
K'	Value of K above which compression reinforcement is required
k_s	Horizontal coefficient of earth pressure
L	Length of wall
L_t	Length of wall in tension
l	Effective span
l_{ex}, l_{ey}	Effective height of a column about x and y axis
l_x	Length of shorter side of two-way spanning slab
l_y	Length of longer side of two-way spanning slab
M	Design ultimate moment at the section considered
M_{PT}	Moment due to prestressing forces

M_R	Moment of resistance
M_t	Design moment transferred between slab and column
m, n	Number of piles in orthogonal directions
m_{sx}	Maximum design ultimate moments of unit width and span l_x
m_{sy}	Maximum design ultimate moments of unit width and span l_y
N	Design ultimate axial load on a column or wall
N_c	Meyerhof's bearing capacity factor
$N_q{}^*$	Pile bearing capacity factor
n	Number of piles in group
P	Prestressing force
P_{av}	Average prestressing force in tendon
Q_a	Capacity of a single pile
$Q_{a,b}$	Allowable capacity of the pile base
$Q_{a,s}$	Allowable capacity of pile shaft skin friction
Q_k	Characteristic imposed load
q'_{o}	Effective overburden pressure
$q'_{o,mean}$	Mean overburden pressure
s	Distance between points of inflection
s	Pile spacing
s_v	Spacing of links along the member
t	Thickness of wall
u	Effective length of the outer perimeter of the zone
u_o	Effective length of the perimeter that touches the loaded area
V	Design shear force due to ultimate loads
v	Design shear stress at a cross section
v_c	Design concrete shear stress
V_{eff}	Design effective shear including allowance for moment transfer
v_{sx}, v_{sy}	Shear force per unit width in x and y directions
W	Total design ultimate load
y	Distance of shear from origin O
y_s	Distance to the shear centre from origin O
Z_b, Z_t	Section modulus bottom, top
z	Lever arm
α	Adhesion factor
α_{sx}, α_{sy}	Moment coefficients for simply supported two-way spanning slabs
β	Effective height factor for columns
β_b	Redistribution ratio
β_{sx}, β_{sy}	Moment coefficients for restrained two-way spanning slabs
β_{vx}, β_{vy}	Shear force coefficients for restrained two-way spanning slabs
γ_b	Factor of safety on the base of the pile
γ_f	Factor of safety on pile capacity
γ_s	Factor of safety on the pile shaft
Δc_{dev}	Allowance in design for deviations
δ	Angle of friction between the soil and the pile face
σ_t	Tensile stress in shear wall
ϕ_b	Bar diameter
ϕ_l	Link diameter

1 Introduction

The Institution of Structural Engineer's Chartered Membership Examination is highly regarded both nationally and internationally and requires the candidate to rapidly justify their initial design concepts. This handbook is written specifically for those who are preparing to sit the examination, but it should also prove useful to all those involved in preparing outline and scheme stage designs on a day-to-day basis.

1.1 How to use this handbook

This handbook is laid out to reflect the examination questions; to date these have followed the same format. Section 1 of the examination is the design appraisal and this is covered in Section 2, which provides essential information required to assist in answering this part of the examination.

Section 2 of the examination is divided into three parts; firstly (section 2c) there is the requirement to prepare sufficient calculations to establish the size and form of all the principal structural elements. Guidance is given in Section 3 of this handbook.

Section 2d requires the candidate to prepare plans, sections and elevations, including critical details, for estimating purposes; examples are given in Section 4. Finally section 2e requires detailed method statements and often an outline programme to be prepared and this is covered in Section 5.

Readers should appreciate that this publication is written for the structural engineer practising in the UK. Whilst many of the principles can be applied to structures around the world, it must be clearly understood that environmental and geotechnical factors encountered may be significantly different. It is also written with the design of building structures in mind rather than bridges or offshore structures. The requirements for question 8, structural dynamics, are not specifically covered.

Text on a pale blue background provides information to help candidates prepare for the examination and is not intended for use in the examination itself.

1.2 Assumptions

It is assumed that the readers and users of this publication are practising engineers and are already familiar with the principles of design to BS 8110–Part 1: 1997[1] (including amendments 1, 2 and 3).

The worked examples given are intended to act as an aide memoir, rather than as a tool for learning to design in concrete. They provide sufficient justification to demonstrate that the elements are suitable for the proposed situation. Other publications and design aids should be referred to for examples of fully designed reinforced concrete elements. The examples given may be more detailed than the examiners would expect to see in a script, but are included to aid learning and as preparation for the examination. The essential calculations that might be provided in the examination are highlighted by blue text. Candidates will need to use their judgement in applying the worked examples to their solutions in the examination, especially as the principal elements will often be more complex than the examples given in this handbook.

It is strongly recommended that a copy of *Economic concrete frame elements*[2] is obtained to assist in the initial sizing of elements as required to answer section 1 of each question. However, for completeness, some guidance on initial sizing has been provided in this handbook.

1.3 The examination

The examination is intended to be a test of the candidate's ability to develop detailed solutions for challenging structural problems. Candidates will need to demonstrate their understanding of structural engineering and be able to produce two alternative solutions that are robust, stable and buildable. They will also have to demonstrate their knowledge and experience through sketches, diagrams, calculations and descriptions of their solutions.

The examination is highly regarded throughout the world and provides a challenging test for the candidate. However, those who have a good all-round experience in structural engineering and who have thoroughly prepared should be able to pass the examination at the first attempt.

At the time of writing (2006) the candidate is expected to answer one question out of eight. The type of material expected to be used in the solution or the type of structure in the eight questions follows a regular pattern:

1. Steel building
2. Steel building
3. Bridge
4. Concrete building
5. Concrete building
6. General structure
7. Offshore structure
8. Structural dynamics

Knowing that a solution to a particular question is expected to use a certain material may lead the candidate to consider solutions in that material only. However, a solution in another material may be equally applicable and indeed there have been occasions in the past where an alternative material has been more appropriate. The candidate should therefore have an open mind; there is no single solution to any of the questions.

1.4 Timing

The examination lasts for seven hours, plus half an hour for lunch. There are 100 marks available and the candidates should plan their time so that all sections are completed. There is no substitute for experience, and sitting a mock examination will enable the candidate to appreciate what can be achieved in the seven hours available. If the time available for each section is allocated in proportion to the marks available, the following timetable can be followed:

09:30 section 1a (40 marks)
12:20 section 1b (10 marks)
13:00 Lunch
13:30 section 2c (20 marks)
14:55 section 2d (20 marks)
16:20 section 2e (10 marks)
17:00 Finish

The largest proportion of the marks is for section 1a and the candidate may want to produce an individual timetable for this section.

1.5 Further information

There is plenty of useful information about the examination that is freely available from the Institution of Structural Engineer's website[3]. Candidates are advised to download this information and study it.

2 Development of solutions (section 1)

2.1 Viable structural solutions

Candidates are required to prepare a design appraisal of **two** distinct and viable solutions for the brief. It is important that two solutions are presented; offering only one solution is likely to result in a failure. Both solutions should be prepared to the same level of detail. Candidates should demonstrate to the examiner the ability to conceive and present distinct options for the proposed structures.

They should indicate the functional framing, load transfer and stability of the schemes. The question asks for the functional framing before the load transfer diagram because this is the necessary and logical sequence. The load path must follow through the framing. Similarly, the means of stabilising the frame can be addressed only after the load paths have been identified.

In developing solutions the candidate should consider the following issues; more detail is given throughout this section.

- Functional framing
- Load transfer
- Stability
- Safety
- Economy
- Buildability
- Robustness
- Durability
- Site constraints
- Speed of construction
- Aesthetics
- Acoustics
- Footfall-induced vibration
- Thermal mass
- Sustainability
- Building movements
- Fire resistance

2.2 Functional framing

The structure should be idealised to show how it functions; this should be presented as a sketch that shows the type of connections, the stiffness of the elements and/or structure and the nature of the foundations and any retaining structures. Examples of appropriate functional framing diagrams are shown in Figure 2.1.

2.3 Load transfer

The candidate should show how the actions on a structure are transferred from the point of application to the supporting ground; again this is most clearly conveyed through a sketch. Examples are given in Figure 2.2. The load transfer diagram illustrates the way the designer expects the structure to behave.

Generally there are three types of load to consider:

- Vertically acting or gravity loads such as self-weight and imposed floor loading.
- Lateral loads such as wind and the notional horizontal load given in Cl. 3.1.4.2.
- Soil loads, which can have either vertical or horizontal components.

The candidate should be able to quickly determine the critical combination of these loads, based on the load factors given in table 2.1 of BS 8110.

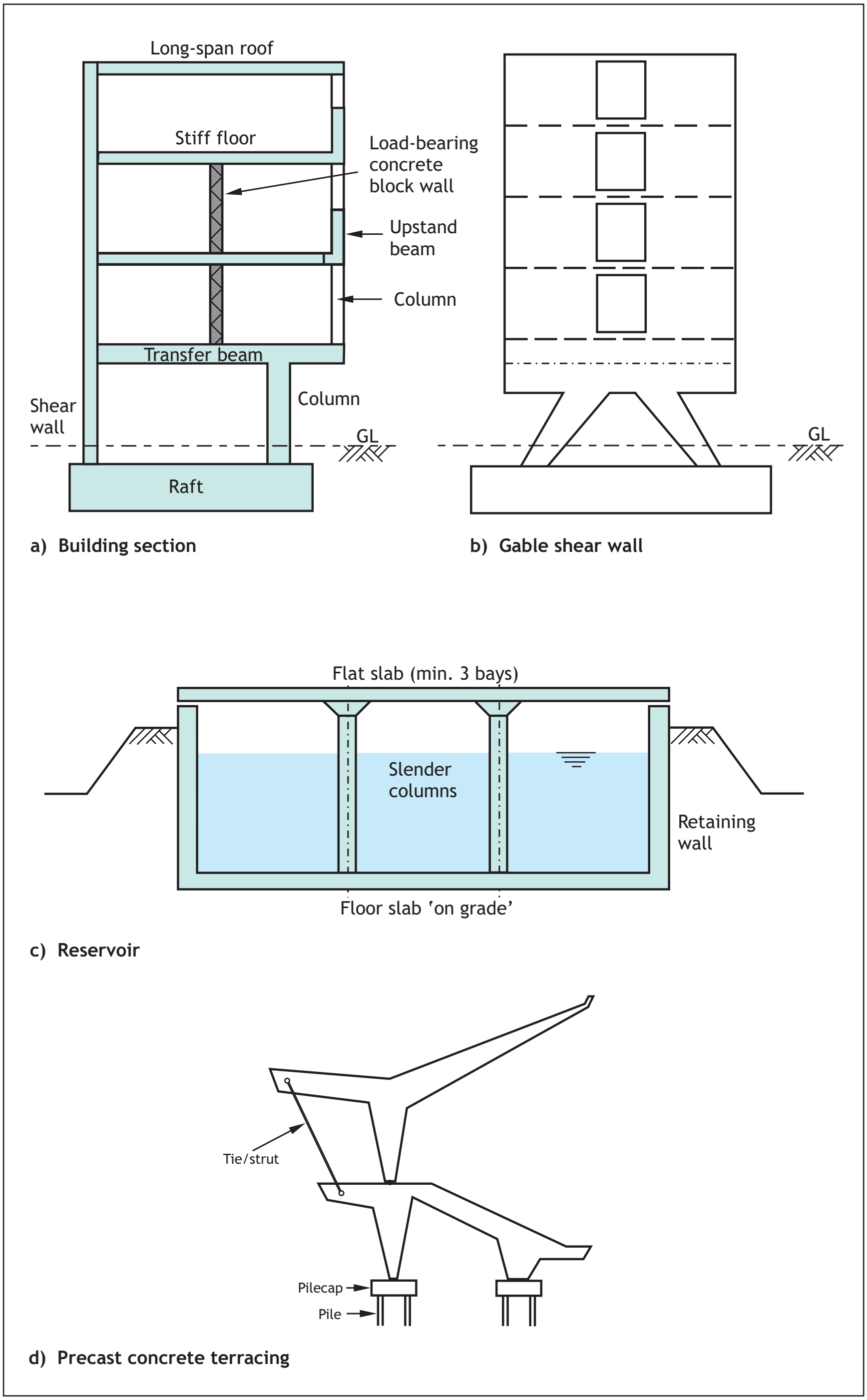


Figure 2.1
Example functional framing diagrams
Diagrams: Bob Wilson

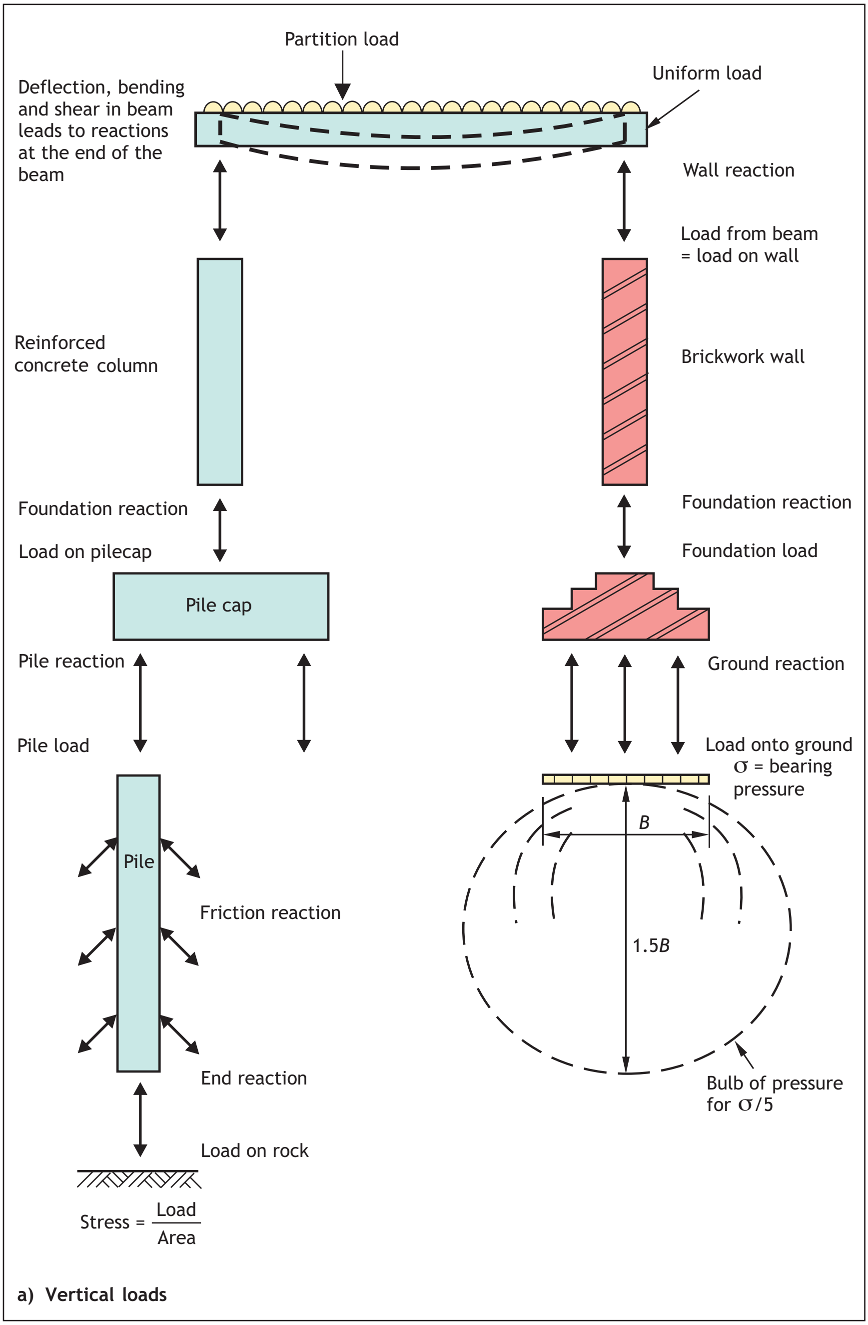


Figure 2.2 (continued overleaf)
Example load transfer diagrams
Diagrams: Bob Wilson

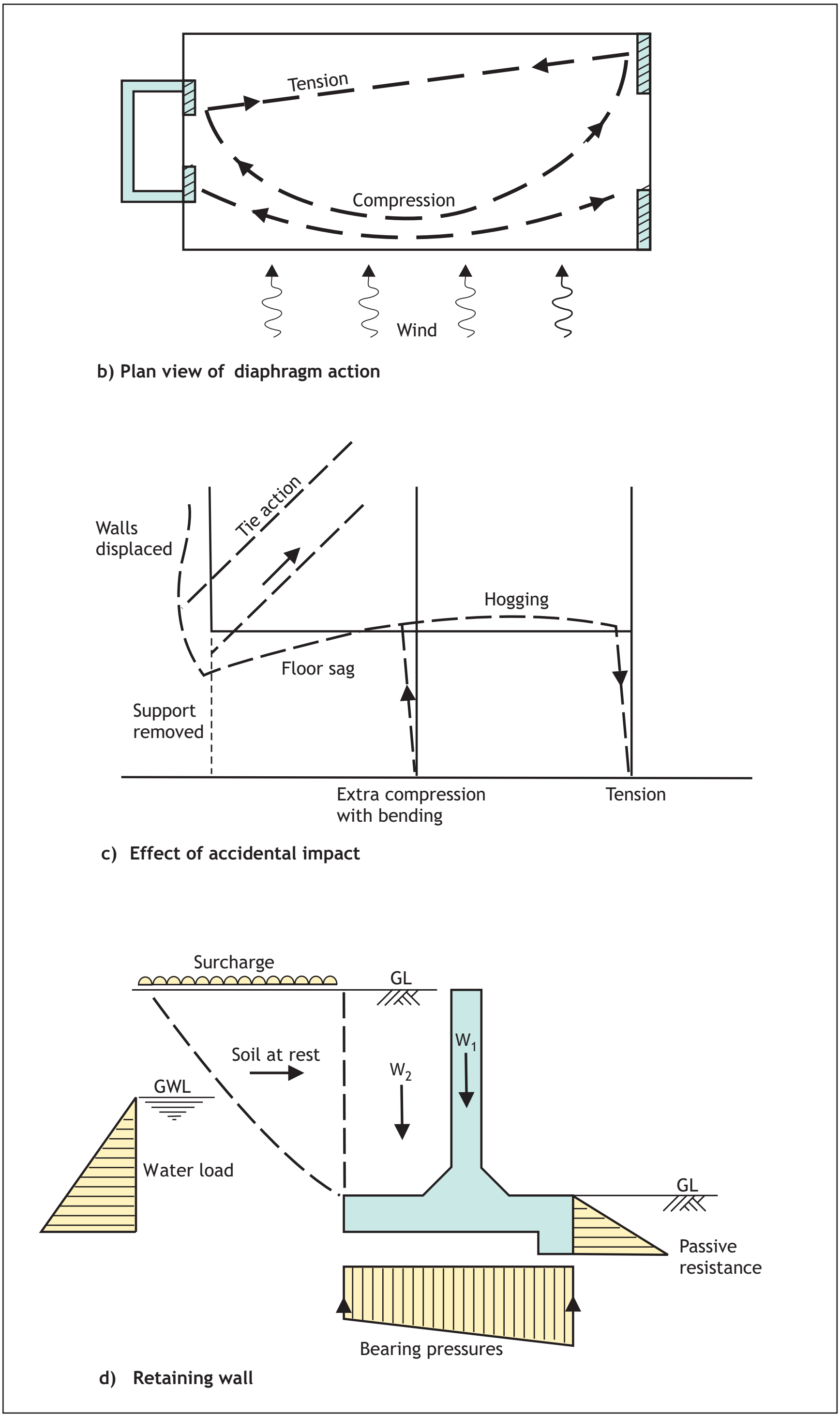


Figure 2.2 continued
Example load transfer diagrams
Diagrams: Bob Wilson

2.4 Stability

It is essential that the building and its various component elements are stable and the candidate is likely to fail if the stability of the structure is not adequately demonstrated. There are two stability criteria to consider; lateral stability and uplift due to water pressure.

2.4.1 Lateral stability

The following are examples of loads that may impose lateral forces on the structure:

- Wind loads.
- Earthquakes.
- Lateral loads due to geometric imperfections.
- Horizontal component of soil loads. (Out-of-balance soil loads should be considered on sloping sites.)
- Accidental loads.

The structure should be designed to resist these loads in two orthogonal directions. For a multi-storey building this can be achieved by using:

- Shear walls (i.e. a braced building) or;
- Moment-resisting frames (i.e. sway frame).

Shear walls should be arranged in plan so that their shear centre coincides with the resultant of the overturning forces (see Figure 2.3). If this is not possible, twisting moments will also occur and the additional forces that result should be added to the other forces in each shear wall. Further guidance on calculating the shear centre and twisting moments is given in Section 3.18.

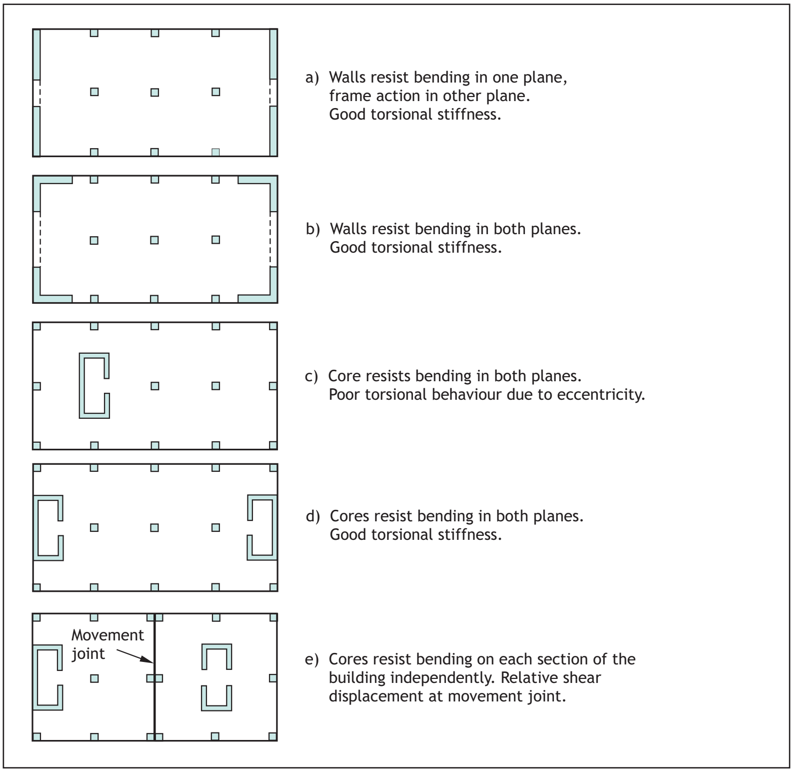


Figure 2.3
Typical shear wall layouts
Diagram: Ciria Report 102[4]

The use of shear walls assumes that the floor will act as a horizontal diaphragm to transfer loads to the shear walls. For in-situ concrete buildings this can normally be assumed and no other checks are necessary. However, further consideration of how the lateral loads are transferred to the shear walls may be required as shown in the following examples:

- Where precast concrete floor units are used, especially where there is no structural topping, e.g. car park deck or under a raised floor.
- Where the loads are transferred across a relatively narrow strip, e.g. walkway linking two parts of the building.
- Where the length of floor plate connected directly to the shear walls is short. This can occur if there are large services risers adjacent to the shear walls.

2.4.2 Uplift

Where the structure requires excavation in ground that has a high water table, the water pressure may cause uplift (buoyancy) of the structure. The candidate should ensure there is suitable resistance to the uplift forces in the final condition, and also determine what measures are required during construction to avoid uplift.

2.5 Concrete frame options

Reinforced concrete is a versatile material that can be formed in many different shapes and forms. Figure 2.4 gives sizing information for the more common solutions for floors. Table 2.1 gives a summary of the common forms of concrete construction that are used, and presents their advantages and disadvantages.

2.6 Foundations and retaining structures

2.6.1 Ground-bearing slabs

Ground-bearing slabs are a popular and economic way to support the loads at the lowest level in the building. However, in the following situations, suspended slabs or ground improvement techniques may be required to reduce settlement:

- Slabs supporting high loads, e.g. industrial floors.
- In areas where the ground conditions include fill.
- In areas with soft clay (void formers may be required where heave can occur).
- On sloping sites, especially where the slab would be supported on varying ground conditions.
- Where trees could can cause settlement or heave.

2.6.2 Foundation solutions

Selecting the correct foundation solution is an important part of the examination; Table 2.2 gives appropriate foundations for a variety of soil conditions. These are generalisations for typical buildings and are not intended to be a substitute for experience. Key considerations at concept stage are:

- Groundwater – a high groundwater level will require special measures during construction and may cause buoyancy of the finished structure.
- Shallow foundations should be just that – shallow. Excavating deeper than 3 m is usually not economic.

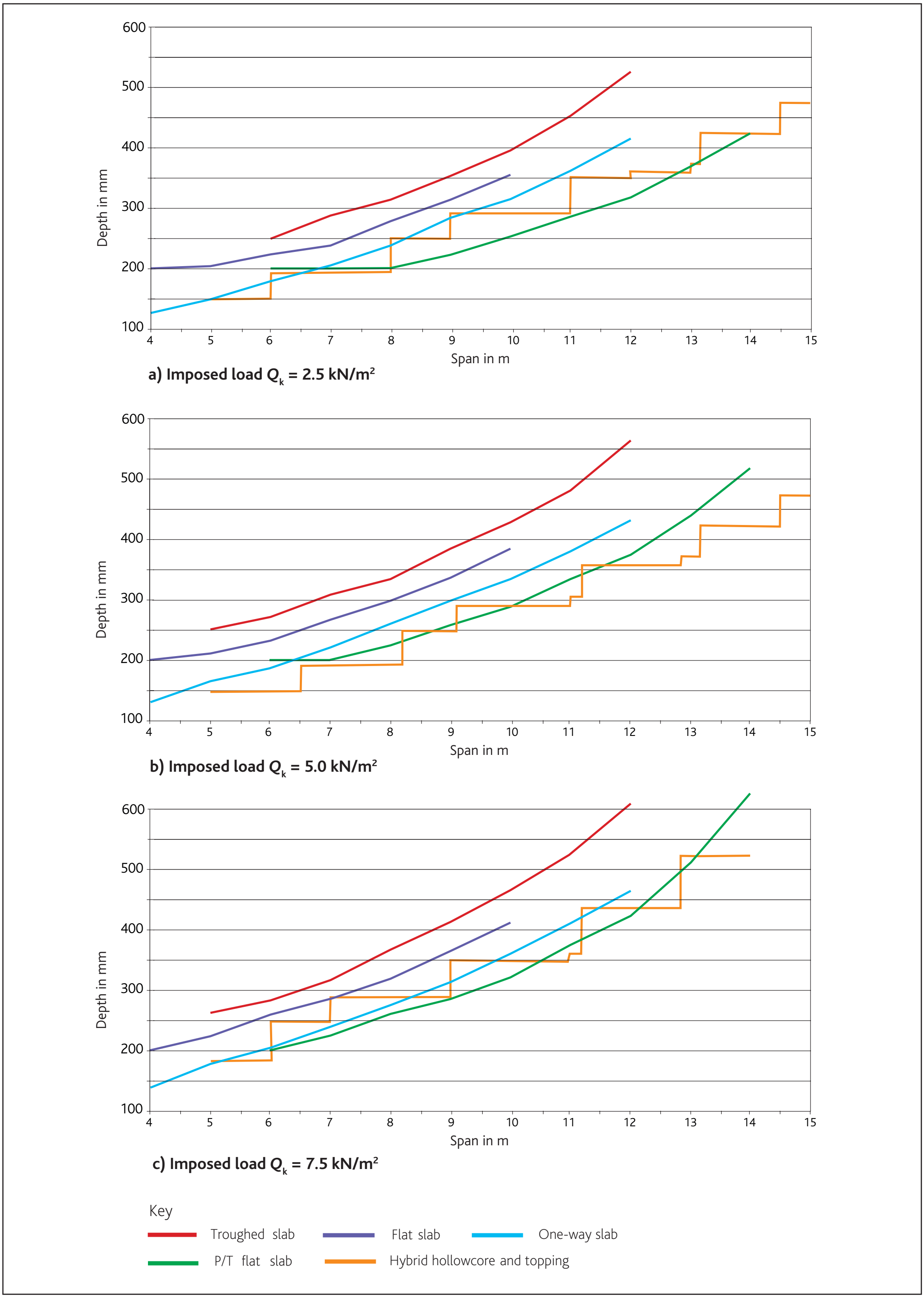


Figure 2.4
Span-to-depth charts for imposed loads of 2.5, 5.0 and 7.5 kN/m²

Table 2.1
Concrete solutions for floors

Concrete option	Span range: Reinforced or prestressed conc. Min	Span range: Reinforced or prestressed conc. Max	Span range: Post-tensioned concrete Min	Span range: Post-tensioned concrete Max	Speed of construction	Economy	Ease of service distribution	Minimises storey height	Flexibility for partitions	Suitability for holes	Punching shear capacity	Deflection control	Minimises self-weight	Soffit can be exposed	Suitable for open plan space	Diaphragm action of floor	Inherent robustness of frame	Off-site construction
Flat slabs																		
Solid flat slab (Continuity improves economy)	4	10	7	13	✓✓	✓✓	✓✓	✓✓	✓✓	O	O	O	O	✓✓	✓✓	✓✓	✓✓	✗
Solid flat slab with drops	4	12	7	14	✓	✓	✓	✓	✓✓	O	✓	✓	✓	O	✓✓	✓✓	✓✓	✗
Solid flat slab with column heads (Forming column head disrupts cycle times and interferes with holes adjacent to columns)	4	10	7	13	✓	✓	✓	✓✓	✓✓	O	✓	O	O	✓✓	✓✓	✓✓	✓✓	✗
Waffle slab	9	12	9	14	O	O	✓✓	✓	O	✓	✓	✓✓	✓✓	✓✓	✓✓	✓✓	✓✓	✗
Bi-axial voided slab (Can be used with in-situ or with precast soffit slab, which would act as permanent formwork)	4	14	7	16	✓✓	✓	✓✓	✓✓	✓✓	O	O	O	✓✓	✓✓	✓✓	✓✓	✓✓	O
One-way slabs																		
Solid one-way slab with beams	4	12	6	14	✓	O	O	O	O	✓✓	✗	✓✓	✓✓	O	✓✓	✓✓	✓✓	✗
Solid slab with band beams	7	12	7	13	✓✓	✓	✓	✓	✓	✓	✓	✓	✓	O	✓✓	✓✓	✓✓	✗
Ribbed slab with beams	7	11	8	12	O	O	✓	✓	O	✓	✗	✓✓	✓✓	✓	✓✓	✓✓	✓✓	✗
Ribbed slab with integral band beams	6	11	8	12	O	✓	✓✓	✓	O	✓	✓	✓	✓	✓✓	✓✓	✓✓	✓✓	✗
Tunnel form (One-way slab on walls) [a]	4	10	✗	✗	✓✓	✓✓	✓✓	✓✓	✗	✓✓	✗	✓✓	✓✓	✓✓	✗	✓✓	✓✓	✗
Composite lattice girder soffit slab [b]	4	8	✗	✗	✓✓	✓	O	O	✓✓	✓	✗	✓	O	✓✓	✓✓	✓✓	✓	✓
Precast hollowcore slab	4	14	✗	✗	✓✓	✓	O	O	✓✓	✓	✗	✓✓	✓✓	O	✓✓	O	O	✓✓
Composite precast slab	4	15	✗	✗	✓✓	✓	O	O	✓✓	✓	✗	✓	✓✓	O	✓✓	✓	O	✓
Precast double 'T' units	6	19	✗	✗	✓✓	✓	O	O	O	✓	✗	✓✓	✓✓	✓	✓✓	O	O	✓✓
Precast crosswall and solid prestressed slab	4	7.5	✗	✗	✓✓	✓	✓✓	✓✓	✗	✓	✗	✓✓	✓	✓✓	✗	✓✓	✓✓	✓
Two-way slabs																		
Solid two-way slab with beams	5	12	✗	✗	O	O	O	O	✓	✓✓	✗	✓✓	✓	O	✓✓	✓✓	✓✓	✗
Two-way waffle slab with beams	9	14	✗	✗	O	O	O	O	O	✓	✗	✓✓	✓✓	O	✓✓	✓✓	✓✓	✗
Waffle slab with integral beams	6	11	✗	✗	O	O	✓✓	✓	O	✓	✓	✓✓	✓	✓✓	✓✓	✓✓	✓✓	✗
Hybrid concrete construction (combination of in-situ and precast concrete)																		
Precast twin wall and lattic girder soffit slab with in-situ infill and topping	4	7.5	✗	✗	✓✓	✓	✓✓	✓✓	✓✓	✓	✗	✓✓	✓	✓✓	✗	✓✓	✓✓	✓
Precast columns and edge beams with in-situ floor slab	4	10	6	12	✓✓	✓✓	✓✓	✓✓	✓✓	✓	O	✓	O	✓	✓✓	✓✓	O	✓
Precast columns and floor units with in-situ beams [b]	4	14	✗	✗	✓✓	✓	✓✓	✓✓	✓✓	✓	✗	✓	✓✓	O	✓✓	✓	✓	✓✓
In-situ columns and beams with precast floor units [b]	4	14	✗	✗	✓✓	✓	✓✓	✓✓	✓✓	✓	✗	✓	✓✓	O	O	✓	✓	✓✓
In-situ columns and floor topping with precast beams and floor units	4	7.5	✗	✗	✓✓	✓	O	O	✓	✓	✗	✓✓	✓✓	O	O	✓	✓	✓

Key

✓✓ Excellent
✓ Good
O Can be used, but may require further consideration
✗ Not applicable or not appropriate

a Requires 100 'tunnels' for maximum economy. Special curing methods required to obtain early age concrete strengths
b Tempory props required

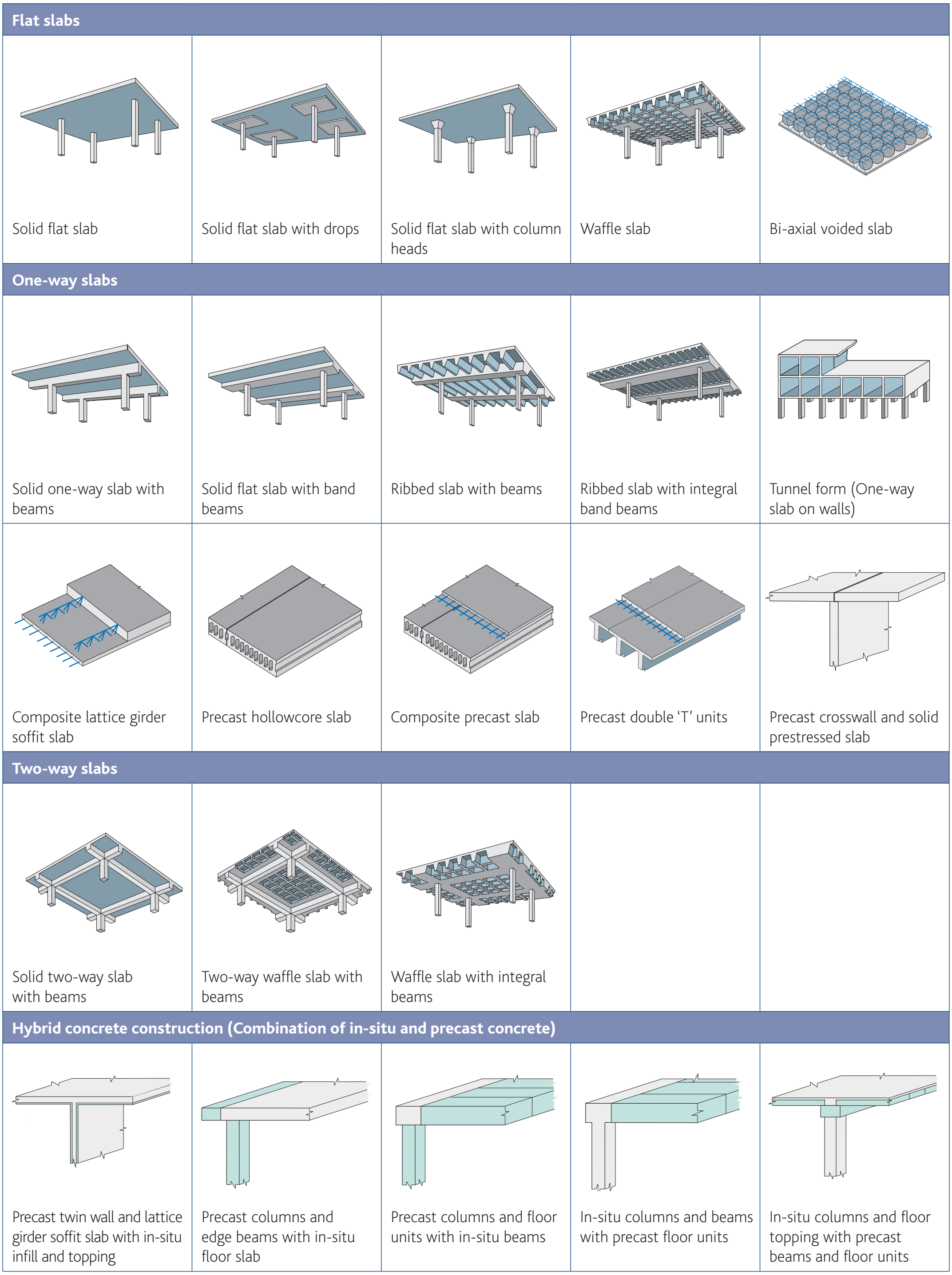


Figure 2.5
Schematic diagrams for the concrete options in Table 2.1

Table 2.2
Foundation solutions

Soil conditions	Suitable foundations	Comments
Rock, hard sound chalk, sand or gravel to great depth	Shallow foundations: strips, pads, rafts	Avoid the base of the foundation being below groundwater level Minimum depth to underside of foundation to avoid frost heave: 450 mm Deep foundation may be required where there are uplift conditions
Uniform firm and stiff clays to great depth, without significant trees in the vicinity	Shallow foundations: strips, pads, rafts	Minimum depth to protect against shrinkage/heave – 900 mm Trench fill can be economic
Uniform firm and stiff clays to great depth, where vegetation could impact on the shrinkage/ expansion of the clay	Options: 1. Piles 2. Deep trench fill (strips) 3. Rafts 4. Piers	Refer to Table 2.3 for strip foundation depths in proximity of trees Use suspended floors with void formers
Firm clay to shallow depth over soft clay to great depth	For lightweight structures, strips, pads or rafts may be appropriate For heavy structures deep foundations will be required	For shallow foundations ensure the load is distributed over a large enough area for the soft clay to support it
Loose sand to great depth	Options: 1. Raft 2. Ground improvement with shallow foundations 3. Piles	Vibration and groundwater changes can induce settlement after construction Driven piles will increase the density of the sand
Soft clay	Options: 1. Piles 2. Wide strip foundation 3. Rafts 4. Ground improvement with shallow foundations	Strip foundations may need reinforcement Service entries into building should be flexible Rafts may not be suitable for highly shrinkable soils
Peat	Options: 1. Piles 2. Ground improvement	Suitable piles: bored in-situ with casing, driven in-situ, driven precast Allow for drag on piles caused by peat consolidation Soils may be acidic
Fill	Options: 1. Piles 2. Wide strip foundation 3. Rafts 4. Ground improvement with shallow foundations 5. Piers	Specially selected and well compacted fill will have greater load bearing capacity Service entries into building should be flexible Consider effects of contaminants in the fill
Clay, increasing in strength as depth increases (from soft to stiff clay)	Piles preferred, but a raft may be suitable for a basement	Settlement is likely to govern the pile design
Soft clay over rock at depth	Use deep foundations	Negative skin friction may add to the loads on piles
Dense sand or stiff clay over layer of soft clay, over stiff clay to great depth	Deep foundations generally required except for light loads. Ground improvement technique could be used with shallow foundations.	
Mining and subsidence areas	Slip-plane raft	Piles not suitable
Sloping site	Foundations to suit soil conditions but the effects of the slope should be considered	Consider overall stability as well as local stability Groundwater will increase instability of site
Site with high groundwater level	All foundation types may be appropriate Dewatering may be used, but consider affects on surrounding structures	In sand and gravel keep foundations above groundwater level Consider uplift forces Stability of excavations should be considered Bored piles require casing or support fluid Continuous flight auger piles suitable Ground conditions may be aggressive

Table 2.3
How to determine foundation depth (m) adjacent to trees in shrinkable soils

Species	Maximum mature height (m)	Exclusion zone 1 (m)	Exclusion zone 2 (m)
High water demand trees			
Elm, Willow	24	24.0	30.0
Eucalyptus	18	18.0	22.5
Hawthorn	10	10.0	12.5
Oak, Cypress	20	20.0	25.0
Poplar	28	28.0	35.0
Moderate water demand trees			
Acacia, Alder, Monkey puzzle, Spruce	18	9.0	13.5
Apple, Bay laurel, Plum	10	5.0	7.5
Ash	23	11.5	17.3
Beech, Cedar, Douglas fir, Larch, Pine	20	10.0	15.0
Blackthorn	8	4.0	6.0
Cherry, Pear, Yew	12	6.0	9.0
Chestnut	24	12.0	18.0
Lime, Sycamore	22	11.0	16.5
Mountain ash	11	5.5	8.3
Plane	26	13.0	19.5
Wellingtonia	30	15.0	22.5
Low water demand trees			
Birch	14	2.8	7.0
Elder	10	2.0	5.0
Fig, Hazel	8	1.6	4.0
Holly, Laburnum	12	2.4	6.0
Hornbeam	17	3.4	8.5
Magnolia, Mulberry	9	1.8	4.5

Foundation depths			
Modified Plasticity Index	Volume change potential	Outside exclusion zone 1	Outside exclusion zone 2
40% and greater	High	1.50	1.00
20% to less than 40%	Medium	1.25	0.90
10% to less than 20%	Low	1.00	0.75

Notes

1 Determine whether a particular species of tree is outside exclusion zone 1 or 2.

2 Determine the foundation depth from the lower part of the table for the particular soil conditions and the appropriate exclusion zone.

3 Where the tree(s) are inside exclusion zone 1 refer to NHBC guidelines [5] on which this table is based.

2.6.3 Piling options

There is a wide choice of piles: the type of pile chosen should be appropriate for the ground and site conditions. Table 2.4 gives some guidance on benefits of a variety of pile types.

2.6.4 Retaining walls

Options for retaining walls are given in Table 2.5.

Table 2.4
Comparison of pile types

Pile type	Advantages	Disadvantages
Driven precast concrete pile	• Quality of pile can be inspected before it is placed in the ground • Construction not affected by groundwater • Can be driven in long lengths • Most appropriate in soft and unobstructed soils • No removed soil to dispose of	• Can be damaged during driving • Pile can be displaced if it hits obstruction • Actual length of pile is known only when proved on site • Relatively large rig required • Noise and vibration, but piling rigs are constantly being improved • Driving force may determine pile properties • Displacement of soil may damage surrounding structures
Driven cast in-situ (A tube is driven into ground and filled with in-situ concrete)	• Length can be readily varied to suit actual ground conditions encountered • Can be driven in very long lengths • Driven with a closed end and therefore groundwater is excluded from hole	• Can be damaged during driving • Relatively large rig required • Noise and vibration, but piling rigs are constantly being improved • Driving force may determine pile properties • Displacement of soil may damage surrounding structures • Concrete cannot be inspected after casting • Large diameters cannot be used
Bored piles	• Can be driven in long lengths • Soil removed can be inspected • Can be installed in large diameters • End enlargements are possible in clay • Can be installed within a limited headroom • Small rigs can be used • Relatively quiet • Low vibration	• Risk of ‘necking’ in ‘squeezing’ ground conditions • Concrete is not placed under ideal conditions and cannot be inspected • Casing may be required in soils lacking cohesion • Removed soil requires disposal • May require underwater concreting • Piling rigs may be large
Augered (e.g. continuous flight auger (CFA))	• Soil removed can be inspected • The ground is continuously supported by the auger • Relatively quiet • Low vibration • Suitable for most soil types (excluding boulders) • Can be installed with a limited headroom • The continuous helical displacement technique (CHD) reduces volume of removed soil and increases soil strength adjacent to pile shaft.	• Maximum 1200 mm pile diameter • Concrete cannot be inspected after pouring • Maximum pile length around 30 m • Limited length of reinforcement cage • Removed soil requires disposal • Efficiency is dependant on regular supply of concrete • Auger may be impeded by relatively stiff soils

Table 2.5
Options for retaining walls

Idealised site conditions	Idealised soil types		
	Dry sand and gravel	**Saturated sand and gravel**	**Clay and silt**
Working space available to allow ground to be battered back during wall construction	Gravity or cantilever retaining wall Precast concrete crib wall	Dewatering during construction of gravity or cantilever retaining wall	Gravity or cantilever retaining wall
Limited working space	King post wall as temporary support Contiguous piled wall Diaphragm wall	Secant bored pile wall Diaphragm wall	King post wall as temporary support Contiguous piled wall Diaphragm wall
Limited working space and special controls on ground movements	Contiguous piled wall Diaphragm wall	Secant bored pile wall Diaphragm wall	Contiguous piled wall Diaphragm wall

2.6.5 Ground improvement

There are two major reasons why the use of ground improvement techniques may be considered. Firstly, where the ground has poor load-carrying capacity, ground improvement is an alternative to deep foundations. Secondly, it may be used to treat contaminated sites prior to redevelopment. On some sites it could be used to overcome both problems. Cementitious products can be used for both situations as discussed in the sections below. There are other treatments available that may be more suitable in certain situations; details of these techniques can be found in publications from the BRE[6], and CIRIA[7] or from the Environment Agency.

Soil mixing

Stabilising land with the use of lime, cement or other binders is a cost-effective method of converting areas of weak soil into a suitable load-bearing material.

Lime can be added to cohesive soils and will cause the following improvements:

- Increased strength.
- Reduced susceptibility to swelling and shrinkage.
- Improved durability to weather and traffic.
- Good handling and compaction characteristics.
- A reduction in plasticity.
- Increased suitability for stockpiling and subsequent reuse.

After treatment with lime, plastic soils break down into fine particles. This makes them suitable for the addition of cement, fly ash (pfa) and ground granulated blastfurnace slag (ggbs) that will impart significant strength.

After the binders have been added and the correct moisture content is achieved, the treated soil is compacted to promote further strength gain and long-term durability. The specification of the treatment should be carried out by specialists.

Soil stabilisation is essentially a mixing process that can be carried out in a number of ways. Normally it is an in-situ process where the binders are mixed into the ground in a layer by powerful rotovators and then compacted with a roller. The layer is nominally 300 mm deep, but any number of layers can be used.

For smaller sites tractor-mounted rotovators are available but these still have a very powerful mixing action. Binders can be applied by spreading them on the ground before mixing or they can be applied during the rotovation process. The latter method eliminates any potential dust problems.

For less cohesive materials it is possible to mix the soil and binders at a central mixing plant. This will involve hauling the soil to the plant for treatment and returning it to the point of deposition. This is usually slower than the in-situ method.

Where the total potential sulfate content is below 0.25% SO_4 there is minimal risk of expansion of the soil due to the reaction between calcium (from lime or cement), alumina (from clay) and sulfate. Where the total potential sulfate content is significant then soil mixing may still be used, but the binders need careful selection to avoid heave due to sulfates.

Grouting

Cementitious grout can also be used to improve load-bearing properties of the soil. There are several different techniques available and success requires specialist knowledge. One common use is to stabilise backfilled old mine shafts. If the backfill to the shaft has not been properly compacted then it can be injected with cement grout to fill major voids and prevent collapse.

2.6.6 Remediation using soil stabilisation/solidification

Soil stabilisation/solidification is a ground remediation technique that involves the controlled addition and mixing of hydraulic binders with contaminated soil to generate a granular or monolithic material in which contaminants are rendered immobile and virtually non-leachable. Although they are not totally removed or destroyed, stabilisation/solidification removes pathways between contaminants and potential receptors. The addition of cement and/or lime has two benefits:

Stabilisation – the production of more chemically stable constituents.
Solidification – the imparting of physical/dimensional stability to contain contaminants in a solid product and reduced access by external agents, such as air or rainfall.

The two processes work together and the chemical and physical changes can be optimised through careful selection of binder materials and minor additives to achieve the desired remedial objectives. At the same time as achieving the remedial objectives the engineering properties of the soil are improved, thereby facilitating the development of the site.

Table 2.6
Contaminants that can be treated with soil stabilisation/solidification [8]

Contaminant	Relative effectiveness of treatment	Comments
Metals (arsenic, cadmium, lead, copper, etc.)	●	Some may require the use of additives to improve the efficacy of treatment
Cyanides and thiocyanates	○	Chemical treatment to modify oxidation state can enhance success
Sulfur and sulfates	●	Treatment enhanced by adding ggbs or pfa. Care required with respect to expansive mineral formation
Salts – chlorides (and other halides), nitrates	○	Success depends on concentration and type of salt
Ammonium salts	○	Limited data available to assess general efficacy
BTEX (benzene, toluene, ethyl benzene, xylene) and other VOC and semi VOCs (volatile organic compounds)	●	Success depends on the type, nature and concentration of particular compounds
PAHs (polyaromatic hydrocarbons)	●	Treatment is enhanced with the use of sorbents
Phenols	○	Treatment is enhanced with the use of sorbents
TPH (total petroleum hydrocarbons) including petrol range and diesel range aliphatics (PRO and DRO)	○	Care required in design of binder to optimise success
Asbestos	●	
PCBs (polychlorinated bi-phenyls), pesticides, dioxins, furans	●	Success with pesticides depends on the particular compounds present
Acids and alkalis	●	
Radionuclides	●	Complete encapsulation or embedment is often used with radioactive wastes

Key
● Treatment is effective with demonstrable applicability
○ Treatment is effective and success is demonstrable. However, efficacy is subject to characteristics of contaminant and appropriate design of S/S treatment system. Pre-treatment may be required before main S/S process is applied

This joint approach can be used for a wide range of different contaminants and can be tailored to meet the requirements of the site-specific risks (see Table 2.6). There is no need to remove the contaminants and this minimises the need to transport spoil to a tip, saving costs and reducing the environmental impact. Certain contaminants cannot be treated in this way e.g. gases.

The soil can be treated using one of the techniques given in Table 2.7.

Table 2.7
Stabilisation/solidification treatments using cementitious binders

In-situ treatment	
Shallow treatment < 0.5 m	The binder is spread over the surface of the ground to be treated at a predetermined dose rate and then mixed in using rotovating equipment. The blended material is then compacted and the reaction between the binder and the moisture in the soil is allowed to take place.
Intermediate treatment 0.5 – 5 m	The binder is mixed into the soil using plant modified to suit the specific site conditions and application.
Deep treatment > 5 m	The binder is introduced into the contaminated soil as a dry powder or slurry using vertical hollow stem augers. The binder is then mixed into the soil as the augers are advanced and/or withdrawn. Often the process uses a grid of augers that overlap to ensure greater efficacy of mixing and treatment.
Ex-situ treatment	
Rotovator or other driven mobile plant	The excavated soil is transported to a final deposition area, where it is spread in layers along with the binder and mixed using rotovating equipment. The blended material is then compacted and the reaction between the binder and the moisture in the soil is allowed to take place.
In-drum mixing	The excavated soil is placed in a drum, into which the binder is added and mixed. The reaction between the binder and the moisture in soil is allowed to take place in the drum, after which the blended material is placed.
Pugmill / batchmixer	The excavated soil is mixed with the binder in a purpose-built plant (mobile or fixed) prior to transportation to a deposition area. The blended material is then compacted and the reaction between the binder and the moisture in the soil is allowed to take place.

Other remediation treatments

As well as soil stabilisation/solidification, other methods of treating contaminated ground using cementitious products are available. Grouting can be used to treat contaminated soils by reducing their permeability. A containment barrier could also be formed to block the groundwater flow using grouting or soil mixing techniques. Alternatively, other approaches such as the construction of a secant pile wall or diaphragm wall to act as a barrier may also be considered.

2.7 Design appraisal

Candidates are asked to select the most appropriate option from the two they have presented. The reasons for these choices should be clearly set out and the following information may be used to help distinguish between the two options and prepare the design appraisal.

2.7.1 Safety

The structural engineer should consider how their design impacts on safety during the following stages in the structure's life:

- Construction
- Use
- Maintenance
- Refurbishment/alteration
- Demolition

In broad terms the designer should use the following procedure to minimise health and safety risks:

- **Identify** the hazards
- **Eliminate** each hazard or reduce its impact
- Where this is not possible **reduce** the risk
- **Provide information** to the contractor

Inevitably decisions will have to be made that strike a balance between protecting against a range of hazards and other design objectives such as:

- Client's functional requirements
- Cost (initial and whole-life)
- Programme
- Aesthetics
- Functionality
- Durability
- Environmental needs

Decisions made at the early stages of the project have the greatest impact, and so health and safety should be given as much consideration as other factors when choosing between options. Further information that may useful when considering health and safety is given in Section 5.1.2.

2.7.2 Economy

For an economically designed structure there will generally be very little difference in the overall building cost between a steel and concrete framed building. This has been confirmed by recent studies of building costs [9, 10, 11, 12].

When making a case for one option or another it is important to consider the whole picture. As well as comparing the cost of the frame, the choice of frame may have an impact on the following areas:

- **Foundations** – a heavier frame will increase foundation size and therefore cost. However, the increase in total frame cost is unlikely to be greater than 0.5% (or 0.04% of total building cost) and will often be less.
- **Cladding** – the cost of cladding is related to the area of the façade. Shorter buildings have a smaller cladding area, which in turn costs less. A structural solution with a lower floor-to-floor height will reduce the overall building cost. As cladding can amount to 25% of the building cost, minimising the floor thickness could save over 1% of the total building cost.
- **Partitions** – sealing and fire stopping at partition heads is simplest with flat soffits. Significant savings of up to 10% of the partitions package can be made compared with the equivalent dry lining package abutting a profiled soffit with downstands. This can represent up to 4% of the frame cost, and a significant reduction in the programme length.
- **Air tightness** – Part L of the UK Building Regulations requires pre-completion pressure testing. Failing these tests means undertaking the time-consuming process of inspecting joints and interfaces and then resealing them where necessary. Concrete edge details are often simpler to seal and therefore have less risk of failure.
- **Services** – the soffit of a concrete flat slab provides a zone for services distribution free of any downstand beams. This reduces coordination effort for the design team and therefore the risk of errors. Services installation is simplest below a flat soffit and this permits maximum off-site fabrication of services. These advantages are reflected in lower costs for services beneath a flat soffit.

The relative economics of one type of structure or another may also depend on the prevailing plant hire, labour and material costs. For instance a flat slab is often chosen in locations where material costs are low in relation to labour costs. This reduces the labour required at the expense of additional materials.

2.7.3 Buildability

Often buildability is affected by how the structure is conceived and detailed by the designer rather than being influenced by the material chosen. The buildability of a concrete frame may be improved by the following:

- **Using flat soffits** – simplifies the formwork, falsework, setting out and services and ceiling installation.
- **Repetition of design elements** – facilitates the re-use of formwork, enables routines to be established more quickly, reduces the learning curve and training requirements, improves safety.
- **Simplification** – reduces complexity of formwork and number of sub-contractors. A simple flat soffit makes it easier to install services.
- **Standardisation** – enables factory production, high quality finishes, speeds up construction, enables damaged parts to be quickly replaced and elements can be selected/approved before final fixing.
- **Rationalisation of reinforcement** – speeds construction, facilitates prefabrication, reduces detailing time.
- Design elements so they can be **precast** or **prefabricated**.
- **Using in-situ concrete** – accommodates late design changes, provides robustness for precast systems, and is appropriate for foundations.

2.7.4 Robustness

An in-situ concrete frame is generally very robust because of its monolithic nature. Usually the tying requirements of the UK Building Regulations to avoid disproportional collapse are met with normal detailing of concrete. Particular attention should be given to transfer structures, where certain elements may become 'key elements' and consideration should be given to ensuring there are alternative load paths.

Precast concrete frames require special consideration with regards to robustness. The precast elements should be tied together to avoid disproportionate collapse; further advice is given in Appendix A.

2.7.5 Durability

A well-detailed concrete frame is expected to have a long life and require very little maintenance. It should easily be able to achieve a 60-year design life and, with careful attention to the specification of the cover and concrete properties, should be able to achieve 120 years even in aggressive environments. BS 8500 is the state-of-the-art standard on durability and gives advice for various environments (see Section 3.3 for more information).

2.7.6 Site constraints

The location of the site may impose constraints that favour one solution over another. These restraints could include:

- Proximity of adjacent buildings, which might affect the design of the structure and the temporary works.
- Size of the site and working space, which may limit the space for site offices and storage areas.
- Access to the site, e.g. city centre site, which could limit the size of vehicles that can access the site.
- Remoteness of the site, which may place limitations on the available materials, labour, plant and the size of vehicle that can reach the site.
- Restrictions on working hours.
- Effect of site processes on neighbours and general public e.g. the effects of noise and pollution. Self-compacting concrete may assist by eliminating the need for vibration.
- Availability of materials. Concrete is the most widely used construction material in the world and the essential ingredients are also found throughout the world.

2.7.7 Speed of construction

It is often assumed that a steel frame will offer a faster form of construction than a concrete frame. This will not necessarily be the case. In-situ concrete has a much reduced lead-in time, often only five weeks, so work can often start on site more quickly, whereas a steel frame may have a lead-in time of 12 weeks. Where there is time for pre-planning, design and precasting, a precast concrete frame can be erected quickly. *Building*[13] magazine regularly updates information on lead-in times.

A key date on any programme for a building is the date that it is weatherproof. Although a prefabricated frame (either steel or precast concrete) can be erected quickly, it does not necessarily follow that the date the building is sealed will be earlier. This relies on follow-on trades, particularly the cladding, which may be able to progress in parallel with an in-situ concrete frame, but which may be delayed until the floors are completed for other types of frame.

With an in-situ frame, installation of services may be commenced earlier in the programme as the floors are usually cast with the frame, giving an immediate substrate to which the services can be fixed. If flat soffits are used, the programme can be reduced because the installation is simpler.

Candidates will need to make their own assessment of the relative speed of construction for their solutions, and whether it is an important criterion for the client. Further guidance on construction periods is given in Section 5.

2.7.8 Aesthetics

Concrete can offer a pleasing aesthetic solution, and this can be achieved with either precast or in-situ concrete. Concrete provides the opportunity to create unusual shapes at a small cost premium. This can be particularly beneficial if circular columns are required for aesthetic reasons or where columns need to be contained in walls, e.g. for apartments. Concrete can also be used for curved beams, unusual plan shapes and shell structures. The layout of the vertical structure can be arranged to suit the use of the building rather than having to rigidly follow a structural grid.

2.7.9 Acoustics

Concrete is a very good sound insulator, even when the source of noise is an impact on the face of the concrete. For this reason concrete floors and walls are often used in residential accommodation, including flats, hotels and student residences, to prevent the passage of sound between units.

Concrete can also be used to prevent the passage of sound into or out of a building. A good example would be the use of concrete floors beneath plant on the roof of a building to prevent the noise penetrating the habitable areas.

2.7.10 Footfall-induced vibration

For some types of buildings the control of vibrations induced by people walking across the floor plate are important. This is particularly the case for hospitals and laboratories containing sensitive equipment, but even in offices long slender spans can cause excessive vibration.

The inherent mass of concrete means that concrete floors generally meet vibration criteria at no extra cost as they do not require additional stiffening. For more stringent criteria, such as for laboratories or hospital operating theatres, the additional cost to meet vibration criteria is small compared with other structural materials.

An independent study[14] into the vibration performance of different structural forms in hospitals has confirmed that concrete can normally be easily designed for the most complete control of vibration over whole areas, without the need for significantly thicker floor slabs than those used for a basic 'office' structure. This gives great flexibility for change in use and avoids the cost penalties of providing this extra mass and stiffness. The findings are summarised in Table 2.8, which could be used as an initial guide as to how much the depth of a floor would have to be increased to control vibration in a hospital. Laboratories are likely to have more stringent criteria.

Table 2.8
Indication of typical structural depths to control vibration [14]

	Concrete				Steel			
	Flat slab		Post-tensioned slab		Composite		Slimdek	
Overall depth	mm	% change	mm	% change	mm	% change	mm	% change
Office areas	300	0	220	0	536	0	316	0
Night ward	330	10	250	14	733	37	424	34
Operating theatre	350	17	290	32	783	46	470	49
Mass	kg/m^2	% change	kg/m^2	% change	kg/m^2	% change	kg/m^2	% change
Office areas	753	0	536	0	227	0	449	0
Night ward	820	9	608	13	525	131	715	59
Operating theatre	868	15	702	31	644	183	818	82

Note
Results based on an analysis of bays 7.5 m x 7.5 m in 15 x 2 bay layout.

2.7.11 Thermal mass

Concrete has a high thermal mass, which makes it ideal to use as part of a fabric energy storage (FES) system. FES utilises the thermal mass of concrete to absorb internal heat gains during a summer's day to help prevent overheating and providing a more stable internal temperature. Night cooling purges the accumulated heat from the slab, preparing it for the next day. FES can be used on its own or as part of a mixed mode system to reduce the energy requirements. The important requirement is to expose the soffit of the slab, or at least allow the air from the room to flow in contact with the concrete. This impacts on the structural solution and should be considered at the early stages of a project. Thermal mass can also be used to maintain warmth in a building during the winter.

Design solutions that allow the soffit to be exposed in an aesthetically pleasing way and provide for cooling are shown in Figure 2.6.

2.7.12 Sustainability

Sustainability is not just about reducing environmental impact; it is finding a balance between social, economic and environmental costs and benefits, both now and in the long term.

A correctly detailed and constructed concrete framed building will last at least 100 years, and should last considerably longer. However, the structural frame will have to provide a flexible layout for the building if it is to be put to a variety of uses over its lifetime. A small increase in expenditure to provide clear spans now may have a significant benefit in environmental and cost terms in the future. With its relatively high self-weight, a concrete frame can easily be adapted to other uses that may require a heavier imposed load. Holes can be cut through slabs and walls relatively simply, and there are methods to strengthen the frame if required.

With regard to the embodied carbon dioxide content (i.e. the carbon dioxide produced in the manufacture and construction of the structure), this is generally very small compared with the carbon dioxide produced in heating and lighting the building during its lifetime. The longer the building lasts the less significant the embodied carbon dioxide becomes.

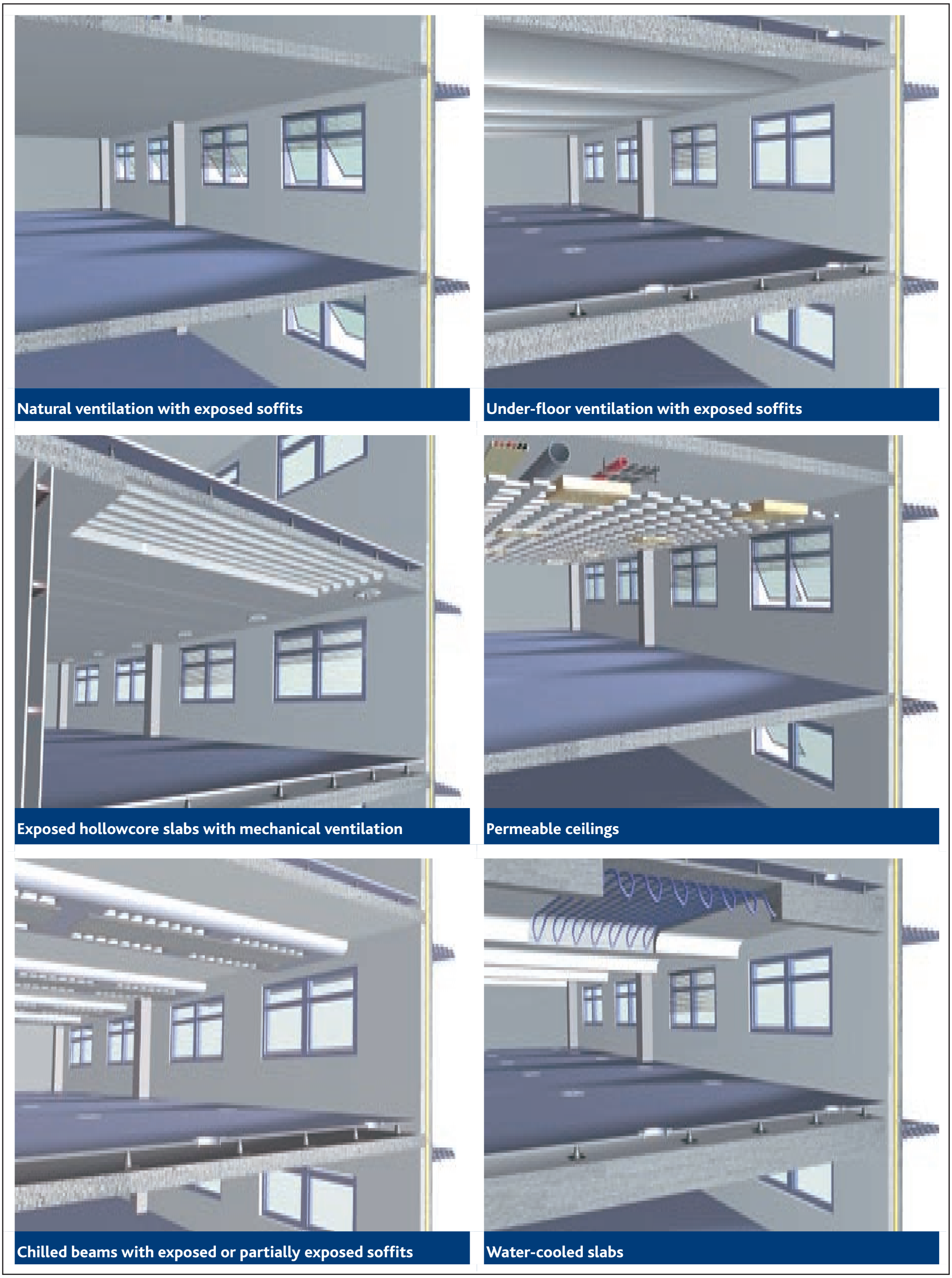


Figure 2.6
Typical concrete floors suitable for use with fabric energy storage

The energy used in heating and cooling the building can be substantially reduced by using fabric energy storage (see Section 2.7.11). The mass of the structure of a private house can have a considerable impact on the energy requirements of keeping the house at a comfortable temperature level (see Figure 2.7).

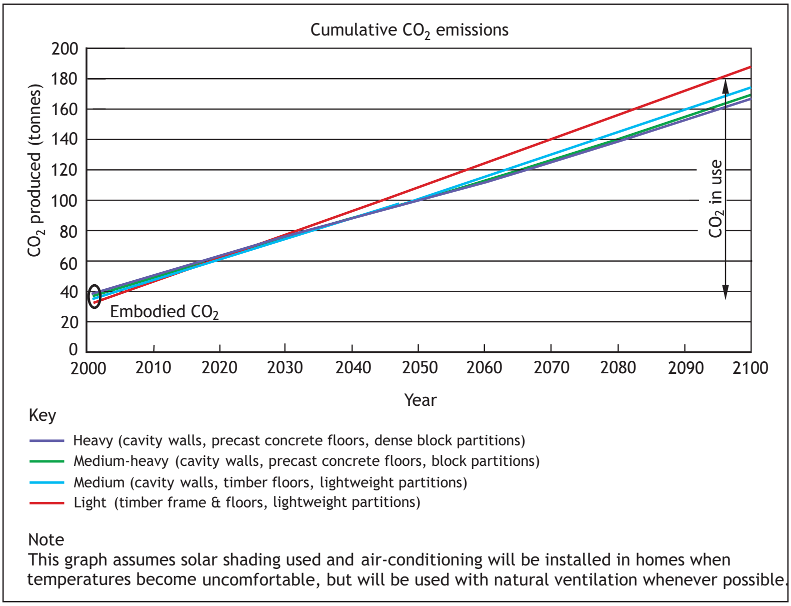


Figure 2.7
Predicted cumulative $C0_2$ emissions for different housing types[15]

2.7.13 Building movements

Movements in concrete buildings can be caused by:

- Autogenous shrinkage
- Long-term drying shrinkage
- Early thermal contraction
- Temperature variations – daily or seasonal expansion and contraction
- Creep – time-dependent increase in compressive strain for a constant compressive stress significantly increases deflections in the long term
- Settlement
- Deflection
- Solar radiation
- Pre-stressing (immediate and long-term)

For a typical building located in the UK, 25 mm wide movement joints located at 50 to 70 m centres will normally be adequate to deal with the effects of shrinkage and temperature variations. The restraint imposed on the slab will depend on the layout of the stability walls (see Figure 2.8). Movement joints may also be required at changes in shape of the building in plan or elevation. Remember that if a movement joint is introduced it should be placed so that it is vertical throughout the height of the structure and each part of the building should be stable.

Movements due to differential settlement are often more critical than overall settlement. Where buildings are founded on varying ground conditions the effect of settlement should be considered and movement joints provided where necessary.

The effect of creep can be important in the design of post-tensioned structures and in tall structures where the shortening of concrete columns should be considered in the design of the cladding system.

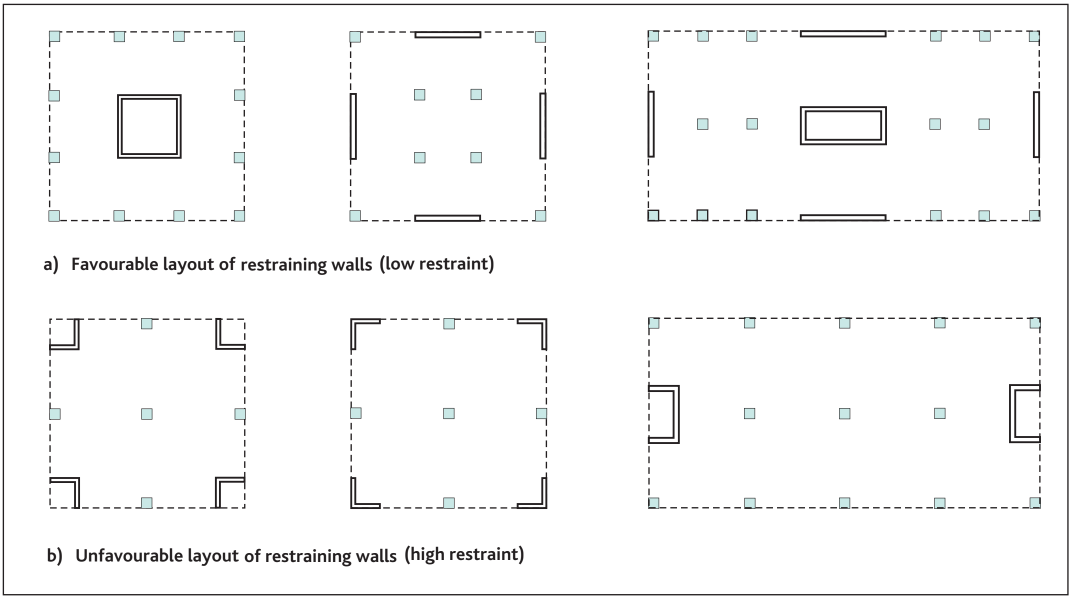


Figure 2.8
Typical floor layouts
Diagram: Concrete Society Technical Report 43 [16]

2.7.14 Fire resistance

Concrete is inherently fire resistant. It is non-combustible and has a slow rate of heat transfer, which makes it a highly effective barrier to the spread of fire. The recommendations for cover to reinforcement in BS 8110 should be followed to ensure fire protection for the specified periods (see Table 3.2 on page 42).

2.8 Typical loads

There is often some guidance provided in the question on imposed loads. However, candidates may well have to make their own assessment of self-weight and other dead loads. Some information on weights of materials and typical load build-ups is given in Tables 2.9 to 2.11.

Table 2.9a
Bulk loads for soils

Material	Bulk density (kN/m^3)	Material	Bulk density (kN/m^3)
Chalk	22	Granular – medium dense	18 – 19
Clay	16 – 22	Granular – dense	19 – 21
Clay – stiff	19 – 22	Granular – very dense	21
Clay – firm	17 – 20	Peat	11
Clay – soft	16 –19	Silty clay	16 – 20
Granular – very loose	<16	Sandy clay	16 – 20
Granular – loose	16 – 18		

Table 2.9b
Bulk loads for materials

Material	Bulk density (kN/m^3)	Material	Bulk density (kN/m^3)
Aluminium	27.2	Gold	194.0
Asphalt	22.5	Granite	27.3
Blocks – aerated concrete (min)	5.0	Hardcore	19.0
Blocks – aerated concrete (max)	9.0	Iron	77.0
Blocks – dense aggregate	20.0	Lead	111.1
Blocks – lightweight	14.0	Limestone (Bathstone – lightweight)	20.4
Books – bulk storage	8 – 11	Limestone (Portland stone – med weight)	22.0
Books on shelves	7.0	Limestone (marble – heavyweight)	26.7
Brass – cast	85.0	Macadam paving	21.0
Brickwork – blue	24.0	MDF	8.0
Brickwork – engineering	22.0	Plaster	14.1
Brickwork – fletton	18.0	Plywood	6.3
Brickwork – London stock	19.0	Sandstone	23.5
Brickwork – sand lime	21.0	Screed – sand/cement	22.0
Bronze – cast	83.0	Steel	77.0
Chipboard	6.9	Terracotta	20.7
Coal	9.0	Timber – Douglas fir	5.2
Concrete – aerated	10.0	Timber – European beech/oak	7.1
Concrete – lightweight	18.0	Timber – Grade C16	3.6
Concrete – normal	24.0	Timber – Grade C24	4.1
Copper	87.0	Timber – Iroko/teak	6.4
Glass	25.6		

Table 2.9c
Typical self-weights of concrete floors

Material	Area load (kN/m^2)	Material	Area load (kN/m^2)
Precast concrete solid units (100 mm)	2.40	Ribbed slab (325 mm)	4.40
Precast concrete hollowcore units (150 mm)	2.40	Ribbed slab (350 mm)	4.70
Precast concrete hollowcore units (200 mm)	2.87	Ribbed slab (400 mm)	5.00
Precast concrete hollowcore units (250 mm)	3.66	Ribbed slab (450 mm)	5.30
Precast concrete hollowcore units (300 mm)	4.07	Ribbed slab (500 mm)	5.70
Precast concrete hollowcore units (350 mm)	4.45	Waffle slab – standard moulds (325 mm)	6.00
Precast concrete hollowcore units (400 mm)	4.84	Waffle slab – standard moulds (350 mm)	6.40
Precast concrete hollowcore units (450 mm)	5.50	Waffle slab – standard moulds (425 mm)	7.30
Ribbed slab (250 mm)	4.00	Waffle slab – standard moulds (450 mm)	7.70
Ribbed slab (275 mm)	4.20	Waffle slab – standard moulds (525 mm)	8.60
Ribbed slab (300 mm)	4.30		

Table 2.9d
Area loads and loads of sheet materials

Material	Area load (kN/m^2)	Material	Area load (kN/m^2)
Aluminium (corrugated)	0.04	Plaster skim coat	0.05
Asphalt (20 mm)	0.46	Plasterboard (9.5 mm)	0.07
Carpet	0.03	Plasterboard (12.5 mm)	0.09
Carpet and underlay	0.05	Plasterboard (15 mm)	0.11
Chipboard (18 mm)	0.12	Plasterboard (19 mm)	0.15
Chipboard (22 mm)	0.15	Plastered brick 102 + 2 x 13 mm	2.60
Dry lining on stud (20 mm)	0.15	Plastered medium density block	2.00
False ceiling – steel framing	0.10	Plastic (corrugated)	0.04
Felt (3-layer) and chippings	0.35	Plywood (12.5 mm)	0.08
Glass – double glazing	0.52	Plywood (15.5 mm)	0.10
Glass – single glazing	0.30	Plywood (19 mm)	0.12
Glass fibre (50 mm)	0.01	Quarry tiles including mortar bedding	0.32
Glass wool (100 mm)	0.01	Raised floor – heavy duty	0.50
Glazing – patent (incl. bars)	0.30	Raised floor – medium weight	0.40
Hardwood parquet (10 mm)	0.08	Raised floor – lightweight	0.30
Insulation – glass fibre (100 mm)	0.02	Render (13 mm)	0.30
Insulation – glass fibre (150 mm)	0.03	Rock wool (25 mm)	0.01
Insulation – polystyrene	0.04	Screed – 50 mm	1.15
Lead sheet – code 3 (1.32 mm)	0.15	Screed – lightweight (25 mm)	0.45
Lead sheet – code 4 (1.80 mm)	0.20	Sheet vinyl	0.03
Lead sheet – code 5 (2.24 mm)	0.25	Stainless steel roofing (0.4 mm)	0.05
Lead sheet – code 6 (2.65 mm)	0.29	Steel (corrugated)	0.15
Lead sheet – code 7 (3.15 mm)	0.35	Suspended ceiling – aluminium	0.05
Lead sheet – code 8 (3.55 mm)	0.39	Suspended ceiling – steel	0.10
Linoleum (3.2 mm)	0.05	Suspended fibreboard tiles	0.05
Paving stones (50 mm)	1.20	T&G boards (15.5 mm)	0.09
Perspex corrugated sheets	0.05	T&G boards (22 mm)	0.12
Plaster - two coat gypsum (12 mm)	0.21	Tiles – ceramic floor on bed	1.00
Plaster board on timber stud	0.35		

Table 2.9e
Roof loads

Material	Area load (kN/m^2)
Battens for slating and tiling	0.03
Metal roof cladding	0.07
Thatch (300 mm)	0.45
Tiles – clay roof (max)	0.67
Tiles – clay roof (min)	0.43
Tiles – natural slate (thick)	0.65
Tiles – natural slate (thin)	0.35
Tiles – interlocking concrete	0.55
Tiles – plain concrete	0.75
Zinc roofing (0.8 mm)	0.06

Table 2.9f
Area loads of partitions

Partitions as area loads	Area load (kN/m^2)
Lightweight	1.00
Heavyweight	3.00

Table 2.10
Typical area loads for cladding and walls (kN/m²)

Cavity wall	
102.5 mm brickwork	2.40
50 mm insulation	0.02
100 mm blockwork	1.40
Plaster	0.21
Total	**4.0**

Lightweight cladding	
Insulated panel	0.20
Purlins	0.05
Dry lining on stud	0.15
Total	**0.4**

Curtain walling	
Allow	1.00

Insulating concrete formwork	
Render	0.60
Insulating formwork	0.10
Concrete (200 mm)	4.80
Plaster	0.21
Total	**5.7**

Timber cladding	
Boarding (25 mm)	0.20
Battens	0.05
Ply (12.5 mm)	0.08
Studs (150 x 50 mm)	0.23
Insulation	0.05
Ply (12.5 mm)	0.08
Dry lining on stud	0.15
Total	**0.8**

Precast concrete cladding	
Facing	1.00
Precast panel (100 mm)	2.40
Insulation	0.05
Dry lining on stud	0.15
Total	**3.6**

Brickwork	
102.5 mm brickwork	2.40
Plaster x 2	0.42
Total	**2.8**

Blockwork – dense	
100 mm dense blockwork	2.00
Plaster x 2	0.42
Total	**2.4**

Blockwork – lightweight	
100 mm lightweight blockwork	1.40
Plaster x 2	0.42
Total	**1.8**

Blockwork – aerated	
100 mm aerated blockwork	0.50
Plaster x 2	0.42
Total	**0.9**

Dry lining	
Metal studs	0.05
Plasterboard and skim x 2	0.40
Total	**0.5**

Timber stud wall	
Timber studs	0.10
Plasterboard and skim x 2	0.40
Total	**0.5**

Table 2.11
Typical area loads (kN/m²)

Office floor						
In-situ flat slab thickness (mm)	**–**	**225**	**250**	**275**	**300**	**325**
Carpet	0.03	0.03	0.03	0.03	0.03	0.03
Raised floor	0.30	0.30	0.30	0.30	0.30	0.30
Self-weight of slab	–	5.40	6.00	6.60	7.20	7.80
Suspended ceiling	0.15	0.15	0.15	0.15	0.15	0.15
Services	0.30	0.30	0.30	0.30	0.30	0.30
Total	**0.8**	**6.2**	**6.8**	**7.4**	**8.0**	**8.6**

Office core area						
In-situ flat slab thickness (mm)	**–**	**225**	**250**	**275**	**300**	**325**
Tiles & bedding (allow)	1.00	1.00	1.00	1.00	1.00	1.00
Screed	2.20	2.20	2.20	2.20	2.20	2.20
Self-weight of slab	–	5.40	6.00	6.60	7.20	7.80
Suspended ceiling	0.15	0.15	0.15	0.15	0.15	0.15
Services	0.30	0.30	0.30	0.30	0.30	0.30
Total	**3.7**	**9.1**	**9.7**	**10.3**	**10.9**	**11.5**

Table 2.11 continued ...

Residential floor					
In-situ flat slab thickness (mm)	**–**	**200**	**225**	**250**	**275**
Carpet	0.05	0.05	0.05	0.05	0.05
Floating floor	0.15	0.15	0.15	0.15	0.15
Self-weight of slab	–	4.80	5.40	6.00	6.60
Suspended ceiling	0.20	0.20	0.20	0.20	0.20
Services	0.10	0.10	0.10	0.10	0.10
Total	**0.5**	**5.3**	**5.9**	**6.5**	**7.1**

School floor					
In-situ flat slab thickness (mm)	**–**	**225**	**250**	**275**	**300**
Carpet/flooring	0.05	0.05	0.05	0.05	0.05
Self-weight of slab	–	5.40	6.00	6.60	7.20
Suspended ceiling	0.15	0.15	0.15	0.15	0.15
Services	0.20	0.20	0.20	0.20	0.20
Total	**0.4**	**5.8**	**6.4**	**7.0**	**7.6**

Hospital floor					
In-situ flat slab thickness (mm)	**–**	**225**	**250**	**275**	**300**
Flooring	0.05	0.05	0.05	0.05	0.05
Self-weight of slab	–	5.40	6.00	6.60	7.20
Screed	2.20	2.20	2.20	2.20	2.20
Suspended ceiling	0.15	0.15	0.15	0.15	0.15
Services (but can be greater)	0.50	0.50	0.50	0.50	0.50
Total	**2.9**	**8.3**	**8.9**	**9.5**	**10.1**

Flat roof/external terrace						
In-situ flat slab thickness (mm)	**–**	**225**	**250**	**275**	**300**	**325**
Paving or gravel (allow)	2.20	2.20	2.20	2.20	2.20	2.20
Waterproofing	0.50	0.50	0.50	0.50	0.50	0.50
Insulation	0.10	0.10	0.10	0.10	0.10	0.10
Self-weight of slab	–	5.40	6.00	6.60	7.20	7.80
Suspended ceiling	0.15	0.15	0.15	0.15	0.15	0.15
Services	0.30	0.30	0.30	0.30	0.30	0.30
Total	**3.3**	**8.7**	**9.3**	**9.9**	**10.5**	**11.1**

Timber pitched roof				
Pitch (°)	**30**	**35**	**40**	**45**
Tiles (range: 0.5 to 0.75 kN/m^2)	0.75	0.75	0.75	0.75
Battens	0.05	0.05	0.05	0.05
Felt	0.05	0.05	0.05	0.05
Rafters	0.15	0.15	0.15	0.15
Insulation	0.05	0.05	0.05	0.05
Plasterboard and skim	0.15	0.15	0.15	0.15
Services	0.10	0.10	0.10	0.10
Ceiling joists	0.15	0.15	0.15	0.15
Total load on plan	**1.6**	**1.7**	**1.8**	**1.9**

Metal decking roof	
Insulated panel	0.20
Purlins	0.10
Steelwork	0.30
Services	0.10
Total	**0.7**

2.9 Typical spatial requirements

2.9.1 Car parks

The standard configuration for car parks is shown in Figure 2.9. Ideally the 'bin width' i.e. the length of two car park spaces and the aisle should be a clear span of 15.6 m. This is an onerous requirement and so the columns can be placed between spaces as shown in Figure 2.9. The sizes of the parking spaces are shown in Table 2.12.

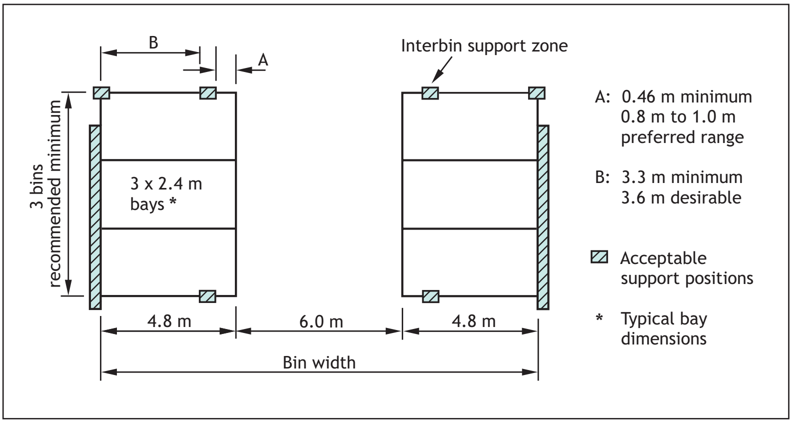


Figure 2.9
Typical car park layout
Diagram: IStructE [17]

Table 2.12
Car park space requirements

Type of parking	Length (m)	Width (m)	Comment
Mixed use	4.8	2.4	Mixed occupancy
Short stay	4.8	2.5	< 2 hours
Long stay	4.8	2.3	One movement per day
Disabled user	4.8	3.6	–
Parent/child	4.8	3.2	–

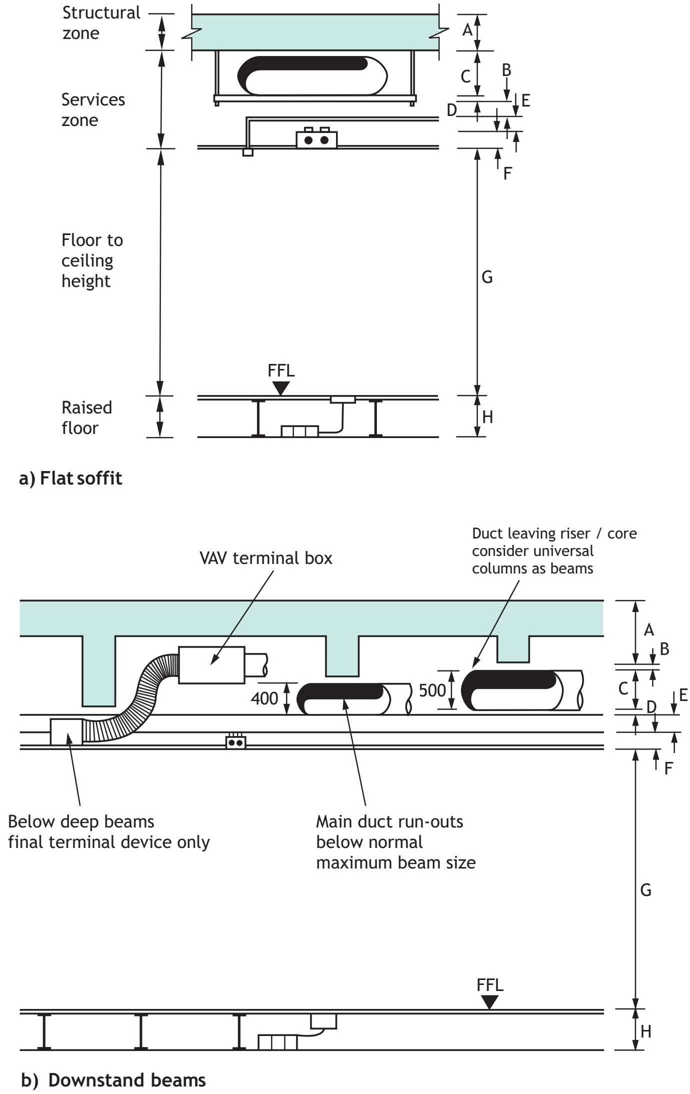
2.9.2 Services

Typical arrangements for integration of structure and sevices are given in Figure 2.10.

2.10 Preliminary sizing

It is strongly recommended that *Economic concrete frame elements*[2] is referred to for preliminary sizing of concrete framed structures. However, as a preliminary guide, the span-to-effective-depth ratios below can be used.

Candidates are expected to show some calculations in Section 1a of the examination, especially for items such as transfer beams and foundations. Indeed candidates will have to do some calculation at this stage to avoid using inappropriate sizes.



Key

A Structural zone
B 50 mm deflection and tolerance
C Approx. 500 mm HVAC duct or terminal device
D 50 mm support and tolerance
E 50 – 150 mm sprinkler zone (if required)
F 150 mm lighting and ceiling zone
G Floor to ceiling height
H Raised floor

Figure 2.10
Typical service zones

2.10.1 Beams

The span-to-depth ratios in Table 2.13 can be used.

Table 2.13
Span-to-depth ratios for beams

Condition	Span-to-depth ratio
Simply supported	15
End-bay	17
Cantilever	6

2.10.2 Transfer beams

The experienced engineer will understand that it is not advisable to determine the section size merely by using span-to-depth tables – careful consideration is needed. Shear strength is often the governing criteria for a reinforced concrete transfer beam. From BS 8110, $v = V/bd$. In no case should v exceed $0.8\sqrt{f_{cu}}$ or 5 N/mm^2 (whichever is smaller). If the section is not to become congested with shear reinforcement it is advisable to limit v to 2 N/mm^2. However, it may be necessary to increase this to 4 N/mm^2. If we assume a well proportioned beam has a width, b, which is half it's depth, then we rearrange the expression above so that:

$d = \sqrt{V}$, where V is in newtons (N) for shear stress of 2 N/mm^2

$d = \sqrt{(V/2)}$ for shear stress of 4 N/mm^2

The headroom under the beam should be checked and consideration given to the connection into the column. Deflection and flexural strength should also be considered because they may govern the design.

2.10.3 One-way spanning slabs

The span-to-depth ratios in Table 2.14 may be used for spans in the range 4 to 10 m.

Table 2.14
Span-to-depth ratios for one-way spanning slabs

Imposed load, Q_k (kN/m^2)	Single span	Multiple span	Cantilever
2.5	27	32	10
5.0	25	30	9
7.5	24	28	8
10.0	23	27	7

2.10.4 Two-way spanning slabs

The span-to-depth ratios in Table 2.15 may be used where the longest span is in the range 4 to 12 m.

Table 2.15
Span-to-depth ratios for two-way spanning slabs

Imposed load, Q_k (kN/m^2)	1:1 panel		2:1 panel (based on shorter span)	
	Single span	Multiple span	Single span	Multiple span
2.5	34	39	30	34
5.0	32	37	28	32
7.5	30	35	26	30
10.0	28	34	25	29

2.10.5 Flat slabs

The span-to-depth ratios in Table 2.16 may be used where the spans are in the range 4 to 10 m.

Table 2.16
Span-to-depth ratios for flat slabs

Imposed load, Q_k (kN/m^2)	Multiple span
2.5	28
5.0	26
7.5	25
10.0	23

Note
This table assumes a 3 x 3 bay layout. Where there are only 2 bays in one direction the ratio will need to be decreased.

Punching shear is often a governing criterion for flat slabs and should be checked at the initial stages of design. Table 2.17 gives the maximum floor area for a selection of imposed loads and column sizes. It assumes a superimposed dead load of 1.5 kN/m^2, internal conditions, a value for v_c of 0.75 N/mm^2, with v limited to $1.6v_c$.

2.10.6 Ribbed slabs

The span-to-depth ratios in Table 2.18 may be used where the spans are in the range 6 to 12 m. Ribbed slabs should be orientated with the ribs running parallel to the longest edge. The most economic ratio of the spans is 4:3.

Table 2.18
Span-to-depth ratios for ribbed slabs

Imposed load, Q_k (kN/m^2)	Supported by beams[a]			Ribs integral with band beam <11 m
	Single span <12 m	Multiple span		
		<10 m	10 – 12 m	
2.5	24	29	27	25
5.0	21	27	24	23
7.5	19	25	21	21
10.0	17	23	17	18

Key
a Refer to Section 2.10.1 to determine depth of beams

2.10.7 Waffle slabs

The span-to-depth ratios in Table 2.19 may be used where the spans are in the range 6 to 12 m.

Table 2.19
Span-to-depth ratios for waffle slabs

Imposed load, Q_k (kN/m^2)	1:1 panel	1.5:1 panel (depth based on shorter span)
	Multiple span	Multiple span
2.5	23	17
5.0	21	16
7.5	19	15
10.0	18	14

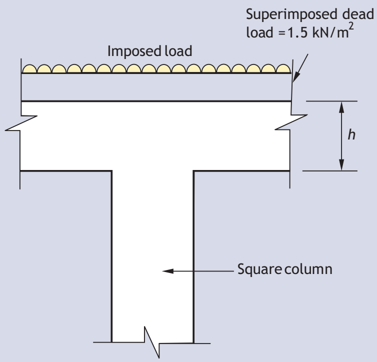
Table 2.17
Punching shear: maximum panel areas for flat slabs (m^2)

Overall slab depth, *h* (mm)	Imposed load (kN/m^2)			
	2.5	5.0	7.5	10
300 x 300 column				
200	38.6	29.4	23.8	19.9
225	46.2	35.7	29.1	24.6
250	54.0	42.3	34.8	29.5
275	59.7	47.3	39.2	33.5
300	67.7	54.3	45.3	38.9
325	75.9	61.4	51.6	44.5
350 x 350 column				
200	41.1	31.3	25.3	21.2
225	49.0	37.9	30.9	26.1
250	56.9	44.6	36.7	31.2
275	62.8	49.8	41.2	35.2
300	70.9	56.9	47.5	40.7
325	79.2	64.2	53.9	46.5
400 x 400 column				
200	43.7	33.3	26.9	22.5
225	51.7	40.0	32.6	27.5
250	59.9	46.9	38.6	32.8
275	65.8	52.2	43.3	36.9
300	74.2	59.5	49.6	42.6
325	82.6	66.9	56.2	48.5

Overall slab depth, *h* (mm)	Imposed load (kN/m^2)			
	2.5	5.0	7.5	10
450 x 450 column				
200	46.2	35.2	28.4	23.9
225	54.5	42.1	34.3	29.0
250	62.8	49.3	40.5	34.4
275	68.9	54.6	45.3	38.6
300	77.4	62.1	51.8	44.4
325	86.0	69.6	58.5	50.4
500 x 500 column				
200	48.7	37.1	30.0	25.2
225	57.2	44.2	36.1	30.5
250	65.8	51.6	42.4	36.0
275	71.9	57.1	47.3	40.4
300	80.6	64.6	53.9	46.3
325	89.4	72.4	60.8	52.4

Notes

1. Superimposed load of 1.5 kN/m^2 included.
2. Cover of 25 mm has been assumed.
3. v_c for main reinforcement is 0.75 N/mm^2.
4. v for punching reinforcement is limited to 1.6 v_c.
5. Shear links should be provided in accordance with BS 8110.



How to use this table

For example:
300 x 300 column
250 thick slab
5 kN/m^2 imposed load
From table maximum area that can be supported = 42.3 m^2
(e.g. 6.5 x 6.5 m grid)

2.10.8 Post-tensioned slabs and beams

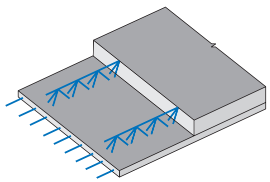
Table 2.20 can be used for initial sizing of post-tensioned slabs, where the spans are in the range 6 to 13 m.

Table 2.20
Span-to-depth ratios for post-tensioned slabs and beams

Imposed load, Q_k (kN/m^2)	Flat slab	Flat slab with band beams		Ribbed slab	Waffle slab (with solid slab at column head)	One-way slab on deep beam	
		Slab	Beam			Slab	Beam
2.5	40	45	25	30	28	42	18
5.0	36	40	22	27	26	38	16
10.0	30	35	18	24	23	34	13

2.10.9 Precast concrete floor units

Figures 2.11 to 2.14 may be used for initial sizing; these are based on manufacturers' data.



Notes
1 Depth includes topping
2 Props required at mid-point

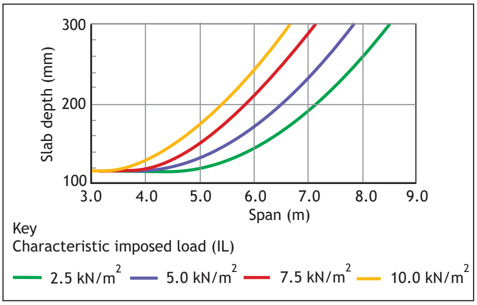
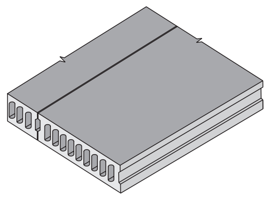


Figure 2.11
Composite lattice girder slabs



Note
Depth excludes topping

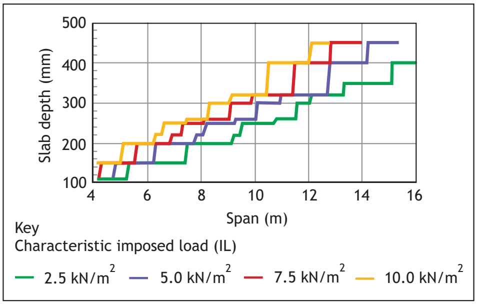


Figure 2.12
Precast hollowcore slabs

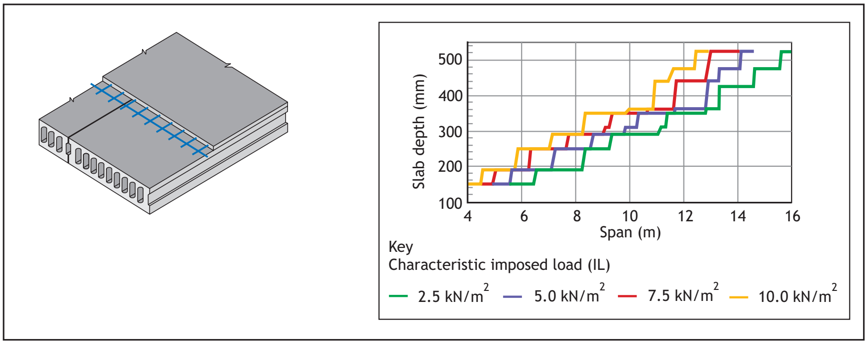


Figure 2.13
Composite hollowcore slabs

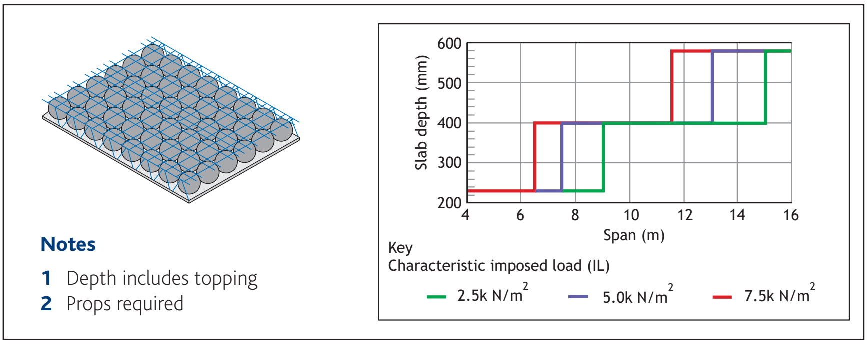


Figure 2.14
Voided slabs (Note: with some systems the void formers can be used with in-situ concrete)

2.10.10 Columns

Tables 2.21 to 2.23 may be used for initial sizing. This is a summary of the data contained in *Economic concrete frame elements*[2] and should be used with the following cautions:

- Loads are ultimate loads in kN.
- Internal columns are assumed to support slabs or beams of similar spans in each orthogonal direction.
- Imposed moments on edge and corner columns have been assumed; for imposed loads greater than 5.0 kN/m^2 alternative justification is required.
- Columns are 'short' and 'braced'.

Concrete columns can be concealed within partitions by using 'blade' columns. Often a 200 x 800 mm section is used because 200 mm is a practical minimum thickness and 800 mm is four times the thickness, which classifies it as a wall. For fire resistance this reduces the cover requirements compared with a column.

Table 2.21
Initial sizing for internal square columns (mm)

Percentage of reinforcement	Ultimate axial load, kN (Class C28/35 concrete)								
	1000	1500	2000	3000	4000	5000	6000	8000	10000
1.0%	240	295	345	420	485	540	595	685	765
2.0%	225	270	310	380	440	490	540	620	695
3.0%	225	250	285	350	405	455	500	570	640
4.0%	225	230	270	330	380	425	465	535	595

Table 2.22
Initial sizing for square edge columns (mm)

	Ultimate axial load, kN (3% rebar, class C28/35 concrete)								
	400	800	1200	1600	2000	3000	4000	5000	6000
2 storeys	230	305	380	450	505				
3 storeys	225	235	280	340	400	505	575		
4 storeys	225	225	260	305	345	435	505	555	
6 storeys	225	225	250	280	315	395	455	515	560

Table 2.23
Initial sizing for square corner columns (mm)

	Ultimate axial load, kN (3% rebar, class C28/35 concrete)								
	200	400	600	800	1000	1200	1600	2000	3000
2 storeys	265	315	410	485	555	–	–	–	–
3 storeys	245	255	305	375	435	485	574	–	–
4 storeys	245	235	270	300	360	410	490	559	–
6 storeys	240	225	225	240	275	315	385	450	569

2.10.11 Shear walls

Shear walls are essentially vertical cantilevers, and may be sized as such; therefore a span-to-depth ratio of 7 is reasonable for a shear wall. However, at this aspect ratio it is highly likely that tension will be developed at the base and this requires justification in the design (see Figure 2.15). Pad foundations should be designed to resist overturning and piles may be required to resist tension.

The wall should be checked to ensure that it is 'short' (see Table 2.24); the minimum practical thickness is 200 mm. The wall should be 'braced', i.e. there should be another shear wall in the orthogonal direction.

Table 2.24
Recommended minimum thicknesses for shear walls

Thickness (mm)	Maximum clear height (m)
200	3.5
215	3.8
250	4.4
300	5.3

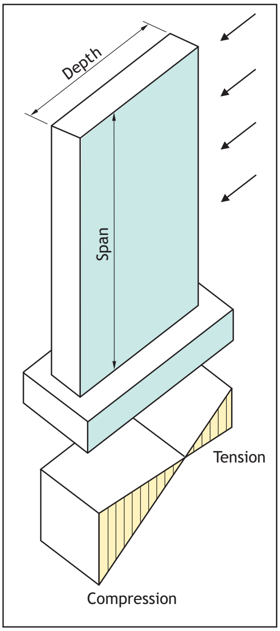


Figure 2.15
Lateral forces on shear wall

2.11 The letter

Section 1b asks candidates to write a letter to their client in response to a change to the scheme. This is a good test of the ability to communicate effectively to a non-technical reader. The examiners will also be assessing the candidate's appreciation of the structural implications of the changes. The examiner's reports regularly highlight that the purpose of the letter is **not** to seek additional design fees, i.e. the purpose of the letter is to deal with the structural aspects of the request not the business aspect.

It should be laid out as a letter and include sketches if necessary, and a recommended course of action. Candidates should NOT sign it, as the scripts should remain anonymous. Include any calculations carried out in preparing the letter in the script (see Figure 2.16).

Candidates should therefore give an explanation of the structural problem, together with a solution to that problem. It should outline clearly and fully how the client's request might be achieved, in terms of:

- Cost
- Programme
- Function
- Safety
- Aesthetics

The ability to write appropriate letters can only come through practical experience. Those candidates who have experience of writing similar letters will be able to undertake this task with confidence. Those who lack experience will find it a challenge and should ensure they prepare adequately for this section of the examination. It is often a good idea to read and understand what is required for the letter before tackling Section 1a of the examination. If the solution required in the letter appears trivial then it may well be that the candidate has overlooked an important part of the question.

Engineering Consultants
99 High Street
Newtown
NW9 9AA

Your ref:
Our ref: 1099/OB/1.01/21-11-06

21 November 2006

FAO Mr J Client

Dear Mr Client

Call Centre – Implications of Recent Minor Seismic Activity

As we are sure you are aware there has recently been a minor earthquake in the region of the recently completed Call Centre. We would like to address the concerns you may have regarding the strength of your building if faced with a similar event.

Initially we would like to confirm that the structure has been designed to the latest Building Regulations and Codes of Practice. There is no requirement to explicitly consider the effects of seismic activity because historically the most significant horizontal forces on structures in this region are wind forces, rather than forces arising from seismic activity.

We would suggest that, before making any decisions about strengthening the structure, we wait for a consensus amongst the experts as to whether this type of event is likely to occur more frequently in the area and what they recommend as the design requirements. We would expect that, if changes are necessary, the Building Regulations will be revised and that clear guidance will emerge on whether existing buildings should be strengthened. The Call Centre is just one of many buildings that could be affected by a similar earthquake in this region.

In the meantime, we would like to reassure you that the building as it stands will have reasonable strength to resist to a minor seismic event. It has been designed with ties to ensure that 'progressive collapse' does not occur and these will act to strengthen the building in the event of an earthquake. There are, however, a number of specific areas where localised damage could occur as a result of a small earthquake; these are detailed below.

- An earthquake-resistant building should be able to carry the lateral loads imposed to a stability core. In this case the precast units and topping screed are designed to act as a stiff diaphragm. However, depending on the intensity of the earthquake, this system may have insufficient strength to carry the forces. A design check can be carried out when there is agreement on the design forces that should be used for an earthquake in this region.
- The current building is divided into two discrete structures, with a small movement joint between them. During an earthquake the structures will move laterally and the width of the joint is unlikely to be sufficient to prevent the two structures from making contact. We think there are two options for overcoming this; either the joint could be made wider by making alterations to the existing structure, or the lateral stiffness of the building could be increased by incorporating more stability walls. The latter would reduce the sideways sway during a seismic event. It may also be the preferred solution if the floor diaphragm requires strengthening. We will need to have a discussion so we can find an acceptable solution that provides minimal disruption to your business and the minimum effect on the architecture.
- The building is founded on weathered rock, which should be sufficiently robust in the event of an earthquake. However, we will need to review the strength of the foundations to resist lateral forces, for which they were not designed. The building has a suspended ground floor, which means that there are beams tying the foundations together. This is beneficial in the event of an earthquake, because it prevents the foundations moving apart possibly causing major damage to the frame.
- The beam-to-column connections are a cause for concern. There are many ways to form a connection and this is generally left to the precast manufacturer to design. This is usually beneficial because he can carry out the work more efficiently and to suit his working methods. On this project the manufacturer has used a 'billet' type connection, which is perfectly adequate for the job it was designed to do. However, it offers less spare capacity to resist the forces from an earthquake and strengthening may be required.
- The two-storey glazed entrance area will require some strengthening measures. The lateral movement that will occur during an earthquake is almost certain to damage the glazing. The current glazing is laminated glass, which means that it will not break into small pieces. However, there is a risk that whole panes will come away from their fixings with the potential to cause loss of life. To overcome this, strengthening of the column–to-roof beam connections will be necessary to reduce lateral sway.

We cannot be certain about how design guidance will change as a result of the earthquake, but the above comments should give you an appreciation of the preventive measures that are likely to be necessary. If you would like to discuss the situation further then please contact us.

Yours sincerely

Figure 2.16
Example letter

3 Design calculations (section 2c)

3.1 Expectations of the examiners

Candidates are asked to 'Prepare sufficient design calculations to establish the form and size of all the principal elements including the foundations'. There are some key points to note from the question. Firstly it asks for **sufficient** calculations, i.e. enough to prove the design is feasible, but not so many that the candidate fails to complete the examination. Secondly, the **principal** elements must be designed i.e. not all of the elements. The initial sizing of the elements should have been carried out in section 1a of the examination. This section is asking for more detail for the elements that are out of the ordinary (e.g. transfer beams) or crucial to the design of the building. Finally, principal elements that are often specifically cited are the foundations, so candidates should ensure they are included.

The candidate has around 85 minutes to answer this part of the examination. It is expected that calculations will be undertaken for between five and seven elements, giving 12 to 17 minutes for each element. There is a total of 20 marks, so each element will gain between three and four marks, no matter how detailed the calculations for that element.

The calculations are intended to be preliminary calculations, which focus on the key issues, sufficient to justify the structural sizes. Candidates should use their experience to determine critical aspects of the design of the element.

Candidates should be aware that there are varying opinions among examiners as to what working should be shown in the calculations. Some like to see design equations included and full workings (they will give marks even where the final answer is wrong); others are content to see results from programmable calculators or look-up tables, because this is more representative of current everyday practice. This is your opportunity to demonstrate your knowledge of structural engineering and perhaps it is best to work in the way that suits you, making sure you take opportunities to demonstrate your abilities.

3.1.1 Principal elements

The following is a list of structural members that could be considered to be principal elements. It may not be a full list and for some buildings these elements might not apply:

- Stability system (including assessment of the loads)
- Foundations (including assessment of the combined effects of gravitational and lateral loads, ground-bearing capacity and specification of materials in aggressive ground)
- Design to resist uplift of structure due to high ground water level
- Piles
- Basement walls
- Retaining walls
- Basement slabs – particularly in the area resisting uplift or heave
- Transfer beams
- Columns
- Slabs
- Mezzanine floors
- Cladding supports
- Curved beams
- Deep beams
- Roof structures, particularly where they support heavy loads or sensitive equipment (e.g. swimming pools or specialist plant)

It is a good idea for candidates to list the key elements they intend to design before they undertake any of the calculations.

The preliminary design of many of these elements is covered in this section. Where they are not discussed suitable references are given in *Further reading*.

3.2 Durability and fire resistance

The cover to concrete should meet the following requirements:

- The requirements for fire resistance given in table 3.4 and figure 3.2 of BS 8110 (reproduced here as Table 3.1 and Figure 3.1).
- The requirements for durability given in BS 8500 (see Table 3.2).
- Cover to all bars to be greater than aggregate size plus 5 mm.
- Cover to main bar to be greater than bar diameter.

Where concrete is used for foundations in aggressive ground conditions Table 3.3 should be referenced to determine the ACEC-class (aggressive chemical environmental for concrete class) and hence the DC-class (design chemical class) from the final two columns of the table. Where designated concrete is to be specified this can be selected using Table 3.4. For designed concrete the DC-class is normally given in the concrete specification.

Table 3.1
Nominal cover (mm) to all reinforcement (including links) to meet specified periods of fire resistance (from table 3.4 of BS 8110)

Fire resistance (hours)	Beams [a]		Floors		Ribs		Columns[a]
	Simply supported	Continuous	Simply supported	Continuous	Simply supported	Continuous	
0.5	20[b]	20[b]	20[b]	20[b]	20[b]	20[b]	20[b]
1.0	20[b]	20[b]	20	20	20	20[b]	20[b]
1.5	20	20[b]	25	20	35	20	20
2.0	40	30	35	25	45	35	25
3.0	60	40	45	35	55	45	25
4.0	70	50	55	45	65	55	25

Notes

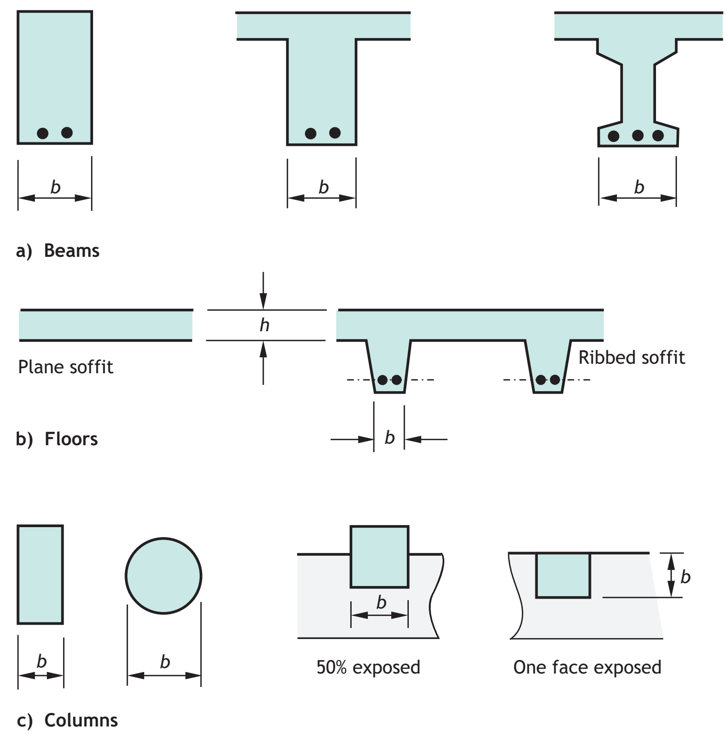
1 The nominal covers given relate specifically to the minimum member dimensions given in Figure 3.1. Guidance on increased covers, which is necessary if smaller members are used, is given in section 4 of BS 8110–2:1985.

2 Cases that lie in the shaded area require attention to the additional measures necessary to reduce the risks of spalling (see section 4 of BS 8110–2: 1985).

Key

a For the purposes of assessing a nominal cover for beams and columns, the cover to main bars which would have been obtained from tables 4.2 and 4.3 of BS 8110–2 has been reduced by a notional allowance for stirrups of 10 mm to cover the range 8 – 12 mm (see also Cl. 3.3.6 of BS 8110–1)

b These covers may be reduced to 15 mm provided that the nominal maximum size of aggregate does not exceed 15 mm (see Cl. 3.3.1.3 of BS 8110–1)



Fire resistance (hours)	Minimum beam width (b) (mm)	Rib width (b) (mm)	Minimum thickness of floors (h) (mm)	Column width (b) (mm)			Minimum wall thickness (mm)		
				Fully exposed	50 % exposed	One face exposed	$p < 0.4$ %	0.4 % $< p < 1$ %	$p > 1$ %
0.5	200	125	75	150	125	100	150	100	75
1.0	200	125	95	200	160	120	150	120	75
1.5	200	125	110	250	200	140	175	140	100
2.0	200	125	125	300	200	160	—	160	100
3.0	240	150	150	400	300	200	—	200	150
4.0	280	175	170	450	350	240	—	240	180

Notes
1 These minimum dimensions relate specifically to the covers given in Table 3.2.
2 p is the area of steel relative to that of concrete.

Figure 3.1
Minimum dimensions of reinforced concrete member for fire resistance

Table 3.2

Selected[a] recommendations for normal-weight reinforced concrete quality for combined exposure classes and cover to

Exposure conditions			Cement/ combination designations[b]
Typical example	**Primary**	**Secondary**	
Internal mass concrete	**X0**	—	All
Internal elements (except humid locations)	**XC1**	—	All
Buried concrete in AC-1 ground conditions[e]	**XC2**	**AC-1**	All
Vertical surface protected from direct rainfall	**XC3** & **XC4**	—	All except IVB
Exposed vertical surfaces		**XF1**	All except IVB
Exposed horizontal surfaces		**XF3**	All except IVB
		XF3 (air entrained)	All except IVB
Elements subject to airborne chlorides only	**XD1**[f]	—	All
Car park decks and areas subject to de-icing spray	**XD3**[f]	—	IIB-V, IIIA
			CEM I, IIA, IIB-S, SRPC
			IIIB, IVB-V
Vertical elements subject to de-icing spray and freezing		**XF2**	IIB-V, IIIA
			CEM I, IIA, IIB-S, SRPC
			IIIB, IVB-V
Car park decks, ramps and external areas subject to freezing and de-icing salts		**XF4**	CEM I, IIA, IIB-S, SRPC
		XF4 (air entrained)	IIB-V, IIIA, IIIB
Exposed vertical surfaces near coast	**XS1**[f]	**XF1**	CEM I, IIA, IIB-S, SRPC
			IIB-V, IIIA
			IIIB
Exposed horizontal surfaces near coast		**XF4**	CEM I, IIA, IIB-S, SRPC

Key

a This table comprises a selection of common exposure class combinations. Requirements for other sets of exposure classes, e.g. XD2, XS2 and XS3 should be derived from BS 8500-1: 2006 [18], Annex A

b See BS 8500-2, table 1. (CEM I is Portland cement, IIA to IVB are cement combinations)

c For prestressed concrete the minimum strength class should be C28/35

d Δc_{dev} is an allowance for deviations. The recommended value is 10 mm

e For sections less than 140 mm thick refer to BS 8500

reinforcement for at least a 50-year intended working life and 20 mm maximum aggregate size

Strength class[c], maximum w/c ratio, minimum cement or combination content (kg/m³), and equivalent designated concrete (where applicable)							
Nominal cover to reinforcement[d]							
15 + Δc_{dev}	**20 + Δc_{dev}**	**25 + Δc_{dev}**	**30 + Δc_{dev}**	**35 + Δc_{dev}**	**40 + Δc_{dev}**	**45 + Δc_{dev}**	**50 + Δc_{dev}**
Recommended that this exposure is not applied to reinforced concrete							
C20/25, 0.70, 240 or RC20/25	<<<	<<<	<<<	<<<	<<<	<<<	<<<
—	—	C25/30, 0.65, 260 or RC25/30	<<<	<<<	<<<	<<<	<<<
—	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	C25/30, 0.65, 260 or RC25/30	<<<	<<<	<<<
—	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	<<<	<<<	<<<	<<<
—	C40/50, 0.45, 340[g] or RC40/50XF[g]	<<<	<<<	<<<	<<<	<<<	<<<
—	—	C32/40, 0.55, 300 plus air [g, h]	C28/35, 0.60, 280 plus air [g, h] or PAV2	C25/30, 0.60, 280 plus air [g, h, j] or PAV1	<<<	<<<	<<<
—	—	C40/50, 0.45, 360	C32/40, 0.55, 320	C28/35, 0.60, 300	<<<	<<<	<<<
—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C28/35, 0.50, 340
—	—	—	—	—	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360
—	—	—	—	—	C32/40, 0.40, 380	C28/35, 0.45, 360	C25/30, 0.50, 340
—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C32/40, 0.50, 340
—	—	—	—	—	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360
—	—	—	—	—	C32/40, 0.40, 380	C32/40, 0.45, 360	C32/40, 0.50, 340
—	—	—	—	—	See BS 8500	C40/50, 0.40, 380[g]	<<<
—	—	—	—	—	C28/35, 0.40, 380 [g, h]	C28/35, 0.45, 360 [g, h]	C28/35, 0.50, 340 [g, h]
—	—	—	See BS 8500	C35/45, 0.45, 360	C32/40, 0.50, 340	<<<	<<<
—	—	—	See BS 8500	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55,320	<<<
—	—	—	C32/40, 0.40, 380	C25/30, 0.50, 340	C25/30, 0.50, 340	C25/30, 0.55, 320	<<<
—	—	—	See BS 8500	C40/50, 0.45, 360[g]	<<<	<<<	<<<

f Also adequate for exposure class XC3/4

g Freeze/thaw resisting aggregates should be specified.

h Air entrained concrete is required.

j This option may not be suitable for areas subject to severe abrasion.

— Not recommended

<<< Indicates that concrete quality in cell to the left should not be reduced

Table 3.3

Classification of ground conditions and selection of DC-class[a] (Based on tables A.2 and A.9 of BS 8500: 2006) [18]

Sulfate and magnesium content					Design sulfate class	Natural soil		Brownfield site[b]		ACEC-class	DC-class for 50 year intended working life
2:1 water/soil extract		Ground-water		Total potential sulfate[c]		Static water pH	Mobile water pH	Static water pH[e]	Mobile water pH[e]		
SO_4 (g/l)	Mg[d] (g/l)	SO_4 (g/l)	Mg[d] (g/l)	SO_4 (%)							
<0.5	—	<0.4	—	<0.24	DS–1	≥2.5	—	≥2.5	—	AC–1s	DC–1
						—	>5.5	—	>6.5	AC–1	DC–1
						—	2.5 – 5.5	—	5.6 – 6.5	AC–2z	DC–2z
						—	—	—	4.5 – 5.5	AC–3z	DC–3z
						—	—	—	2.5 – 4.5	AC–4z	DC–4z
0.5 –1.5	—	0.4 – 1.4	—	0.24 – 0.6	DS–2	>3.5	—	>5.5	—	AC–1s	DC–1
						—	>5.5	—	>6.5	AC–2	DC–2
						2.5 – 3.5	—	2.5 – 5.5	—	AC–2s	DC–2
						—	2.5 – 5.5	—	5.6 – 6.5	AC–3z	DC–3z
						—	—	—	4.5 – 5.5	AC–4z	DC–4z
						—	—	—	2.5 – 4.5	AC–5z	DC–4z[f]
1.6 – 3.0	—	1.5 – 3.0	—	0.7 – 1.2	DS–3	>3.5	—	>5.5	—	AC–2s	DC–2
						—	>5.5	—	>6.5	AC–3	DC–3
						2.5 – 3.5	—	2.5 – 5.5	—	AC–3s	DC–3
						—	2.5 – 5.5	—	5.6 – 6.5	AC–4	DC–4
						—	—	—	2.5 – 5.5	AC–5	DC–4[f]
3.1 – 6.0	≤1.2	3.1 – 6.0	≤1.0	1.3 – 2.4	DS–4	>3.5	—	>5.5	—	AC–3s	DC–3
						—	>5.5	—	>6.5	AC–4	DC–4
						2.5 – 3.5	—	2.5 – 5.5	—	AC–4s	DC–4
						—	2.5 – 5.5	—	2.5 – 6.5	AC–5	DC–4[f]
3.1 – 6.0	≤1.2[c]	3.1 – 6.0	≤1.0[d]	1.3 – 2.4	DS–4m	>3.5	—	>5.5	—	AC–3s	DC–3
						—	>5.5	—	>6.5	AC–4m	DC–4m
						2.5 – 3.5	—	2.5 – 5.5	—	AC–4ms	DC–4m
						—	2.5 – 5.5	—	2.5 – 6.5	AC–5m	DC–4m[f]
>6.0	≤1.2	>6.0	1.0	>2.4	DS–5	>3.5	—	>5.5	—	AC–4s	DC–4
						2.5 – 3.5	≥2.5	2.5 – 5.5	≥2.5	AC–5	DC–4[f]
>6.0	≤1.2[d]	>6.0	1.0[c]	>2.4	DS–5m	Not found in UK natural ground		>5.5	—	AC–4ms	DC–4m
								2.5 – 5.5	≥2.5	AC–5m	DC–4m[f]

Key

a Where the hydrostatic head of groundwater is greater than five times the section width, refer to BS 8500. [18]

b Brownfield sites are those that might contain chemical residues remaining from previous industrial use or from imported wastes

c Applies only to sites where concrete will be exposed to sulfate ions (SO_4), which can result from the oxidation of sulfides such as pyrite, following ground disturbance

d The limit on water-soluble magnesium does not apply to brackish groundwater (chloride content between 12 g/l and 18 g/l). This allows these sites to be classified in the row above

e An additional account is taken of hydrochloric and nitric acids by adjustment to sulfate content (see BRE Special Digest 1[19], Part 1)

f Where practicable, this should include APM3 (surface protection) as one of the APMs; refer to BS 8500

DC-class	Appropriate designated concrete
DC-2	FND2
DC-2z	FND2Z
DC-3	FND3
DC-3z	FND3Z
DC-4	FND4
DC-4z	FND4Z
Note Strength class for all FND concrete is C25/30.	

Table 3.4
Guidance on selected designated concrete for reinforced concrete foundations

3.3 Assessing the design moments

At detailed design stage most engineers are used to analysing the whole building structure or breaking concrete structures into sub-frames. These approaches are not suitable for the examination, even though the use of laptop computers is now permitted. So, how can a continuous concrete beam or slab be analysed? The following techniques may be adopted.

1. Firstly, wherever possible the coefficients for design moments and/or shear forces from the tables contained in BS 8110 should be used.
2. Only the critical sections of the element should be checked, i.e. for a continuous beam it is obvious from table 3.5 of BS 8110 that the critical location for bending is the hogging moment at the first interior support. So, for bending, this is the only location where the moment has to be assessed. Therefore the candidate does not need to carry out a sub-frame analysis; the design moment for an element can be assessed from a simple formula, for example, $M = -0.11Fl$.
3. Where the structure falls outside the scope of the tables (i.e. has less than three spans, cantilevers or spans differing in length by more than 15%) the candidate will need to make a quick approximate assessment of the forces. An element can be assumed to be continuous with no contribution from the columns. The charts in Appendix C may also be useful. Typically the maximum moments in a continuous beam are the hogging moments at the supports under full ultimate load. Remember, in the time available, the candidates are expected to calculate reasonable design forces only.
4. If you are very short of time, an approximate moment of $WL/10$ for uniformly distributed loads and $WL/5$ for point loads may be used for continuous beams.

Some engineers would advocate that assessing moments using method 4 above is all that is necessary at a preliminary stage. The key point is that candidates must demonstrate to the examiner that they understand what forces are acting on the element and can make a reasonable assessment of their magnitude.

There are some questions that require a sway-frame for stability; in these situations Figure C.2 in Appendix C may be used to assist in determining the design moments.

3.4 Flexure

The quantity of bending reinforcement required can be determined either from charts (as given in part 3 of the Code) or from the design formula given in the Code, which are set out below:

$K' = 0.156$ where redistribution of moments does not exceed 10%
$K' = 0.402\,(\beta_b - 0.4) - 0.18(\beta_b - 0.04)^2$ where redistribution of moment exceeds 10%
$K = M/bd^2f_{cu}$

If K ≤ K′, compression reinforcement is not required and the lever arm, z, can be calculated from:

$$z = d\left\{0.5 + \sqrt{0.25 - \frac{K}{0.9}}\right\}$$ but not greater than 0.95d i.e. if $K < 0.04275$

The area of reinforcement required can then be calculated using the following expression:

$A_s = M/0.87 f_y z$

For a preliminary design, compression reinforcement should be avoided.

It may be useful to write programs for a suitable calculator to carry out these calculations. The program can be written to return a value for K, which is useful for quickly making an assessment of the efficiency of the design. Useful values for K are given in Table 3.5.

Table 3.5
Useful values for *K*

Comment	Concrete cube strength, f_{cu}				
	30	35	40	45	50
Minimum % of reinforcement (0.13%)[a]	0.020	0.017	0.015	0.013	0.012
Lever arm, $z = 0.95d$	0.043	0.043	0.043	0.043	0.043
Recommended limit of K for slabs[b]	0.067	0.060	0.050	0.044	0.040
K' for 30% redistribution	0.114	0.114	0.114	0.114	0.114
K' for 20% redistribution	0.149	0.149	0.149	0.149	0.149
K' for 10% or less redistribution	0.156	0.156	0.156	0.156	0.156

Key
a Based on $d/h = 0.9$
b Recommended limit to avoid excessive deflection

3.5 Shear

Where shear stress (v) is considered to be critical, it can be calculated as follows:

$$v = \frac{V}{bd}$$

For beams, v must be less than the smaller of $0.8\sqrt{f_{cu}}$ or 5 N/mm^2 (see also Table 3.6). Ideally it should be less than 2 N/mm^2 to avoid congestion, but this may not be possible for transfer beams where shear is critical.

Table 3.6
Limiting values of shear stress

Concrete cube strength, f_{cu} (N/mm^2)	Maximum shear stress (N/mm^2)
25	4.00
30	4.38
35	4.73
40 and above	5.00

For slabs v should be less than v_c to avoid shear reinforcement in the slab.

The design of shear links is carried out using table 3.8 of BS 8110 to determine v_c and tables 3.7 (beams) or 3.16 (slabs) to design the links. These tables are included in Appendix B for ease of reference. Appendix C contains some look-up tables for the values of v_c for concrete with characteristic compressive strengths of 30, 35 and 40 N/mm^2.

Remember, A_s is the area of longitudinal reinforcement that continues for a distance of d past the section being considered, and that the spacing of the link should not exceed 0.75d.

3.6 Deflection

Where deflection is considered important it should be checked using tables 3.9 to 3.11 of BS 8110 (see Appendix B). Where the span exceeds 10 m the values from table 3.9 should be multiplied by 10/span.

3.7 Estimating reinforcement quantities

The following methods are available to estimate the quantity of reinforcement:

- Use the values given in *Economic concrete frame elements*[2].
- Use Method 2 given in the *Manual for design of reinforced concrete structures*[20], where formulae are provided to assess the quantity of reinforcement. This method is probably too time-consuming to use in the examination.
- Use Method 3 given in the *Manual for design of reinforced concrete structures*, where all the reinforcement in the element is determined and then the total weight is calculated. This method is definitely too time-consuming in the examination.
- Use experience to estimate the weight of the reinforcement per cubic metre of concrete. Consultants usually keep records for this purpose and a typical range of reinforcement rates for various elements is given in Table 3.7. The requirements for principal elements should be given separately as they are not likely to be 'typical'.

Remember that, for the cost of the project to be established, an indication of the bar sizes is required in addition to their total weight.

Table 3.7
Typical reinforcement rates (kg/m^3)

Element	Low	High
Slabs, one-way	75	110
Slabs, two-way	65	110
Flat slabs	75	220
Ribbed slabs	70	140
RC pad footings	70	90
Pile caps	110	150
Rafts	60	115
Columns	100	450
Ground beams	225	330
Beams	90	330
Retaining walls	90	130
Stairs	100	150
Walls	40	100

Note
The actual reinforcement quantity in the element will vary according to detailing practice and efficiency of the concrete element.

3.8 Detailing

3.8.1 Maximum and minimum areas of reinforcement

The maximum area of either the tension or compression reinforcement in a horizontal element is 4% of the gross cross-sectional area of the concrete. In an in-situ column the maximum reinforcement is 6% or 10% at laps. The minimum percentages are given in Table B7 (see Appendix B).

3.8.2 Minimum spacing of bars

The minimum spacing of the bars is the maximum size of the coarse aggregate plus 5 mm or the bar size, whichever is the greater. For 20 mm aggregate and bars of 25 mm in diameter and over, the maximum number of bars in a layer is:

$$\text{No. of bars} \approx \frac{b_w - 2c - 2\phi_l}{2\phi_b}$$

where

c = cover
ϕ_l = link diameter
ϕ_b = bar diameter

This expression allows for the radius of the link displacing the outermost longitudinal bars towards the centre of the beam.

Table 3.8 gives the maximum number of bars for a variety of beam sizes and covers.

Table 3.8
Maximum number of bars per layer in a beam

Beam width, b_w (mm)	Bar diameter, ϕ_b (mm)								
	25 mm cover			30 mm cover			35 mm cover		
	25	32	40	25	32	40	25	32	40
300	4	3	2	4	3	2	3	3	2
350	5	4	3	5	4	3	4	3	3
400	6	4	3	6	5	4	5	4	3
450	7	5	4	7	5	4	6	5	4
500	8	6	5	8	6	5	7	6	4
550	9	7	5	9	7	5	8	6	5
600	10	8	6	10	8	6	9	7	6
650	11	8	7	11	8	7	10	8	6
700	12	9	7	12	9	7	11	9	7
750	13	10	8	13	10	8	12	10	8
800	14	11	8	14	11	9	13	10	8
900	16	12	10	16	12	10	15	12	9
1000	18	14	11	18	14	11	17	13	11
1100	20	15	12	20	16	12	19	15	12
1200	22	17	13	22	17	14	21	17	13
1300	24	19	15	24	19	15	23	18	14
1400	26	20	16	26	20	16	25	20	16
1500	28	22	17	28	22	17	27	21	17

Note
These values are suitable for a link diameter of up to 16 mm.

3.8.3 Maximum spacing of bars

The maximum spacing is given in table 3.28 of BS 8110 (see Appendix B).

Shear links should be at a spacing of no more than 0.75d, and no longitudinal bar should be more than 150 mm or d from a vertical leg.

3.9 Design of beams

3.9.1 Governing criteria

Usually the governing criterion is the bending strength. Deflection may become critical for long spans or cantilevers, and shear is likely to be critical for transfer beams, especially for heavily loaded short spans.

3.9.2 Analysis

Wherever possible, use the coefficients presented in Table 3.9, which are appropriate provided the following conditions are met:

- Q_k must not exceed G_k
- Loads must be uniformly distributed
- Variations in the span length must not exceed 15% of the longest
- Redistribution of 20% is included in the figures (therefore $K' = 0.149$)

Table 3.9
Design ultimate bending moments and shear forces for beams

	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0	$0.09Fl$	$-0.11Fl$	$0.07Fl$	$-0.08Fl$
Shear	$0.45F$	—	$0.6F$	—	$0.55F$

Notes
1 l is the effective span; F is the total design ultimate load $(1.4G_k + 1.6Q_k)$.
2 No redistribution of the moments calculated from this table should be made.

Further guidance on determining bending moments can be found in Section 3.4.

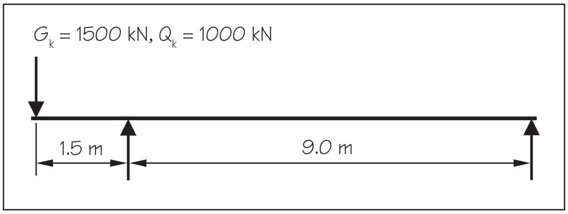
3.9.3 Flanged beams

A flanged beam may be treated as a rectangular beam, of full width, b, when the neutral axis is within the flange. In this case the moment of resistance in compression of the section is:

$$M_R = 0.45 f_{cu} b h_f (d - h_f/2)$$

When the applied moment is greater than the moment of resistance of the flange (M_R) the neutral axis lies in the web, and the beam cannot be designed as a rectangular beam as discussed above. In this case, reference should be made to BS 8110.

The Concrete Centre™	Project details	Calculated by	Job no.
	Worked example 1 Transfer beam	OB	CCIP - 018
		Checked by: JB	Sheet no.: TB1
		Client: TCC	Date: Dec 06



Initial sizing

Shear stress not to exceed 4 N/mm^2
(to avoid reinforcement congestion).

Ultimate load = 1.4 x 1500 + 1.6 x 1000 = 3700 kN
(ignoring self-weight)

Take b = 600

$$\therefore d = \frac{V}{vb} = \frac{3700 \times 10^3}{4 \times 600} = 1542 \text{ mm}$$

Eqn 3, BS 8110

Take overall depth as 1650 mm (d = 1550)

Bending

M_{max} = 3700 x 1.5 = 5550 kNm

For M_{max} = 5550, b = 600, d = 1550, f_{cu} = 40 N/mm^2

K = 0.096, A_s = 9370 mm^2

Use $\boxed{\text{8 H40 (10100 mm}^2\text{) in 2 layers.}}$

Shear

$$\frac{100A_s}{b_v d} = \frac{100 \times 10100}{600 \times 1550} = 1.09$$

Table 3.8, BS 8110

$\therefore v_c$ = 0.75 N/mm^2

Table 3.7, BS 8110

$$\frac{A_{sv}}{s_v} \geq \frac{b_v (v - v_c)}{0.87 f_{yv}}$$

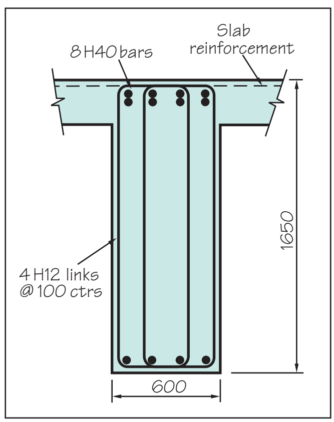
$$\geq \frac{600\ (4.0 - 0.75)}{0.87 \times 500}$$

$\geq$ 4.48 mm

Try H12 links

s_v = 452/4.48 = 101 mm

Say $\boxed{\text{4 H12 links @100 mm ctrs}}$



Comments

Remember to check headroom beneath the beam

H40 bars will be heavy; if there is no reasonable alternative, ensure that the contractor is aware so he may take steps to safeguard the health and safety of the steel fixers.

3.10 One-way spanning slabs

3.10.1 Governing criteria

Bending strength and deflection are usually the governing criteria. The end span condition should be checked because the moments are larger in this span unless there is a cantilever or the span is shorter than the interior spans.

3.10.2 Analysis

Wherever possible use the coefficients presented in Table 3.10, which are appropriate provided the following conditions are met (note that 20% redistribution is included in the coefficients):

1. The area of the slab exceeds 30 m^2 (e.g. 5 m x 6 m).
2. The ratio of characteristic imposed load to characteristic dead load does not exceed 1.25.
3. The characteristic imposed load does not exceed 5 kN/m^2 excluding partitions.
4. The spans are approximately equal. (This is generally assumed to mean that variations in the span lengh must not exceed 15% of the longest, but is not specified in the Code).
5. Redistribution of 20% is included in the figures (therefore K' = 0.149).

The requirements of conditions 1 and 2 will usually be met with most building designs.

Table 3.10
Design ultimate bending moments and shear forces for slabs

	End support/slab connections				At first interior support	At middle of interior spans	At interior supports
	Simply supported		Continuous				
	At outer support	Near middle of end span	At outer support	Near middle of end span			
Moment	0	0.086Fl	– 0.04FL	0.075Fl	– 0.086Fl	0.063Fl	– 0.063Fl
Shear	0.40F	—	0.46F	—	0.60F	—	0.50F

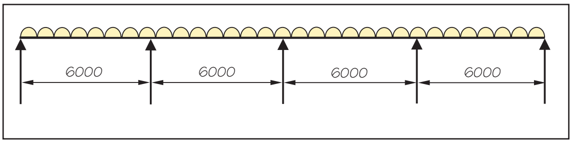
Notes
1 l is the effective span; F is the total design ultimate load (1.4G_k+ 1.6Q_k).
2 No redistribution of the moments calculated from this table should be made.

3.10.3 Detailing

General rules for spacing are given in Section 3.8.

The maximum spacing is given in Cl. 3.12.11.2.7. However, for initial sizing, table 3.28 of BS 8110 (see Appendix B) can be used conservatively.

The Concrete Centre™	Project details	Calculated by	Job no.
	Worked example 2 One-way slab	OB	CCIP - 018
		Checked by: JB	Sheet no.: OW1
		Client: TCC	Date: Dec 06



Imposed load = 2.5 kN/m^2
Superimposed dead load = 1.5 kN/m^2
Concrete class C28/35
Cover = 25 mm

Initial sizing

From *Economic concrete frame elements* – 178, say 200 mm — Section 2.10
or 6000/32 = 187.5, say 200 mm

Loading

ULS = 1.4 (0.2 x 24 + 1.5) + 1.6 x 2.5 = 12.8 kN/m^2

Bending

Check first support from end
$M = -0.086Fl$ — Table 3.12, BS 8110
$= -0.086 \times 12.8 \times 6^2$
$= -39.6$ kNm

For $b = 1000$, $d = 200 - 25 - 10 = 165$, $f_{cu} = 35$
Then $K = 0.041$, $A_{s,\,req} = 581$ mm^2
Use $\boxed{\text{H12 @ 175 ctrs}}$ ($A_{s,prov} = 646$ mm^2)

Shear

$V = 0.6F = 0.6 \times 12.8 \times 6 = 46.1$ kN — Table 3.12, BS 8110

$$v = \frac{V}{bd} = \frac{46.1 \times 10^3}{1000 \times 165} = 0.28 \text{ N/mm}^2$$

$$\frac{100\,A_s}{bd} = \frac{100 \times 646}{1000 \times 165} = 0.39$$

$\therefore v_c = 0.64$ — Table 3.8, BS 8110
$v < v_c$ $\therefore$ no shear links required

Deflection

Maximum sagging moment $= 0.075\,Fl$ — Table 3.12, BS 8110
$= 0.075 \times 12.8 \times 6^2 = 34.56$ kNm

For $f_{cu} = 35$, $d = 165$, $b = 1000$
Then $K = 0.036$, $A_{s,req} = 507$ mm^2

Use H12s @ 200 ctrs ($A_s = 566$ mm^2)
Span/depth = 6000/165 = 36.4

$$f_s = \frac{2\,f_y A_{s\,req}}{3A_{s,\,prov}} = \frac{2 \times 500 \times 500}{3 \times 566} = 294 \text{ N/mm}^2$$ — Table 3.10, BS 8110

$M/(bd^2) = 0.036 \times 35 = 1.26$ — Table C.7

$\therefore MF = 1.25$
Allowable $l/d = 1.25 \times 26 = 32.5 < 36.4$
$\therefore$ use $\boxed{\text{H12s @ 175 ctrs}}$ $A_{s,prov} = 646$ mm^2
$f_s = 257$ $MF = 1.40$
Allow $l/d = 1.40 \times 26 = 36.4 \geq 36.4$ OK

3.11 Two-way spanning slabs

3.11.1 Governing criteria

Bending strength and deflection are usually the governing criteria. The corner panel should be checked because the moments are larger in this panel, assuming a regular grid.

3.11.2 Analysis

Definitions

l_x = length of shorter side
l_y = length of longer side
m_{sx} = maximum design ultimate moments of unit width and span l_x
m_{sy} = maximum design ultimate moments of unit width and span l_y
n = total design ultimate load per unit area $(1.4G_k + 1.6Q_k)$

Simply supported slabs

Simply supported slabs (unrestrained) that do not have adequate provision to resist torsion at the corners or to prevent the corners from lifting, can be designed using the coefficients from Table 3.11. These coefficients are suitable only where the slab is not cast monolithically with the supporting beams.

The maximum moments per unit width are given by the following equations:

$$m_{sx} = \alpha_{sx}\, n l_x^{\,2}$$
$$m_{sy} = \alpha_{sy}\, n l_x^{\,2}$$

Table 3.11
Bending moment coefficients for slabs spanning in two directions at right angles, simply supported on four sides

l_y/l_x	1	1.1	1.2	1.3	1.4	1.5	1.75	2
α_{sx}	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118
α_{sy}	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029

Restrained slabs

The maximum bending moments per unit width for a slab restrained at each corner are given by the following equations:

$$m_{sx} = \beta_{sx}\, n l_x^{\,2}$$
$$m_{sy} = \beta_{sy}\, n l_x^{\,2}$$

Tables 3.12 and 3.13 may be used to determine the bending moments and shear force, provided the following rules are adhered to:

- The characteristic dead and imposed loads on adjacent panels are approximately the same as on the panel being considered.
- The span of adjacent panels in the direction perpendicular to the line of the common support is approximately the same as the span of the panel considered in that direction.

The rules to be observed when the equations are applied to restrained slabs (continuous or discontinuous) are as follows.

1. Slabs are considered as being divided in each direction into middle strips and edge strips as shown in figure 3.9 of BS 8110, the middle strip being three-quarters of the width and each edge strip one-eighth of the width.

2 The maximum design moments calculated as above apply only to the middle strips and no redistribution should be made.

3 Reinforcement in the middle strips should be detailed in accordance with Cl. 3.12.10 of BS 8110 (simplified rules for curtailment of bars).

4 Reinforcement in an edge strip, parallel to the edge, need not exceed the minimum given in Cl. 3.12.5 of BS 8110 (minimum areas of tension reinforcement), together with the recommendations for torsion given in points 5, 6 and 7 below.

5 Torsion reinforcement should be provided at any corner where the slab is simply supported on both edges meeting at that corner. It should consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers should be three-quarters of the area required for the maximum mid-span design moment in the slab.

6 Torsion reinforcement equal to half that described in the preceding paragraph should be provided at a panel corner contained by edges over only one of which the slab is continuous.

7 Torsion reinforcement need not be provided at any panel corner contained by edges over both of which the slab is continuous.

Table 3.12
Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corners (from table 3.14 of BS 8110)

Type of panel and moments considered	Short span coefficients, β_{sx} Values of l_y/l_x								Long span coefficients, β_{sy} for all values of l_y/l_x
	1	1.1	1.2	1.3	1.4	1.5	1.75	2	
Interior panels									
Negative moment at continuous edge	0.031	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0.032
Positive moment at mid-span	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
One short edge discontinuous									
Negative moment at continuous edge	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.037
Positive moment at mid-span	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0.050	0.028
One long edge discontinuous									
Negative moment at continuous edge	0.039	0.049	0.056	0.062	0.068	0.073	0.082	0.089	0.037
Positive moment at mid-span	0.030	0.036	0.042	0.047	0.051	0.055	0.062	0.067	0.028
Two adjacent edges discontinuous									
Negative moment at continuous edge	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093	0.045
Positive moment at mid-span	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070	0.034
Two short edges discontinuous									
Negative moment at continuous edge	0.046	0.050	0.054	0.057	0.060	0.062	0.067	0.070	—
Positive moment at mid-span	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
Two long edges discontinuous									
Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.045
Positive moment at mid-span	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
Three edges discontinuous (one long edge continuous)									
Negative moment at continuous edge	0.057	0.065	0.071	0.076	0.081	0.084	0.092	0.098	—
Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.063	0.069	0.074	0.044
Three edges discontinuous (one short edge continuous)									
Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.058
Positive moment at mid-span	0.042	0.054	0.063	0.071	0.078	0.084	0.096	0.105	0.044
Four edges discontinuous									
Positive moment at mid-span	0.055	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

The maximum shear force per unit width is given by the following equations:

$v_{sx} = \beta_{vx} n l_x$

$v_{sy} = \beta_{vy} n l_x$

The coefficients β_{vx} and β_{vy} are obtained from Table 3.13.

3.11.3 Detailing

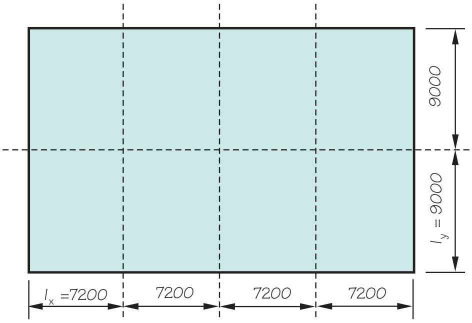
General rules for spacing are given in Section 3.8.

The maximum spacing is given in Cl. 3.12.11.2.7. However, for initial sizing, table 3.28 of BS 8110 (see Appendix B) can be used conservatively.

Table 3.13
Shear force coefficients for rectangular panels supported on four sides with provision for torsion at corners (from table 3.15 of BS 8110)

Type of panel and location	β_{vx} for values of l_y/l_x								β_{vy}
	1	1.1	1.2	1.3	1.4	1.5	1.75	2	
Four edges continuous									
Continuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.5	0.33
One short edge discontinuous									
Continuous edge	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
Discontinuous edge	—	—	—	—	—	—	—	—	0.24
One long edge discontinuous									
Continuous edge	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
Discontinuous edge	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	—
Two adjacent edges discontinuous									
Continuous edge	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
Discontinuous edge	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
Two short edges discontinuous									
Continuous edge	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	—
Discontinuous edge	—	—	—	—	—	—	—	—	0.26
Two long edges discontinuous									
Continuous edge	—	—	—	—	—	—	—	—	0.40
Discontinuous edge	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	—
Three edges discontinuous (one long edge discontinuous)									
Continuous edge	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	—
Discontinuous edge	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.29
Three edges discontinuous (one short edge discontinuous)									
Continuous edge	—	—	—	—	—	—	—	—	0.45
Discontinuous edge	0.29	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
Four edges discontinuous									
Discontinuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

The Concrete Centre™	Project details Worked example 3 Two-way slab	Calculated by OB	Job no. CCIP - 018
		Checked by JB	Sheet no. TW1
		Client TCC	Date Dec 06



Superimposed dead load = 1.5 kN/m^2

Imposed load = 5 kN/m^2

Cover = 25 mm

Concrete class C28/35

Initial sizing	From *Economic concrete frame elements:* 210 mm or 9000/36 = 250 mm say 250 mm	Section 2.10
Loads	$n = 1.4\ (1.5 + 6) + 1.6 \times 5 = 18.5$ kN/m^2 (ULS)	
Check short span	$d = 250 - 25 - 8 = 217$ mm $\frac{l_y}{l_x} = \frac{9.0}{7.2} = 1.3$ $\beta_{sx} = -0.069$ and $\beta_{sx} = 0.051$	Table 3.14, BS 8110
Support moment critical in bending	$m_{sx} = \beta_{sx}\ n\ l_x^2 = -0.069 \times 18.5 \times 7.2^2 = -66.2$ kNm/m For $m_{sx} = 66.2$, $b = 1000$, $d = 217$, $f_{cu} = 35$ Then $A_s = 738$ mm^2/m ($K = 0.040$) Use $\boxed{\text{H12s @ 150 ctrs}}$ ($A_{s,prov} = 754$ mm^2/m)	
Shear	$\beta_{vx} = 0.50$ $V_{sx} = \beta_{vx}\ n\ l_x$ $= 0.5 \times 18.5 \times 7.2$ $= 66.6$ kN/m width $v = \frac{V}{bd} = \frac{66.6 \times 10^3}{1000 \times 217} = 0.31$ N/mm^2 $\frac{100\ A_s}{bd} = \frac{100 \times 754}{1000 \times 217} = 0.35$	Table 3.15, BS 8110
	$\therefore v_c = 0.58$ N/mm^2 > 0.31 ∴ no shear links required	Table 3.8, BS 8110
Deflection	Maximum sagging moment = $0.051 \times 18.5 \times 7.2^2 = 48.9$ kN/m For $b = 1000$, $d = 217$, $f_{cu} = 35$ Then $K = 0.030$, $A_{s\ req} = 543$ mm^2 Try $A_{s,prov} = 646$ mm^2 Actual span/depth = 7200/217 = 33.2	
	$f_s = \frac{2 f_y\ A_{s,req}}{3\ A_{s,prov}} = \frac{2 \times 500 \times 543}{3 \times 646} = 280$ N/mm^2, $\frac{M}{bd^2} = 1.04$	Table 3.7, BS 8110
	∴ MF = 1.39	Table C.7
	Allowable span/depth = 1.39 x 26 = 36.2 > 33.2 OK Use $\boxed{\text{H12s @ 175}}$ ($A_{s,prov} = 646$ mm^2)	

3.12 Flat slabs

3.12.1 Governing criteria

Bending strength, punching shear and deflection can be the governing criteria for flat slabs. The end span condition should be checked because the moments are larger in this span unless there is a cantilever or the span is shorter than the interior spans.

3.12.2 Analysis

Wherever possible use the coefficients presented in Table 3.14. These are appropriate provided the following conditions are met (note that 20% redistribution is included in the coefficients):

- The area of the slab exceeds 30 m^2 (i.e. 5 m x 6 m).
- The ratio of characteristic imposed load to characteristic dead load does not exceed 1.25.
- The characteristic imposed load does not exceed 5 kN/m^2 excluding partitions.
- The spans are approximately equal (generally assumed to be 15% of the longest span, but not specified in the Code).
- Redistribution of 20% is included in the figures (therefore K' = 0.149).

Table 3.14
Design ultimate bending moments and shear forces for slabs

	End support/slab connections				At first interior support	At middle of interior spans	At interior supports
	Simply supported		Continuous				
	At outer support	Near middle of end span	At outer support	Near middle of end span			
Moment	0	0.086*Fl*	– 0.04*FL*	0.075*Fl*	– 0.086*Fl*	0.063*Fl*	– 0.063*Fl*
Shear	0.40*F*	—	0.46*F*	—	0.60*F*	—	0.50*F*

Notes
1 l is the effective span; F is the total design ultimate load ($1.4G_k + 1.6Q_k$).
2 No redistribution of the moments calculated from this table should be made.

Where these criteria are not met the moments in the slab will have to be assessed using the tables and charts from Appendix C. (Refer to Section 3.3 for further information.)

When the moments have been determined, the floor plate should to be divided into notional column strips and middle strips (see Figure 3.2) and the total moment across the panel width should be apportioned to the column and middle strips in accordance with Table 3.15. Two-thirds of the reinforcement in the middle strip should be placed in half the column width centred over the column.

The critical areas to check in a flat slab are bending strength (usually in hogging over the support), deflection and punching shear. The design should demonstrate that the flat slab is suitable for moments and deflection in two orthogonal directions.

Table 3.15
Distribution of design moments in panels of flat slabs

Design moment expressed as percentages of the total negative or positive design moment	Apportionment between column and middle strip	
	Column strip, %	Middle strip, %
Negative	75	25
Positive	55	45

Note
For the case where the width of the column strip is taken as equal to that of the drop, and the middle strip is thereby increased in width, the design moments to be resisted by the middle strip should be increased in proportion to its increased width. The design moments to be resisted by the column strip may be decreased by an amount such that the total positive and the total negative design moments resisted by the column strip and middle strip together are unchanged.

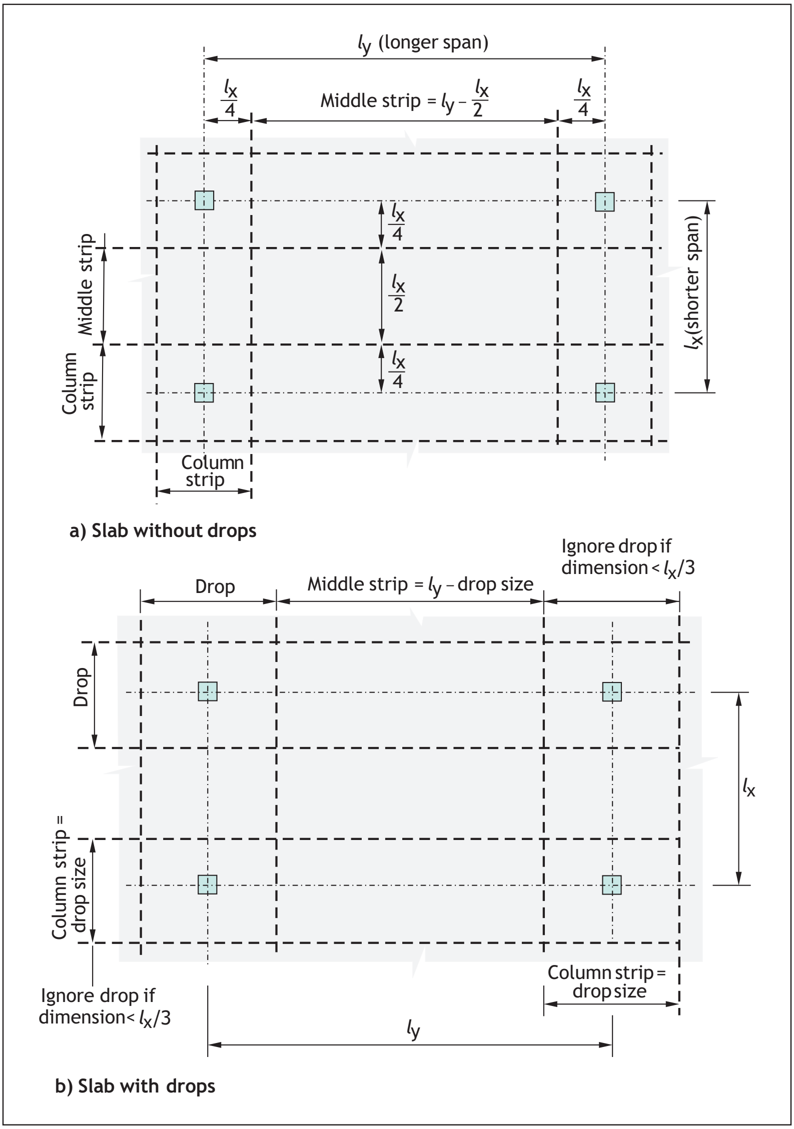


Figure 3.2
Division of panels in flat slabs

3.12.3 Punching shear reinforcement

The maximum design shear stress at the face of the column (v_{max}) can be calculated as follows:

$$v_{max} = \frac{V_{eff}}{u_o d}$$

where

V_{eff} = design effective shear force, which for initial design can be calculated from Figure 3.3.

u_o = length of the perimeter of the column

The design shear stress at a particular perimeter (v) can be calculated as follows:

$$v = \frac{V_{eff}}{u\,d}$$

where

V_{eff} = design effective shear force as calculated from Figure 3.3
u = length of the perimeter

The length of the perimeter can be determined from Figure 3.4. The Code provides guidance on designing shear reinforcement where the shear stress (v) is less than $2v_c$. Therefore, for initial design it advised that $v < 2v_c$ (and ideally $v < 1.6v_c$ to avoid excessive reinforcement). v_c can be determined from table 3.8 of BS 8110. Appendix C contains some look-up tables for the values of v_c for concrete with characteristic compressive strengths of 30, 35 and 40 N/mm^2.

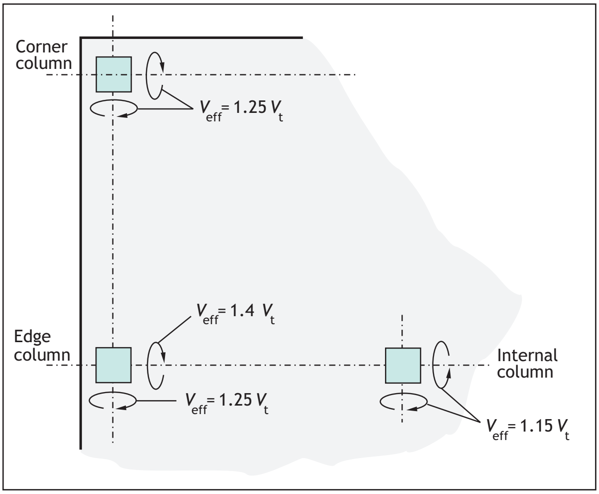


Figure 3.3
Determining effective shear force

3.12.4 Transfer moments

The maximum design moment $M_{t\ max}$, which can be transferred to a column, is given by:

$$M_{t\ max} = 0.15\ b_e\ d^2\ f_{cu}$$

where

b_e = breadth of effective moment transfer strip (see Figure 3.5)
d = effective depth for the top reinforcement in the column strip

$M_{t\ max}$ should be not less than half the design moment obtained from an equivalent frame analysis or 70% of the design moment if a grillage or finite element analysis has been used. If $M_{t\ max}$ is calculated to be less than this, the structural arrangements should be changed.

3.12.5 Deflection

Deflection should be checked using tables 3.9 and 3.10 of BS 8110 (see Appendix B), which are appropriate for slabs spanning up to 10 m. For flat slabs a factor of 0.9 should be applied to the allowable span-to-effective-depth ratio.

3.12.6 Detailing

General rules for spacing are given in Section 3.8.

The maximum spacing is given in Cl. 3.12.11.2.7. However, for initial sizing, table 3.28 of BS 8110 (see Appendix B) can be used conservatively.

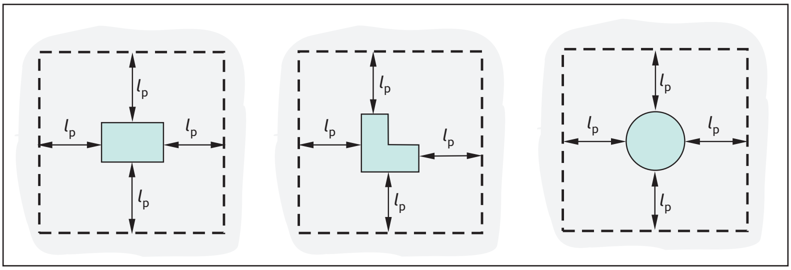


Figure 3.4
Definition of a shear perimeter for typical cases

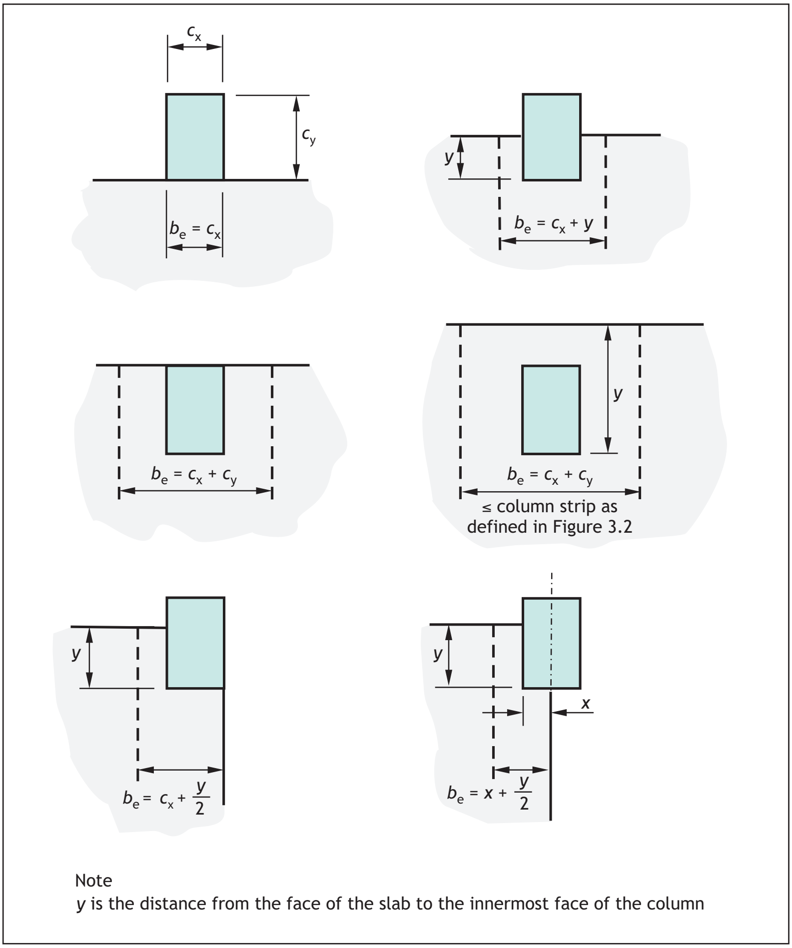
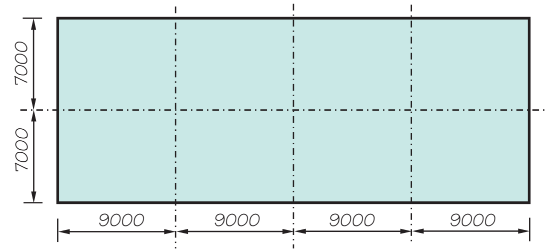


Figure 3.5
Definition of breadth of effective moment transfer strip b_e for various typical cases

The Concrete Centre™	Project details	Calculated by	Job no.
	Worked example 4 Flat slab	OB	CCIP - 018
		Checked by	Sheet no.
		JB	FS1
		Client	Date
		TCC	Dec 06



Imposed load = 5 kN/m^2

Superimposed dead load = 1.5 kN/m^2

Concrete class C28/35

Cover = 25 mm

Initial sizing	Using *Economic concrete frame elements*: 300 mm	
	Or 8000/26 = 307 mm, say 300 mm	Section 2.1
Loading	ULS = 1.4 (1.5 + 0.3 x 24) + 1.6 x 5 = 20.2 kN/m^2	
Bending	Check long span end bay condition	
	$M = -0.086Fl = -0.086 \times 20.2 \times 7 \times 9^2 = -985$ kNm	Table 3.12, BS 8110
Centre strip critical	Design moment = 0.75 x 985 = 739 kNm	Table 3.18, BS 8110
	For b = 3500, d = 300 – 25 –12.5 = 262 mm, f_{cu} = 35	
	Then K = 0.088, A_s = 7280 mm^2	
	Centre column strip = 4853 mm^2 (2773 mm^2/m)	
	Use $\boxed{\text{T20 @ 100 ctrs}}$ ($A_{s,prov}$ = 3140 mm^2/m)	
	Outer column strip = 2417 mm^2 (1389 mm^2/m)	
	Use $\boxed{\text{T20 @ 200 ctrs}}$ ($A_{s,prov}$ = 1570 mm^2/m)	
Punching shear	$V_t = 20.2 \times 7.0 \times 9.0 = 1273$ kN	
	$V_{eff} = 1.15\ V_t = 1.15 \times 1273 = 1464$ kN	
	Assume 350 sq columns	
	At column face $v = \dfrac{V_{eff}}{u_o\ d} = \dfrac{1464 \times 10^3}{4 \times 350 \times 250} = 4.2 < 4.73$	Eqn 27, BS 8110
	Shear resistance without links: $\dfrac{100A_s}{bd} = \dfrac{100 \times 3140}{1000 \times 250}$	Table 3.8, BS 8110
	$\therefore\ v_c = 0.85$ N/mm^2	
	First perimeter: Length = (350 + 2 (1.5 x 250)) x 4 = 4400	
	$v = \dfrac{1464 \times 10^3}{4400 \times 250} = 1.33 < 2v_c\ (1.7)$	Cl 3.7.7.5, BS 8110
	∴ Can be designed for punching shear.	
Deflection	Maximum sagging moment $= 0.075\ Fl$	
	$= 0.075 \times 20.2 \times 7 \times 9^2$	
	= 859 kNm	

Centre strip critical	Design moment $= 0.55 \times 859 = 472$ kNm	
	$M/(bd^2) = \dfrac{472 \times 10^6}{3500 \times 262^2} = 1.96$	
	For $b = 3500$, $d = 262$ and $f_{cu} = 35$	
	$K = 0.056$, $A_s = 4438$ mm^2 Or $A_s = 1268$ mm^2/m	
	Span/depth $= 9000/262 = 34.4$	
	Min $MF = \dfrac{34.4}{26 \times 0.9} = 1.47$	Table 3.10, BS 8110
	$\therefore$ max $f_s = 160$ N/mm^2	
	$f_s = \dfrac{2\, f_y A_{s,req}}{3\, A_{s,prov}}$	
	$\therefore A_{s,prov} = \dfrac{2\, f_y A_{s,req}}{3\, f_s} = \dfrac{2 \times 500 \times 1262}{3 \times 160} = 2629$ mm^2/m	
	Use $\boxed{\text{H25 @ 175 ctrs}}$ ($A_{s,prov} = 2810$ mm^2/m)	
Comments	1. Reinforcement area increased significantly for deflection, but the compression reinforcement has not been included. 2. The design in the orthogonal direction has not been included. 3. For the design of the strips across two panels only, coefficients from Table 3.14 are not appropriate.	

3.13 Ribbed slabs

3.13.1 Governing criteria

Bending strength and deflection are usually the governing criteria. The end span condition should be checked because the moments are larger in this span unless there is a cantilever or the span is shorter than the interior spans.

3.13.2 Geometry

BS 8110 gives geometrical limits for ribbed slabs (see Figure 3.6). The width of the rib is determined by cover, bar spacing and fire requirements. The absolute minimum is 125 mm (see Figure 3.1). Typically spacings for ribs are 600 mm, 750 mm, 900 mm and 1200 mm, with a rib width of 150 mm. Average self-weight of the ribbed slab can be determined from Figure 3.7, or an assumed self-weight can be obtained from Table 2.9c.

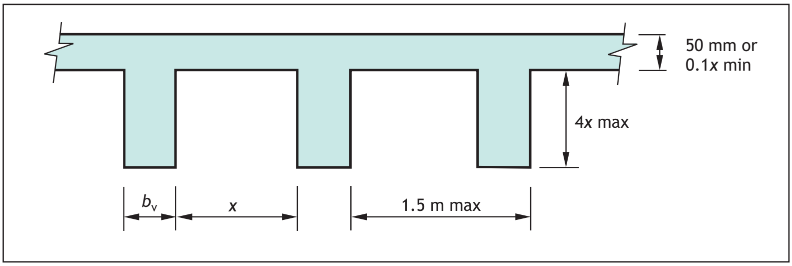


Figure 3.6
Geometrical limitations for ribbed slabs

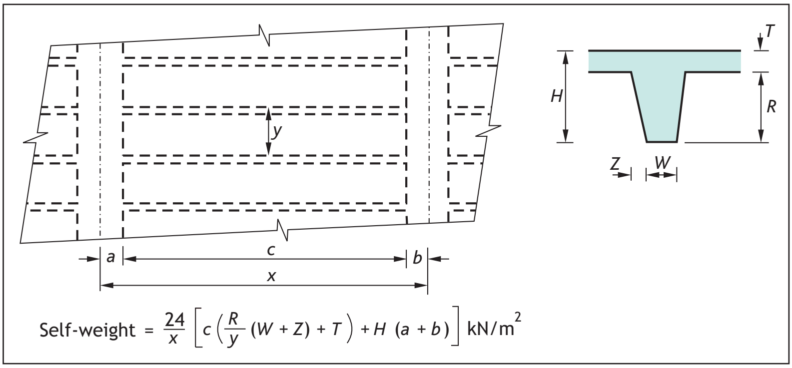


Figure 3.7
Calculation of ribbed slab self-weight

3.13.3 Analysis

A ribbed slab should be designed as a one-way slab with the rib and topping acting as a T-beam. Wherever possible use the coefficients presented in Table 3.14, which are appropriate provided the following conditions are met (note that 20% redistribution is included in the coefficients):

1. The area of the slab exceeds 30 m^2 (5 m x 6 m).
2. The ratio of characteristic imposed load to characteristic dead load does not exceed 1.25.
3. The characteristic imposed load does not exceed 5 kN/m^2 excluding partitions.
4. The spans are approximately equal (generally assumed to be 15% of the longest span, but not specified in the Code).

The requirements of conditions 1 and 2 will usually be met by most building designs.

3.13.4 Bending reinforcement

The quantity of bending reinforcement required is calculated in the way described for beams (see Section 3.4).

A flanged beam may be treated as a rectangular beam, of full width, *b*, when the neutral axis is within the flange. In this case the moment of resistance in compression of the section is:

$M_R = 0.45 f_{cu} b h_f\,(d - h_f/2)$ where h_f is the thickness of the topping slab

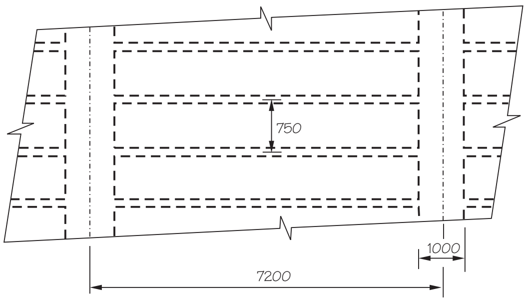
When the applied moment is greater than the moment of resistance of the flange (M_R) the neutral axis lies in the flange, and the beam cannot be designed as a rectangular beam and reference should be made to BS 8110.

The mid-span section is designed as a T-beam with flange width equal to the distance between ribs. The section at the support may need to be checked in two locations; firstly as a solid section at the location of the peak moment, secondly at the junction of the rib and the solid section as a rectangular beam of width equal to the rib.

3.13.5 Reinforcement in the topping

A fabric mesh reinforcement should be provided in the centre of the topping. The cross-sectional area of the reinforcement should be greater than 0.12% of the area of the topping (e.g. an A142 mesh is required for 100 mm thick topping). The wire spacing should not exceed half the spacing of the ribs. If the ribs are widely spaced (e.g. greater than 900 mm for a 100 mm thick topping or greater than 700 mm for a 75 mm thick topping) the topping should be designed for moment and shear as a one-way slab between ribs.

The Concrete Centre	Project details Worked example 5 Ribbed slab	Calculated by OB	Job no. CCIP - 018
		Checked by JB	Sheet no. RS1
		Client TCC	Date Dec 06



Imposed load = 2.5 kN/m^2

Superimposed load = 1.5 kN/m^2

Concrete class: C28/35

Cover = 20 mm

Topping thickness = 100 mm

Initial sizing — Using *Economic concrete frame elements*: 250 mm or 7200/29 = 248 mm, say 250 mm

Assume self-weight= 4.0 kN/m^2 — Table 2.9c

Loading — ULS = 1.4 (4.0 + 1.5) + 1.6 (2.5) = 11.7 kN/m^2

Bending — Check sagging moment in end bay

d = 250 – 20 – 6 – 10 = 214 mm

Effective length = 6.2 + 0.214^2 = 6.41 m — Cl 3.4.1.2, BS 8110

M = 0.075Fl = 0.075 x 11.7 x 6.41^2 x 0.75 = 27.0 kN/m per rib — Table 3.12, BS 8110

Check $M_A < M_R$

27.0 < $0.45 f_{cu} b_f h_f (d - h_f/2)$
< 0.45 x 35 x 750 x 100 (212 – 100/2) x 10^{-6}
< 194 kNm

∴ N.A. in flange – design as rectangular section

For b = 750, d = 214, f_{cu} = 35

K = 0.022, $A_{s,req}$ = 306 mm^2

Deflection — Span/depth = 7200/214 = 33.6

Min MF = $\dfrac{33.6}{20.8}$ = 1.62

M/(bd^2) = 27.0 x 10^6/(750 x 214^2) = 0.80

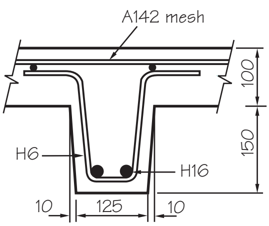
∴ Max f_s = 258 — Table 3.10

$A_{s,prov} = \dfrac{2 f_y A_{s,req}}{3 f_s}$

$= \dfrac{2 \times 500 \times 306}{3 \times 258}$

= 395 mm^2

Use $\boxed{\text{2 H16s per rib}}$ ($A_{s,prov}$ = 402 mm^2)



3.13.6 Detailing

Maximum and minimum areas of reinforcement

The maximum area of either the tension or compression reinforcement is 4% of the gross cross-sectional area of the concrete. The minimum percentages are given in table 3.25 of BS 8110 (see Appendix B).

Maximum and minimum spacing of bars

The minimum spacing of the bars is maximum size of the coarse aggregate plus 5 mm or the bar size, whichever is the greater.

The maximum spacing is given in Cl. 3.12.11.2.7. However, for initial sizing table 3.28 of BS 8110 (see Appendix B) can be used conservatively.

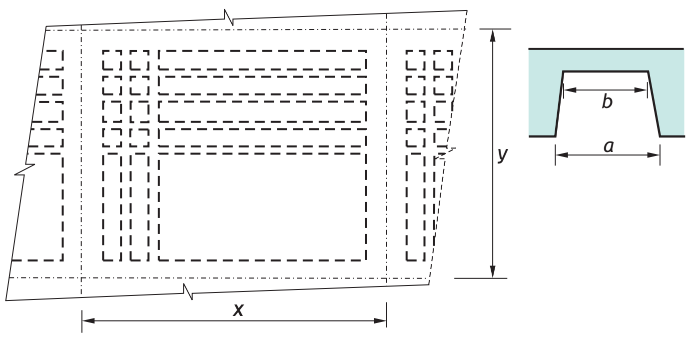
3.14 Waffle slabs

3.14.1 Governing criteria

Bending strength and deflection are usually the governing criteria. The end span condition should be checked because the moments are larger in this span unless there is a cantilever or the span is shorter than the interior spans.

3.14.2 Geometry

Average self-weight of the waffle slab can be determined from Figure 3.8, or can be assumed from Table 2.9c. Standard moulds are 225, 325 and 425 mm deep and are used with toppings between 50 and 150 mm thick. The ribs are 125 mm wide on a 900 mm grid.



$$\text{Self-weight} = \frac{24}{xy}\left[xyH - N\left(\frac{c}{3}\left(a^2 + b^2 + \sqrt{a^2 b^2}\right)\right)\right], \quad N = \text{number of waffles}$$

Figure 3.8
Calculation of waffle slab self-weight

3.14.3 Analysis

A waffle slab should normally be designed as a flat slab with the rib and topping acting as a T-beam. It may be designed as a two-way slab, but it should be noted that the torsional stiffness of a waffle slab is smaller than that of a solid two-way slab and the bending coefficients from Table 3.14 may not be appropriate.

3.14.4 Bending reinforcement

The quantity of bending reinforcement required is calculated in the same way as for beams (see Section 3.4).

A flanged beam may be treated as a rectangular beam, of full width, *b*, when the neutral axis is within the flange. In this case the moment of resistance in compression of the section is:

$$M_R = 0.45\, f_{cu} b h_f (d - h_f/2)$$

When the applied moment is greater than the moment of resistance of the flange (M_R) the neutral axis lies in the web, and the beam cannot be designed as a rectangular beam as discussed above. Where this does not apply, reference should be made to BS 8110.

The mid-span section is designed as a T-beam with flange width equal to the distance between ribs. The section at the support may need to be checked in two locations, firstly as a solid section at the location of the peak moment, secondly at the junction of the rib and the solid as a rectangular beam of width equal to the rib.

3.14.5 Shear reinforcement

Shear stress can be calculated as follows:

$$v = \frac{V}{b_w d}$$

and should be less than v_c (which can be obtained from table 3.8 of BS 8110, see Appendix B) for a practical and economic slab.

3.14.6 Reinforcement in the topping

A fabric mesh reinforcement should be provided in the centre of the topping, with the cross-sectional area greater than 0.12% of the area of the topping. The wire spacing should not exceed half the spacing of the ribs. If the ribs are widely spaced (e.g. greater than 900 mm for a 100 mm thick topping or greater than 700 mm for a 75 mm thick topping) the topping should be designed for moment and shear as a two-way slab between ribs.

3.14.7 Deflection

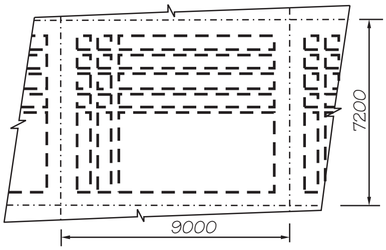
Deflection should be checked using tables 3.9 and 3.10 of BS 8110 (see Appendix B), which are appropriate for slabs spanning up to 10 m.

3.14.8 Detailing

General rules for spacing are given in Section 3.8.

The maximum spacing is given in Cl. 3.12.11.2.7. However, for initial sizing, table 3.28 of BS 8110 (see Appendix B) can be used conservatively.

The Concrete Centre™	Project details Worked example 6 Waffle slab	Calculated by OB	Job no. CCIP - 018
		Checked by JB	Sheet no. WS1
		Client TCC	Date Dec 06



Imposed load = 2.5 kN/m^2

Superimposed load = 1.5 kN/m^2

Concrete class: C28/35

Cover = 20 mm

Initial sizing	Using *Economic concrete frame elements*: 425 mm thick (i.e. 325 moulds + 100 topping)	
	or 9000/20 = 450, say 425 thick	Section 2.9
	Assume self-weight = 7.3 kN/m^2	Table 2.9c
Ultimate load	n = 1.4 (7.3 + 1.5) + 1.6 (2.5) = 16.3 kN/ m^2	
	There is a substantial beam along the column strips, which can therefore resist torsion at the corners. Design as a two-way spanning slab.	
	$\frac{l_y}{l_x} = \frac{9.0}{7.2} = 1.25$	
	Interpolating for a corner panel	
	Hogging moments $\beta_{sx} = -0.066$, Sagging moments $\beta_{sx} = 0.049$	Table 3.14 (BS 8110)
Bending at support	$M_{sx} = \beta_{sx}\, n l_x^2 = -0.066 \times 16.3 \times 7.2^2 = -55.8$ kN/m width	
	For ribs @ 900 ctrs	
	$M = 55.8 \times 0.9 = 50.2$ kNm/rib	
	For $d = 425 - 20 - 6 - 10 = 389$ mm, $b = 900$, $f_{cu} = 35$	
	Then $A_{s,req} = 310$ mm^2	
	But $A_{s,min} = 0.13\% \times 900 \times 425 = 497$ mm^2	
Deflection	$M_{sag} = 0.049 \times 16.3 \times 7.2^2 \times 0.9 = 37.3$ kNm/rib	
	$d = 425 - 20 - 6 - 10 = 389$ mm	
	check $M_{sag} < M_{flange} < 0.45 f_{cu} b_f h_f (d - h_f/2)$	
	$< 0.45 \times 35 \times 900 \times 100 (389 - 50) \times 10^{-6} < 481$ kNm	
	N.A. in flange ∴ rectangular section	
	For $b = 900$, $f_{cu} = 35$	
	Then $A_{s,req} = 232$ mm^2 ($K = 0.008$)	
	Span/depth = 7200/389 = 18.5	
	$\therefore MF = \frac{18.5}{20.8} = 0.89$	
	Maximum $f_s = 307$ ∴ no increase for deflection.	
	Use $\boxed{\text{2 x H16 per rib}}$ ($A_{s,prov} = 402$)	

3.15 Precast flooring systems

The design of precast concrete floor systems is usually undertaken by the manufacturer, and charts are provided by them for use by the building designer. In the examination all the candidate can realistically use to demonstrate the suitability of one of these systems is to refer to manufacturer's literature. Refer to Section 2.10.9 for suitable charts that have been compiled from the manufacturers' data.

There will be an interface between the precast flooring units and supporting structure. The following issues should be addressed in the design in addition to the strength of the flooring units:

- The minimum bearing should be 75 mm (although this can be reduced with special details).
- The units may need to be tied into the structure to meet robustness requirements (see Appendix A).
- If the floor is part of the stability system then either a structural screed will be required or the floor should be specifically designed to resist the horizontal loads. Further advice can be found in Cl. 5.3.7 of BS 8110.

3.16 Post-tensioning

The design of post-tensioned elements is a highly iterative process that in practice is carried out using specialist computer software. To enable a design to be carried out in the examination (taking at most 15 minutes) significant simplifying assumptions will be required. The design methods presented below are strictly for preliminary design only and assume some understanding of post-tensioning design and construction. For detailed design reference should be made to TR43 *Post-tensioned concrete floors* [16] or similar references. Properties of strand generally available are given in Table 3.16.

Table 3.16
Specification of commonly used strand in the UK

Strand type	Nominal tensile strength (MPa)	Nominal diameter (mm)	Cross-sectional area (mm^2)	Characteristic value of maximum force (kN)	Maximum value of maximum force (kN)	Characteristic value of 0.1% proof force (kN)
12.9 'Super'	1860	12.9	100	186	213	160
15.7 'Super'	1770	15.7	150	265	302	228
15.7 'Euro'	1860	15.7	150	279	319	240
15.2 'Drawn'	1820	15.2	165	300	342	258

3.16.1 Restraint

All concrete elements shrink due to drying and early thermal effects but, in addition, prestressing causes elastic shortening and ongoing shrinkage due to creep. Stiff vertical members such as stability walls restrain the floor slab from shrinking, which prevents the prestress from developing and thus reducing the strength of the floor. This should be considered in the design of the stability system or allowed for in the method of construction (see Section 2.7.13).

3.16.2 Load balancing

The traditional design method is 'load balancing' where the prestress on the concrete element is designed such that it imposes an upwards load on the element which counteracts the gravity loads on the element (see Figure 3.9). Usually only a part of the imposed loads are designed to be balanced by the prestress, say the dead loads only. The balancing forces on the element depend on the profile of the tendon, and are therefore chosen to match the shape of the bending moment diagram. By following the profile of the bending moment diagram the tendon will be located at the tension face for both the maximum hogging and sagging moments. The balancing forces can be calculated as follows:

UDL due to parabolic profile: $w = 8aP_{av} / s^2$

where

a = drape of tendon measured at centre of profile between points of inflection
P_{av} = average prestressing force in tendon
s = distance between points of inflection

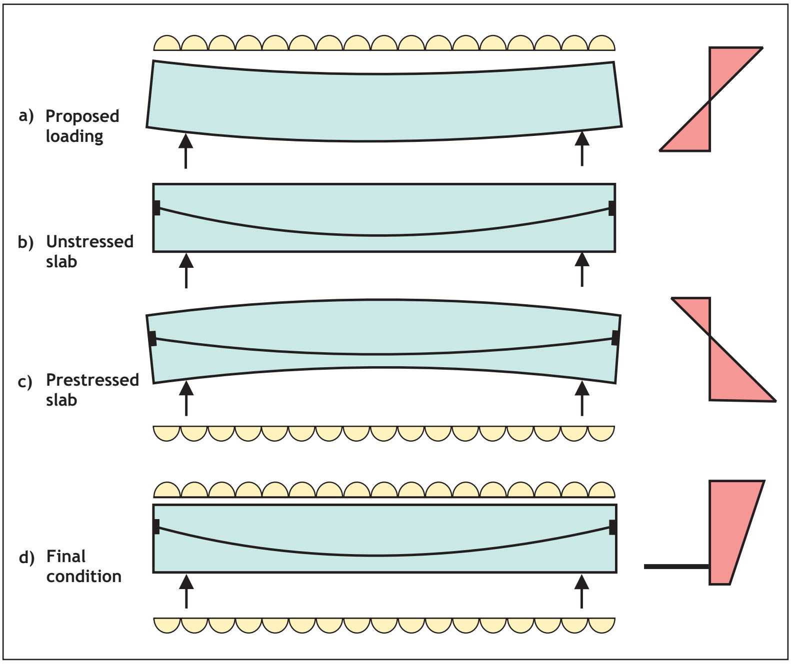


Figure 3.9
Load balancing method

3.16.3 Stresses

The stress in the element can be calculated as follows:

Stress at the top of the section, $\sigma_t = \dfrac{P}{A_c} + \dfrac{M_b}{Z_t}$

Stress at the bottom of the section, $\sigma_b = \dfrac{P}{A_c} + \dfrac{M_b}{Z_b}$

where

A_c = area of the concrete
M_a = balanced moment (i.e. including post-tensioning effects)
Z_t = section modulus - top
Z_b = section modulus - bottom
P = prestress force

3.16.4 Cover

Cover should be determined in the normal way i.e. for durability and fire resistance, but in addition the cover to the duct should be at least half the duct width. Cover is measured to the outside of a bonded duct. (Remember that the centre of tendon will be offset from the centre of the duct.) At supports it is usual to place the uppermost tendon in the T2 layer, with the tendon in the other direction below this (T3), see Figure 3.10.

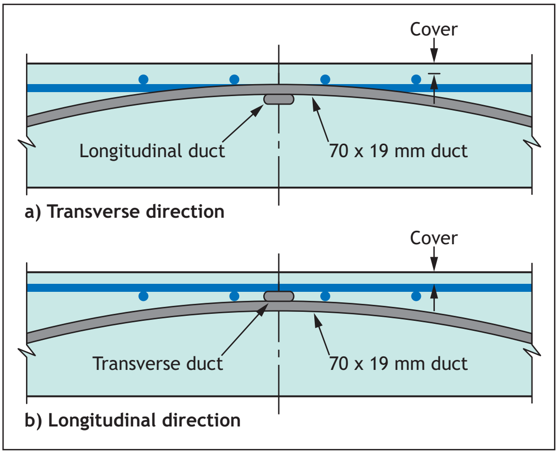


Figure 3.10
Positioning of tendons and reinforcing steel

3.16.5 Stress limits

The permissible stress limits are given in Table 3.17.

Table 3.17
Design flexural stresses for post-tensioned members (N/mm^2) (Based on tables 4.1 to 4.3 of BS 8110)

			Concrete class		
			C25/30	C32/40	⩾ C40/50
Compressive stresses					
In span and at cantilever supports			9.9	13.2	16.5
At support (except cantilevers)			12.0	16.0	20.0
Tensile stresses					
Class 1[a]			0.0	0.0	0.0
Class 2[b]			2.1	2.3	2.6
Class 3[c]	Crack width (mm)	Member depth (mm)			
	0.2	200	4.18	5.50	6.38
		400	3.80	5.00	5.80
		600	3.42	4.50	5.22
		800	3.04	4.00	4.64
		≥1000	2.66	3.50	4.06
	0.1	200	3.52	4.51	5.28
		400	3.20	4.10	4.80
		600	2.88	3.69	4.32
		800	2.56	3.28	3.84
		≥1000	2.24	2.87	3.36

Key
a Class 1 serviceability condition does not allow flexural tensile stresses
b Class 2 serviceability condition allows flexural tensile stresses but no visible cracking
c Class 3 serviceability condition allows flexural tensile stresses with maximum crack width of 0.1 mm for exposure classes XS2, XS3, XD2, XD3, XF3 and XF4, otherwise a crack width of 0.2 mm may be used

3.16.6 Initial design

The serviceability condition is usually the critical design criterion. At the initial stages it is reasonable to check the stresses at SLS and assume that the ULS requirements can be met in detailed design. Modern design software will allow further iteration and potentially a reduction in reinforcement. The following prestresses (i.e. *P/A*) are a good guide for initial design:

Slabs: 1.4 – 2.5 N/mm^2
Beams: 2.5 – 4.5 N/mm^2

For slabs, punching shear should also be checked (see 3.12.3), because this can often be a critical factor. A short, heavily loaded transfer beam may also be critical in shear and this may need checking.

The tendon profile should be idealised to simplify the calculations.

3.17 Columns

Usually the critical column location is in the lowest storey. However, moments in the edge and corner columns at the highest level may be also be critical.

3.17.1 Design

Short columns

A column should be checked to ensure that it is 'short'. The Code defines this as a column where l_{ex}/h and l_{ey}/b are less than 15 for a braced frame or less than 10 for an unbraced frame. (l_{ex} and l_{ey} are the effective lengths obtained from $l_e = \beta l_o$, where β is obtained from Table 3.18 .) Table 3.19 gives the minimum column dimensions where the clear height and β are known.

Table 3.18
Values of β for braced and unbraced columns

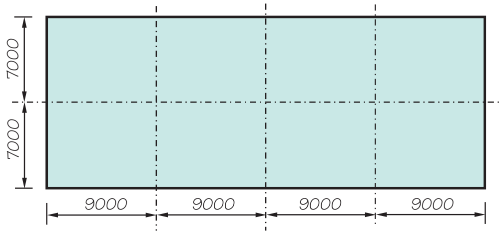
End condition at top	Values of β for braced columns			Values of β for unbraced columns		
	End condition at bottom			End condition at bottom		
	1	2	3	1	2	3
1	0.75	0.80	0.90	1.2	1.3	1.6
2	0.80	0.85	0.95	1.3	1.5	1.8
3	0.90	0.95	1.00	1.6	1.8	–
4	–	–	–	2.2	–	–

End conditions

There are four end conditions defined in the Code. These are:

- **Condition 1**. The end of the column is connected monolithically to beams on either side that are at least as deep as the overall dimension of the column in the plane considered. Where the column is connected to a foundation structure, this should be of a form specifically designed to carry moment.
- **Condition 2**. The end of the column is connected monolithically to beams or slabs on either side that are shallower than the overall dimension of the column in the plane considered.
- **Condition 3**. The end of the column is connected to members, which, while not specifically designed to provide restraint to rotation of the column will, nevertheless, provide some nominal restraint.
- **Condition 4**. The end of the column is unrestrained against both lateral movement and rotation (e.g. the free end of a cantilever column in an unbraced structure).

The Concrete Centre™	Project details	Calculated by OB	Job no. CCIP - 018
	Worked example 7	Checked by JB	Sheet no. PT1
	Post-tensioned slab	Client TCC	Date Dec 06



Imposed load = 5 kN/m^2

Superimposed dead load = 1.5 kN/m^2

Concrete class C32/40

Initial sizing 9000/36 = 250 mm thick

Geometry

Area of concrete, A_c = 7000 x 250 = 1750 x 10^3 mm^3

Second moment of area, I_c = 7000 x 250^3/12 = 9.11 x 10^9 mm^4

Distance to extreme fibres from neutral axis, $y_b = y_t$ = 125 mm

Section modulus, $Z_b = Z_t$ = 9.11 x 10^9/125 = 72.9 x 10^6

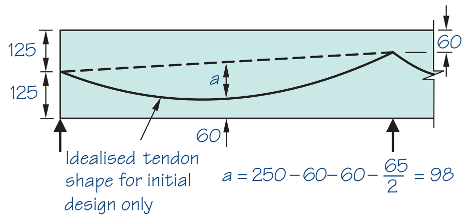
Strand diameter = 12.9 mm

Minimum cover = 20 mm

Design for class 3 serviceability condition

Distance to centre of strand = 60 mm

Tendon profile for end span (most critical):



$$a = 250 - 60 - 60 - \frac{65}{2} = 98$$

Number of strands required

Characteristic value of maximum force, 186 kN

Initial prestress = 0.8 x 186 = 149 kN (allow for 80% of characteristic force)

Prestress in service condition = 0.7 x 149 = 104 kN (allow for 10% loss at transfer and 20% loss at service, check in detailed design)

Balance dead loads with prestressing

$P = ws^2/(8a)$ = 7 x 6 x 9^2/(8 x 0.098) = 4339 kN

∴ No. of tendons required = 4339/104 = 41.7, try 8 x 5 strands per duct
Total force = 4160 kN

Moments (SLS)

Applied loads w_a = 7 x (6 + 1.5 x 5) = 87.5 kN/m

Balancing load $w_b = 8aP/s^2$ = 8 x 0.098 x 4160 / 9^2 = 40.2 kN/m

Balanced moment $M \approx (w_a - w_b)\, l^2/10$ = (87.5 – 40.2) x 9^2/10 = 383.1 kNm

Stresses

$$\sigma_t = \frac{P}{A_{ct}} + \frac{M}{Z_t} = \frac{4160 \times 10^3}{1750 \times 10^3} + \frac{383.1 \times 10^6}{72.9 \times 10^6} = 2.4 + 5.3 = 7.7 \text{ N/mm}^2$$

For class C32/40 allowable compressive stress is 13.2 N/mm^2 ∴ OK

$$\sigma_b = \frac{P}{A_{ct}} - \frac{M}{Z_b} = \frac{4160 \times 10^3}{1750 \times 10^3} - \frac{383.1 \times 10^6}{72.9 \times 10^6} = 2.4 - 5.3 = 2.9 \text{ N/mm}^2$$

For class C32/40 allowable tensile stress is 5.5 N/mm^2 ∴ OK

Comments

This leaves scope for reducing the number of tendons in detailed design

Punching shear should also be checked at this stage for a flat slab (see 3.12.3)

It is assumed that deflection, the ULS requirements and transfer requirements can be met with passively stressed reinforcement in detailed design, as is usually the case.

Table 3.19
Minimum column dimension for a given height and β-value (mm)

Clear height, l_o (m)	β for braced columns						β for unbraced columns					
	0.75	0.8	0.85	0.9	0.95	1	1.2	1.3	1.5	1.6	1.8	2.2
2.2	110	117	125	132	139	147	264	286	330	352	396	484
2.4	120	128	136	144	152	160	288	312	360	384	432	528
2.6	130	139	147	156	165	173	312	338	390	416	468	572
2.8	140	149	159	168	177	187	336	364	420	448	504	616
3.0	150	160	170	180	190	200	360	390	450	480	540	660
3.2	160	171	181	192	203	213	384	416	480	512	576	704
3.4	170	181	193	204	215	227	408	442	510	544	612	748
3.6	180	192	204	216	228	240	432	468	540	576	648	792
3.8	190	203	215	228	241	253	456	494	570	608	684	836
4.0	200	213	227	240	253	267	480	520	600	640	720	880
4.2	210	224	238	252	266	280	504	546	630	672	756	924
4.4	220	235	249	264	279	293	528	572	660	704	792	968
4.6	230	245	261	276	291	307	552	598	690	736	828	1012
4.8	240	256	272	288	304	320	576	624	720	768	864	1056
5.0	250	267	283	300	317	333	600	650	750	800	900	1100
5.2	260	277	295	312	329	347	624	676	780	832	936	1144
5.4	270	288	306	324	342	360	648	702	810	864	972	1188
5.6	280	299	317	336	355	373	672	728	840	896	1008	1232
5.8	290	309	329	348	367	387	696	754	870	928	1044	1276
6.0	300	320	340	360	380	400	720	780	900	960	1080	1320
6.2	310	331	351	372	393	413	744	806	930	992	1116	1364
6.4	320	341	363	384	405	427	768	832	960	1024	1152	1408
6.6	330	352	374	396	418	440	792	858	990	1056	1188	1452
6.8	340	363	385	408	431	453	816	884	1020	1088	1224	1496
7.0	350	373	397	420	443	467	840	910	1050	1120	1260	1540
7.2	360	384	408	432	456	480	864	936	1080	1152	1296	1584
7.4	370	395	419	444	469	493	888	962	1110	1184	1332	1628
7.6	380	405	431	456	481	507	912	988	1140	1216	1368	1672
7.8	390	416	442	468	494	520	936	1014	1170	1248	1404	1716
8.0	400	427	453	480	507	533	960	1040	1200	1280	1440	1760

Design using equations 38 and 39 from BS 8110

There are two practical methods of designing short braced columns available, either through equations 38 and 39 from BS 8110 or by using design charts.

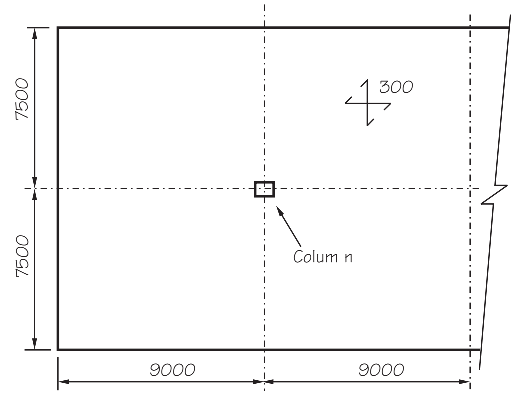
BS 8110 states that equation 38 can be used for columns that are not subject to 'significant' moments (i.e. where a symmetrical arrangement of beams is supported). The equation can re-arranged in terms of A_{sc}

$$A_{sc} = \frac{N - 0.4\, f_{cu}\, bh}{0.75 f_y}$$

Since it is not clear what 'significant' moments are, it is often prudent to use equation 39, which can be used for an approximately symmetrical arrangement of beams (variation of up to 15% of the longer) and for uniformly distributed loads. This has been rearranged in terms of A_{sc} below:

$$A_{sc} = \frac{N - 0.35\, f_{cu}\, bh}{0.67\, f_y}$$

The Concrete Centre™	Project details Worked example 8 Column	Calculated by OB	Job no. CCIP - 018
		Checked by JB	Sheet no. C1
		Client TCC	Date Dec 06



Imposed load = 5 kN/m^2

Superimposed load = 1.5 kN/m^2

Concrete class: C32/40

Cover = 25 mm

Loads

Column supports 4 storeys

Ultimate load per floor = 1.4 (0.3 x 24 + 1.5) + 1.6 x 5 = 20.2 kN/m^2

Total ultimate axial load = 4 x 20.2 x 7 x 9

N = 5085 kN

Initial sizing

Using *Economic concrete frame elements* – 450 mm square

Column design — Eqn 39, BS 8110

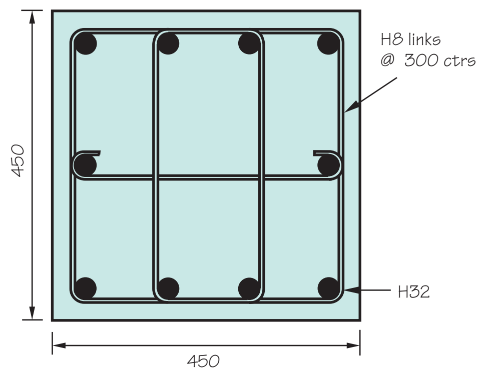
$$A_{sc} = \frac{N - 0.35 f_{cu} bh}{0.67 f_y}$$

$$= \frac{5085 \times 10^3 - 0.35 \times 40 \times 450^2}{0.67 \times 500}$$

= 6716 mm^2 (use 10 H32s – 8040 mm^2)

Comments

This is 4% reinforcement in the bottom storey, which will be reduced throughout its height



Where the geometry falls outside the scope of equation 39 the following factors can be applied to the load on the column from the floor above and used with equation 39 to approximately check the column capacity:

Internal column	1.25
Edge column	1.5
Corner column	2.0

3.17.2 Detailing

Maximum and minimum areas of reinforcement

The maximum area of either the tension or compression reinforcement is 6% of the gross cross-sectional area of the concrete for vertically cast columns, although this may be increased to 10% at laps. The minimum percentages are given in table 3.25 of BS 8110 (see Appendix B).

Maximum and minimum spacing of bars

The minimum spacing of the bars is maximum size of the coarse aggregate plus 5 mm or the bar size, whichever is the greater.

The maximum spacing is unlikely to apply to a principal column member.

Requirements for links

Minimum bar diameter is 6 mm or one quarter of the compression bar diameter.

Maximum link spacing is 12 times smallest compression bar, as shown in Table 3.20.

Table 3.20
Detailing requirements for links in columns

Bar diameter (mm)	16	20	25	32	40
Max. spacing (mm)	192	240	300	384	480
Min. link diameter (mm)	6[a]	6[a]	8	8	10

Key
a 6 mm bars are not readily available in the UK

3.18 Shear walls

3.18.1 Design issues

The wall should be checked for the worst combination of vertical loads, in-plane bending and bending out of the plane of the wall.

Stability walls should be continuous throughout the height of the structure. In plan the shear centre of the structure should coincide as much as possible with the centre of action of the applied horizontal loads in two orthogonal directions. Otherwise twisting should be considered.

Use the following design formula to calculate the maximum applied tensile stress:

$$\sigma_t = \frac{N}{Lt} \pm \frac{M}{tL^2 / 6}$$

where

N = ultimate axial load
M = ultimate in-plane moment
L = length of wall
t = thickness of wall

The ultimate compressive load should be less than the ultimate load capacity of the wall (F_c), i.e.

$$F_c \leq 0.35 f_{cu} A_c + 0.67 f_y A_{sc}$$

where

A_c = area of concrete per unit length of wall
A_{sc} = area of vertical reinforcement per unit length of wall

The required area of tension reinforcement can be calculated from:

$$A_s = \frac{0.5\sigma_t L_t t}{0.87 f_y}$$

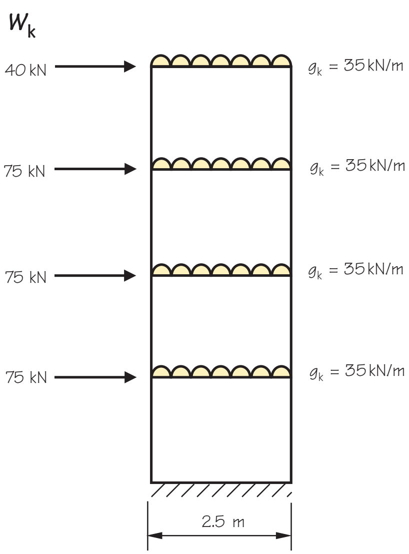
where

L_t = length of wall in tension

The length of the wall in tension is often taken as 1 m, but can be calculated to give a longer length of wall in tension, although the benefits may not be significant.

If there is tension at the base of the wall, this must be resisted by the weight of a mass foundation, or by designing the piles to resist tension.

The Concrete Centre™	Project details	Calculated by	Job no.
	Worked example 9 Shear walls	OB	CCIP - 018
		Checked by: JB	Sheet no.: SW1
		Client: TCC	Date: Dec 06



Assume critical combination is 1.0 G_k + 1.4 W_k

$f_t = \frac{N}{Lt} - \frac{M}{(tL^2/6)}$

$N = 1.0 \times 35 \times 4 \times 2.5 = 350$ kN

$M = 1.4\,(40 \times 12 + 75\,(9 + 6 + 3)) = 2562$ kNm

$f_t = \frac{350 \times 10^3}{2500 \times 200} - \frac{2562 \times 10^6}{(200 \times 2500^2/6)}$

$= 0.70 - 12.3$

$= -11.6$ N/mm^2

Assume that the tension is resisted by 1 m at the end of the wall.

$A_s = \frac{0.5\, f_t\, L_t\, t}{0.87\, f_{yv}}$

$= \frac{0.5 \times 11.6 \times 1000 \times 200}{0.87 \times 500} = 2667$ mm^2 or 1333 mm^2/face

Use $\boxed{\text{T16s @ 150 ctrs}}$ (1340 mm^2/face)

3.18.2 Walls with unequal stiffness

Where stability walls resisting a lateral load are not of equal stiffness, a torsion will be induced and the building will twist in plan if this not adequately resisted. It can be assumed that the floor diaphragm is infinitely stiff and therefore the lateral load on the walls can be apportioned according to their stiffness (see Figure 3.11).

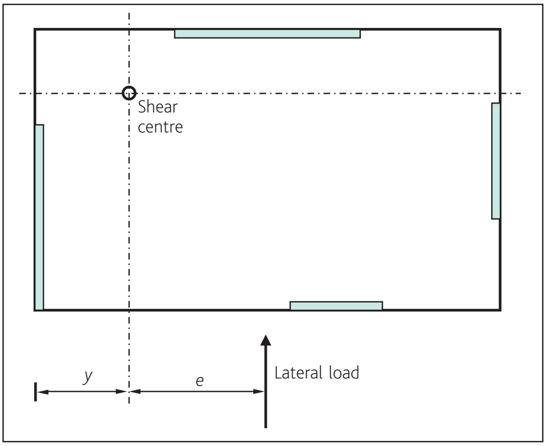


Figure 3.11
Shear centre for stability walls

The shear centre (centre of stiffness) can be calculated using the following expression:

$$y = \frac{\Sigma I_y}{\Sigma I}$$

A moment arises because the shear centre is offset and can be calculated, and walls in the orthogonal direction can be designed to resist the moment, see Worked example 10.

3.18.3 Detailing

Maximum and minimum areas of reinforcement

The maximum area of either the tension or compression reinforcement is 4% of the gross cross-sectional area of the concrete. The minimum percentages are 0.4% for vertical reinforcement and 0.25% for horizontal reinforcement.

Minimum spacing of bars

The minimum spacing of the bars is maximum size of the coarse aggregate plus 5 mm or the bar size, whichever is the greater.

3.19 Ground-bearing slabs

For the purpose of design, ground-bearing slabs can be divided into two categories:

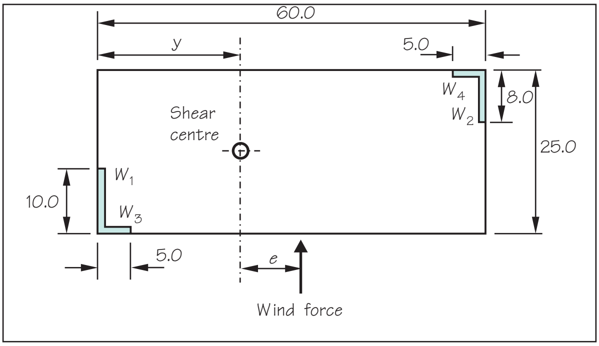
1. Normally loaded slabs for general use for which shrinkage is the governing criterion.
2. Heavily loaded slabs for industrial use.

Industrial ground floors are not considered further in this handbook, and reference should be made to TR34, *Concrete industrial ground floors*[21].

For normally loaded slabs with an imposed load up to 10 kN/m^2 the following guidance can be used:

- Use concrete Class C25/30.
- Slab should be placed on a slip membrane (e.g. 0.2 mm polythene sheet).
- Use a well-compacted sub-base not less than 150 mm thick.
- Isolation joints to be provided at the junction with external walls and around internal columns. See Figure 4.5.
- Depth, reinforcement and joint spacing requirements can be determined from Table 3.21. Note that the thicker the slab the closer the movement joints should be.

	Project details	Calculated by	Job no.
The Concrete Centre™	Worked example 10 Shear walls with varying stiffness	OB	CCIP - 018
		Checked by JB	Sheet no. SW2
		Client TCC	Date Dec 06



Take wind pressure as 1.0 kN/m^2

The relative stiffness of the walls can be calculated as follows (note as the walls are rectangular and the same thickness we can use the depth only)

W_1	$= 10^3$	$= 1000\ m^3$
W_2	$= 8^3$	$= 512\ m^3$
	Total	$= 1512\ m^3$

$$y = \frac{0.1 \times 1000 + 59.9 \times 512}{1512} = 20.3\ m$$

Eccentricity, $e = 30 - 20.3 = 9.7$ m

Twisting moment, $M_t = 9.7 \times 1.0 \times 60 \times 3.6 = 2095$ kNm per floor

In the worst case this moment can be resisted by the walls W_3 and W_4 only, the force (F) in each wall is:

$F = 2095/24.8 = 84.5$ kN per floor

To check for the critical case the direct design force on the walls W_3 and W_4 from the wind parallel to these walls = $1.0 \times 3.6 \times 25/2 = 45$ kN, and therefore the forces imposed by the twisting action are more onerous and should be used for design.

Comments — This layout is not particularly eccentric and yet still imposes large torsional forces.

Table 3.21
Maximum spacing of movement joints for ground bearing slabs (m)

Fabric reinforcement	Slab thickness (mm)				
	125	150	175	200	225
A142	25	21	18	16	14
A193	34	28	25	21	19
A252	44	37	31	28	25
A393	69	58	49	44	38
Unreinforced	–	6	6	6	6

3.20 Shallow foundations

3.20.1 Key considerations

- Settlement often controls the design.
- Foundations in clay, silt and chalk should be founded at 450 mm or below to avoid damage due to frost action.
- Foundations in shrinkable clay should be founded at 900 mm or below to protect against shrinkage/heave. Where trees are present this depth may have to be increased, see Table 2.3.
- The groundwater level will be crucial in the design; a high groundwater level will reduce the soil bearing capacity and may make the structure buoyant. The level adjacent to rivers and the sea can be expected to fluctuate.

3.20.2 Allowable bearing pressure

An estimate of allowable bearing pressures for typical soils found in the UK can be obtained from Tables 3.22 to 3.25 and Figure 3.12 for granular soils.

Table 3.22
Presumed allowable bearing values under static loading (Based on table 1 of BS 8004[22])

Category	Type of soil	Presumed allowable bearing value (kN/m²)	Remarks
Non-cohesive soils	Dense gravel, or dense sand and gravel	> 600	Width of foundation not less than 1 m. Groundwater level assumed to be below the base of the foundation.
	Medium dense gravel, or medium dense sand and gravel	< 200 to 600	
	Loose gravel, or loose sand and gravel	< 200	
	Compact sand	> 300	
	Medium dense sand	100 to 300	
	Loose sand	< 100	
Cohesive soils	Very stiff boulder clay and hard clay	300 to 600	Susceptible to long-term consolidation settlement
	Stiff clay	150 to 300	
	Firm clay	75 to 150	
	Soft clay and silt	<75	
	Very soft clay and silt	Not applicable	

Note
These values are for preliminary design purposes only.

Table 3.23

Presumed allowable bearing values for high porosity chalk (Based on table 2 of BS 8004)

Grade	Brief description	Presumed bearing value kN/m^2
VI	Extremely soft structureless chalk containing small lumps of intact chalk	See Cl. 2.2.2.3.1.8 of BS 8004
V	Structureless remoulded chalk containing lumps of intact chalk. Dry chalk above the water table	125 – 250
IV	Rubbly, part weathered chalk with bedding and jointing. Joints 10 mm to 60 mm apart, open to 20 mm, and often infilled with remoulded chalk and fragments	250 – 500
III	Rubbly to blocky unearthed chalk. Joints 60 mm to 200 mm apart, open to 3 mm, and sometimes infilled with fragments	500 – 750
II	Blocky medium hard (weak) chalk. Joints more than 200 mm apart and closed	750 – 1000
1	As for grade II, but hard (moderately weak) and brittle	1000 – 1500
Note For further information see Ward et al. [23].		

Table 3.24

Presumed allowable bearing values for Keuper Marl (Based on table 3 of BS 8004)

Degree of weathering	Zone	Description	Notes	Presumed bearing value (kN/m^2)
Fully weathered	IVb	Matrix only	Can be confused with solifluction or drift deposits, but contains no pebbles. Plastic slightly silty clay. May be fissured	See Cl. 2.2.2.3.1.9 of BS 8004
Partially weathered	IVa	Matrix with occasional clay-stone pellets less than 3 mm in diameter but more usually coarse sand size	Little or no trace of original (zone 1) structure, though clay may be fissured. Lower permeability than underlying layers	125 – 250
	III	Matrix with frequent lithorelicts up to 25 mm. As weathering progresses lithorelicts become less angular	Water content of matrix greater than that of lithorelicts	250 – 500
	II	Angular blocks of unweathered marl with virtually no matrix	Spheroidal weathering. Matrix starting to encroach along joints: first indications of chemical weathering	500 – 750
Unweathered	I	Mudstone (often fissured)	Water content varies due to depositional variations	750 – 1000
Note For further information see Chandler [24]				

Table 3.25

Undrained shear strength of cohesive materials (Based on table 6 of BS 8004)

Consistency			Undrained (immediate) shear strength kN/m^2
In accordance with BS 5930	Widely used descriptions	Field indications	
Very stiff	Very stiff or hard	Brittle or very tough	> 150
Stiff	Stiff	Cannot be moulded in the fingers	100 – 150
	Firm to stiff		75 – 100
Firm	Firm	Can be moulded in the fingers by strong pressure	50 – 75
	Soft to firm		40 – 50
Soft	Soft	Easily moulded in the fingers	20 – 40
Very soft	Very soft	Exudes between the fingers when squeezed in the fist	< 20

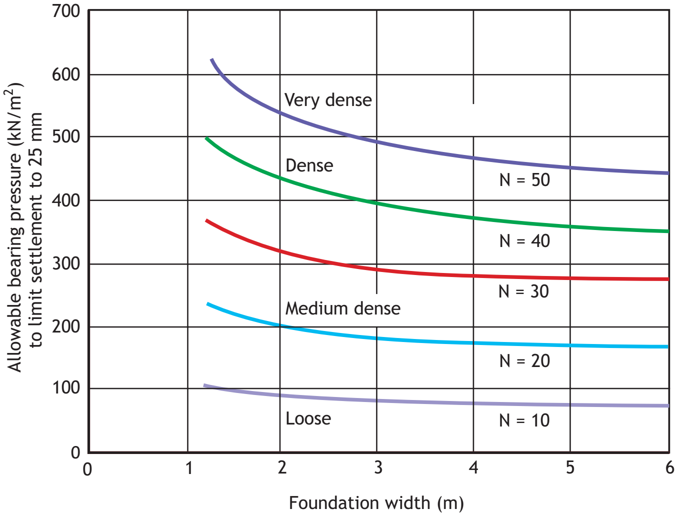


Figure 3.12
Chart for estimating allowable bearing pressure on granular material using standard penetration test
Diagram: Terzaghi and Peck

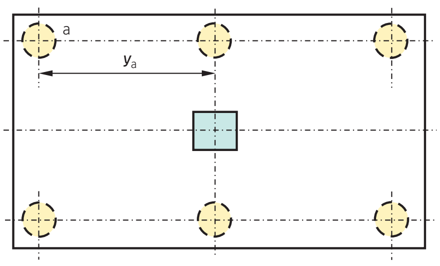
3.21 Piled foundations

3.21.1 Pilecap design

There are two generally accepted methods for pilecap design:

- Using bending theory, in which case they may be designed as a beam (see Section 3.9).
- Using truss analogy, which is presented in Table 3.26 for two-, three- and four-pile pilecaps.

For larger pilecaps bending theory should be used and the pile reactions can be calculated using Figure 3.13.



The force in pile 'a' is given by:

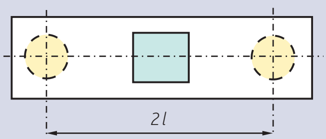
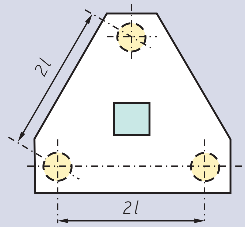
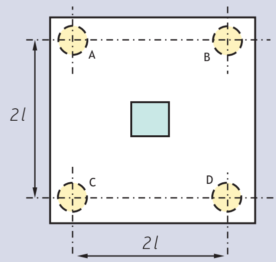
$$F_a = \frac{1}{N}\left(P \pm \frac{My_a}{\sum y_n^2}\right)$$

where

N = number of piles in group
P = total load applied
M = bending moment applied
y_n = local distance of the pile from the neutral axis of the group
y_a = local distance of pile 'a' from the neutral axis of the group

Figure 3.13
Pile forces for large pilecaps

Table 3.26
Truss analogy for pilecap design

Pilecap layout	Tension force in reinforcement
	$F_t = Pl/(2d)$ where P = load in the column l = distance from column to pile (see diagram) d = effective depth
	$F_{t(AB)} = F_{t(BC)} = F_{t(AC)}$ $= 2Pl/(9d)$ where P = load in the column l = distance from column to pile (see diagram) d = effective depth
	$F_{t(AB)} = F_{t(AC)} = F_{t(BD)} = F_{t(CD)}$ $= Pl/(4d)$ Force in longitudinal and transverse direction: $F_t = Pl/(2d)$ where P = load in the column l = distance from column to pile (see diagram) d = effective depth

Notes
1 Where column size is taken into account there may be efficiencies to be gained.
2 It is usual to space piles at three times their diameter.

3.21.2 Piles in cohesive material

The allowable bearing capacity of a pile in cohesive material is given by:

$$Q_a = \frac{Q_s}{\gamma_s} + \frac{Q_b}{\gamma_b} = \frac{\alpha\, c\, A_s}{\gamma_s} + \frac{c_u\, N_c\, A_b}{\gamma_b}$$

where

- α = adhesion factor. For bored piles use 0.3 for heavily fissured clay and 0.45 for firm to stiff clay. For driven piles use Nordland's adhesion factors (see Figure 3.14)
- c = average undrained shear strength over the length of the pile
- A_s = surface area of the pile ($\pi\, d\, l$, where d = diameter of pile and l = length of pile)
- γ_s = factor of safety on the shaft (use 3.0)
- c_u = undisturbed shear strength at the base of the pile
- N_c = Meyerhof's bearing capacity factor (use 9.0)
- A_b = area of the pile base ($\pi\, r^2$)
- γ_b = factor of safety on the base (use 2.5)

Table 3.27 can be used to look up the base capacity.

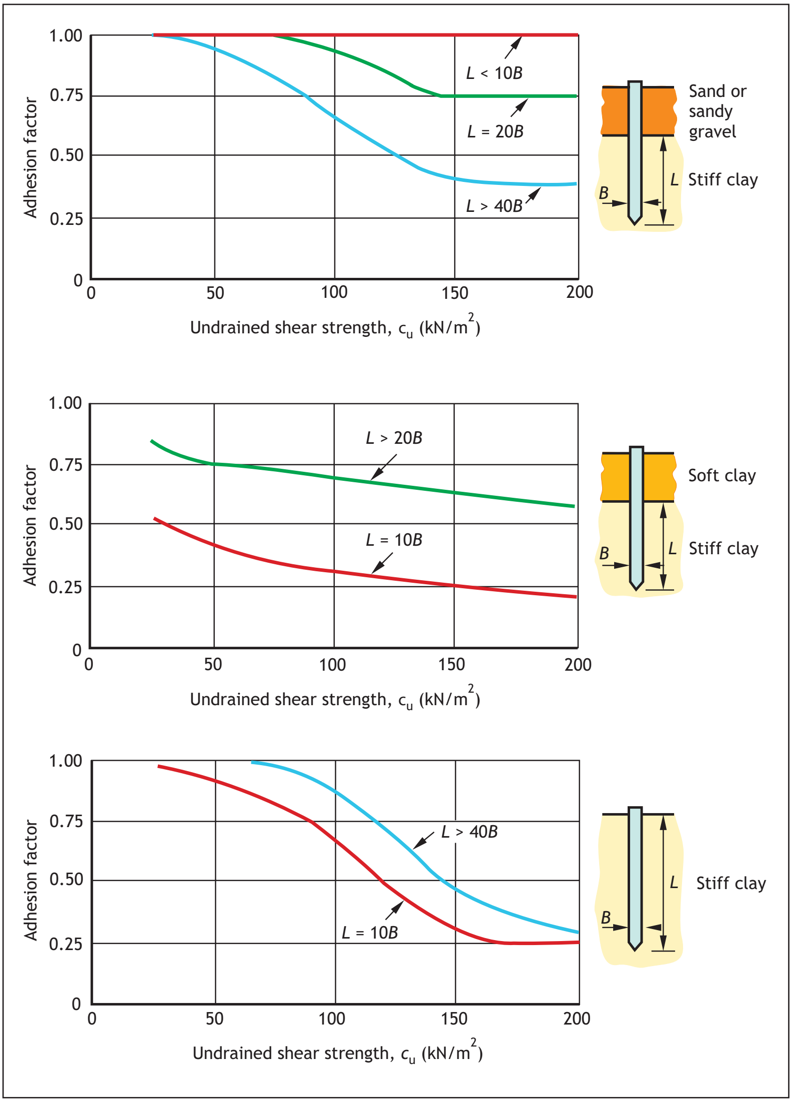


Figure 3.14
Nordlund's adhesion factors for driven piles

Table 3.27
Allowable pile base capacities $c_u N_c A_b / \gamma_b$ (kN)

Undisturbed shear strength, c_u (kN/m^2)	Pile diameter, mm				
	300	450	600	750	900
50	13	29	51	80	115
60	15	34	61	95	137
70	18	40	71	111	160
80	20	46	81	127	183
90	23	52	92	143	206
100	25	57	102	159	229
110	28	63	112	175	252
120	31	69	122	191	275
130	33	74	132	207	298
140	36	80	143	223	321
150	38	86	153	239	344
160	41	92	163	254	366
170	43	97	173	270	389
180	46	103	183	286	412
190	48	109	193	302	435
200	51	115	204	318	458
210	53	120	214	334	481
220	56	126	224	350	504
230	59	132	234	366	527
240	61	137	244	382	550
250	64	143	254	398	573
260	66	149	265	414	595
270	69	155	275	429	618
280	71	160	285	445	641
290	74	166	295	461	664
300	76	172	305	477	687

Note
$N_c = 9$ and $\gamma_b = 2.5$.

3.21.3 Group action – bored piles in clay

For a group of bored piles in a cohesive material the following expression can be used:

$$Q_{a,group} = n\, Q_a\, E_f$$

where

n = number of piles in group
Q_a = capacity of a single pile
E_f = group efficiency ratio

$$= 1 - \left(\tan^{-1}\frac{D}{s}\right)\frac{m(n-1) + n(m-1)}{90\, m\, n}$$

where

D = pile diameter
s = pile spacing
m = number of piles in one direction
n = number of piles in orthogonal direction

If $s = 3D$

$$E_f = 1 - \frac{18.43m(n-1) + n(m-1)}{90mn}$$

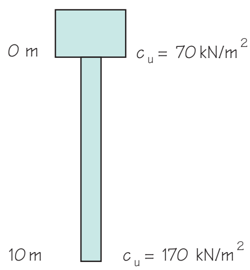
The values of E_f can be obtained from Table 3.28.

Table 3.28
Values for E_f for piles spaced at three times diameter

Number of piles, *m*	Number of piles, *n*				
	2	3	4	5	6
1	0.90	0.86	0.85	0.84	0.83
2	0.80	0.76	0.74	0.73	0.73
3	–	0.73	0.71	0.70	0.69
4	–	–	0.69	0.68	0.68
5	–	–	–	0.67	0.67
6	–	–	–	–	0.66

Note
For other combinations use 0.65.

The Concrete Centre™	Project details	Calculated by	Job no.
	Worked example 11 Bored pile in clay	OB	CCIP - 018
		Checked by: JB	Sheet no.: BP1
		Client: TCC	Date: Dec 06



Find safe working load of a 450 mm diameter bored pile in clay

$$Q_f = \frac{\alpha\, c\, A_s}{\gamma_s} + \frac{c_u N_c A_b}{\gamma_b}$$

$$Q_f = \frac{0.45\ (70 + 170)/2\ \pi \times 0.45 \times 10}{3.0} + \frac{170 \times 9 \times \pi \times 0.225^2}{2.5}$$

$= 254 + 97.3$

$= 351$ kN

3.21.4 Piles in granular soil

The allowable bearing capacity of a pile in granular material is given by:

$$Q_a = \frac{N_q{}^* A_b q'_o + A_s q'_{o,mean} k_s \tan\delta}{\gamma_f}$$

where

- $N_q{}^*$ = pile bearing capacity factor (see Table 3.29)
- A_b = area of the pile base
- q'_o = effective overburden pressure
- A_s = surface area of the pile shaft in the granular soil
- $q'_{o,mean}$ = mean overburden pressure
- k_s = horizontal coefficient of earth pressure (see Table 3.30)
- δ = angle of friction between the soil and the pile face (see Table 3.29)
- γ_f = factor of safety (2.5 to 3.0)

Table 3.29
Typical values of pile bearing capacity factor, N_q*

Angle of internal friction, ϕ (°)	Ratio of pile length/pile diameter		
	5	20	70
25	16	11	7
30	29	24	20
35	69	53	45
40	175	148	130

Based on charts by Berezantsev for N_q

Table 3.30
Typical values for δ and k_s in granular material

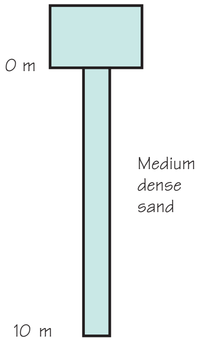
Angle of internal friction, ϕ (°)	Angle of friction between the soil and the pile face, δ (°)		Horizontal coefficient of earth pressure, k_s		
	In-situ concrete piles	Precast concrete piles	In-situ concrete piles	Large driven piles	Small driven piles
26	26	23.4	0.393 – 0.562	0.562 – 1.123	0.421 – 0.702
27	27	24.3	0.382 – 0.546	0.546 – 1.092	0.410 – 0.683
28	28	25.2	0.371 – 0.531	0.531 – 1.061	0.398 – 0.663
29	29	26.1	0.361 – 0.515	0.515 – 1.030	0.386 – 0.644
30	30	27.0	0.350 – 0.500	0.500 – 1.000	0.375 – 0.625
31	31	27.0[a]	0.339 – 0.485	0.485 – 0.970	0.364 – 0.606
32	32	27.0[a]	0.329 – 0.470	0.470 – 0.940	0.353 – 0.588
33	33	27.0[a]	0.319 – 0.455	0.455 – 0.911	0.342 – 0.569
34	34	27.0[a]	0.309 – 0.441	0.441 – 0.882	0.331 – 0.551
35	35	27.0[a]	0.298 – 0.426	0.426 – 0.853	0.320 – 0.533
36	36	27.0[a]	0.289 – 0.412	0.412 – 0.824	0.309 – 0.515
37	37	27.0[a]	0.279 – 0.398	0.398 – 0.796	0.299 – 0.498
38	38	27.0[a]	0.269 – 0.384	0.384 – 0.769	0.288 – 0.480
39	39	27.0[a]	0.259 – 0.371	0.371 – 0.741	0.278 – 0.463
40	40	27.0[a]	0.250 – 0.357	0.357 – 0.714	0.268 – 0.447
41	41	27.0[a]	0.241 – 0.344	0.344 – 0.688	0.258 – 0.430
42	42	27.0[a]	0.232 – 0.331	0.331 – 0.662	0.248 – 0.414
43	43	27.0[a]	0.223 – 0.318	0.318 – 0.636	0.239 – 0.398
44	44	27.0[a]	0.214 – 0.305	0.305 – 0.611	0.229 – 0.382
45	45	27.0[a]	0.205 – 0.293	0.293 – 0.586	0.220 – 0.366
46	46	27.0[a]	0.196 – 0.281	0.281 – 0.561	0.210 – 0.351

Key
a It is recommended that δ should be based on loose conditions for precast piles

Table 3.31
Soil properties for granular soils

Description	SPT 'N' blows	Effective angle of internal friction, ϕ' (°)	Bulk density (kN/m^3)
Very loose	0 – 4	26 – 28	<16
Loose	4 – 10	28 – 30	16 – 18
Medium dense	10 – 30	30 – 36	18 – 19
Dense	30 – 50	36 – 42	19 – 21
Very dense	>50	42 – 46	21

The Concrete Centre™	Project details	Calculated by	Job no.
	Worked example 12 Bored pile in sand	OB	CCIP - 018
		Checked by	Sheet no.
		JB	BP2
		Client	Date
		TCC	Dec 06



Find safe working load of a 450 mm diameter bored pile in sand

$$Q_a = \frac{N_q^* A_b q_o' + A_s q_{o,mean}' k_s \tan\delta}{\gamma_f}$$

Take $\phi' = 33°$, $l/d = 10000/450 = 22.2$

$\therefore N_q^* \approx 41$

$$Q_a = \frac{41 \times \pi \times 0.225^2 \times 18 \times 10 + \pi \times 0.45 \times 10 \times 18 \times 5 \times 0.387 \times \tan 33}{3}$$

$$Q_a = \frac{1174 + 320}{3}$$

$= 498$ kN

3.21.5 Piles in chalk

The allowable bearing capacity of a pile in chalk is given by:

$Q_a = A_s\, Q_{a,s} + A_s\, Q_{a,b}$

where

A_s = surface area of the pile shaft in the soil
$Q_{a,s}$ = allowable capacity of pile shaft skin friction (see Table 3.32)
A_b = area of the pile base
$Q_{a,b}$ = allowable capacity of the pile base (see Table 3.32)

Table 3.32
Values for allowable capacity of pile shaft skin friction, $Q_{a,s}$ and allowable capacity of the pile base, $Q_{a,b}$ for piles in chalk

SPT 'N' blows	Pile shaft skin friction capacity			Pile base capacity		
	Unfactored, $Q_{u,s}$	Factor of safety, γ_s	Allowable, $Q_{a,s}$	Unfactored, $Q_{u,b}$	Factor of safety, γ_b	Allowable, $Q_{a,b}$
10	35	2	18	2500	5	500
15	70	2	35	3750	4	940
20	105	2	53	5000	3	1670
25	170	2	85	6250	3	2080
30	250	2	125	7500	3	2500
35	250	2	125	7750	3	2580
≥40	250	2	125	8000	3	2670

3.21.6 Basement construction

It is generally considered that a concrete basement wall should be at least 300 mm thick. Concrete can be designed to prevent the ingress of water, although attention to detail is required at joints. Concrete cannot be designed to prevent the passage of water vapour.

The waterproofing of a basement is an important consideration, and the starting point is the intended use. Guidance is given in BS 8102[25] and CIRIA 139[26]. Table 3.33 is based on table 1 of BS 8102 and table 2.1 of CIRIA 139.

There are three generic types of construction given in Table 3.33. Type A is 'Tanking', Type B is 'Structural integral protection' and Type C is 'Drained cavity protection'. These are discussed in more detail below and typical details are included in Section 4.

Tanking (Type A)

Tanking provides a continuous barrier system, which excludes water and/or vapour. It can be either a membrane or liquid applied, and can be installed in the following places:

- Onto an external source of support (reversed)
- On the exterior face of walls and floors (external)
- Within the construction (sandwiched)
- On the interior of the walls (internal)

The systems rely on good joints in the membrane systems, protection from damage and being adequately bonded to the substrate. Internal systems may be unable to resist the hydrostatic pressures.

Structural integral protection (Type B)

Structural reinforced concrete walls and floors can be designed and constructed to protect against water penetration. The concrete should be high quality both in terms of design and construction. Attention should be paid to preventing cracks from forming and to detailing simple joints to avoid problems with workmanship. The design should be to BS 8007 for grade 2 to 4 environments and may be to BS 8110 for grade 1 environments.

Most consultants consider that this option is a high-risk strategy, especially for grade 3 and 4 environments. There are various concrete additives, which the manufacturers claim will make concrete watertight, and some come with long guarantees. Many consultants are prepared to use these products where the risks are transferred to the manufacturer of the product.

Drained cavity protection (Type C)

Drained cavity systems are designed to collect moisture that does penetrate the retaining wall. The retaining wall should still be designed to minimise water penetration. The cavity is formed by introducing an inner non load-bearing wall, which may need to be designed as free standing to prevent moisture paths across cavity ties. Moisture is collected in a sump and pumped away.

Drained systems are probably the most expensive, and also reduce the usable floor area. There is also the requirement to run and maintain the pumps, and the discharge of the water requires a licence from the water authority. However, they are considered to be the most robust solution because workmanship, although important, is less critical to their success. There is also the opportunity to carry out remedial measures to the retaining wall to reduce significant water penetration before the inner wall is constructed.

Table 3.33
Guide to level of protection to suit basement use

Grade (description)	Basement usage	Performance level	Form of construction	Comment
1 (basic utility)	Car parking; plant rooms (excluding electrical equipment); workshops	Some seepage and damp patches can be tolerated	**Type B** Reinforced concrete design in accordance with BS 8110	For a workshop to comply with the Buildings Regulations this level of protection is unlikely to be suitable. The crack width should be less than 3 mm; design to BS 8110–2
2 (better utility)	Workshops and plant rooms requiring drier environment; retail storage areas	No water penetration but moisture vapour can be tolerated	**Type A** Tanking **Type B** Reinforced concrete design in accordance with BS 8007	A high level of supervision is required during construction. Performance relies on the workmanship
3 (habitable)	Ventilated residential and working areas including offices, restaurants etc., leisure centres	Dry environment	**Type A** Tanking **Type B** With reinforced concrete design to BS 8007 **Type C** With wall and floor cavity and DPM	High standard of workmanship required. Additional protection recommended with Type B construction
4 (special)	Archives and stores requiring controlled environment	Totally dry environment	**Type A** Tanking **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	High standard of workmanship required. A combination of Type B and Type C construction will give the best protection

4 Plans, sections, elevations and critical details (section 2d)

The question asks for plans, sections and elevations to show the dimensions, layout and disposition of the structural elements and critical details for estimating purposes. The important aspects to note from the question are that it is usually necessary to include more than one plan showing the following as appropriate:

- Foundations
- Basement
- Ground floor
- Suspended floors

It is not necessary to show the full extent of every floor; where there is symmetry or repetition only a portion needs to be drawn along with a clear note. The drawings should be drawn on the A3 graph paper provided; they should look professional and include dimensions and the structural grid. A full height cross-section is generally essential and sometimes a longitudinal section is required too. An elevation should be included where it is going to convey additional information such as the cladding supports.

There is at most 85 minutes to answer this section, so the candidate will need to work swiftly to complete the drawings and there is no substitute for experience. Only half the marks will be awarded for the plans, so avoid the trap of spending too much time on the plans at the expense of the other requirements.

4.1 Plans, sections and elevations

A typical example of a floor plan and section for a reinforced concrete building is shown in Figure 4.1. Plans and cross-sections for foundations should be shown in a similar level of detail.

It is assumed that a cost estimate will be prepared using the drawings and so the following information should be provided on the plans:

- Concrete class
- Reinforcement estimates for all elements, especially unusual elements
- Type of reinforcement
- Concrete finishes
- Dimensions to grids and structural sizes
- Cladding supports
- Movement joints
- Waterproofing details, including construction joints for watertight concrete
- Any other information required to enable a cost estimate to be prepared

4.2 Critical details

The critical details provided here are by no means the complete range of details that may be encountered. **Merely reproducing these in the examination will not demonstrate your competency to the examiners; the details will need to be adapted to the particular circumstances of your design**. The question usually notes the critical details are for estimating purposes; so in the time available you should ensure that you provide the details that will have the most effect on the cost of the project, not the details that are easiest to produce.

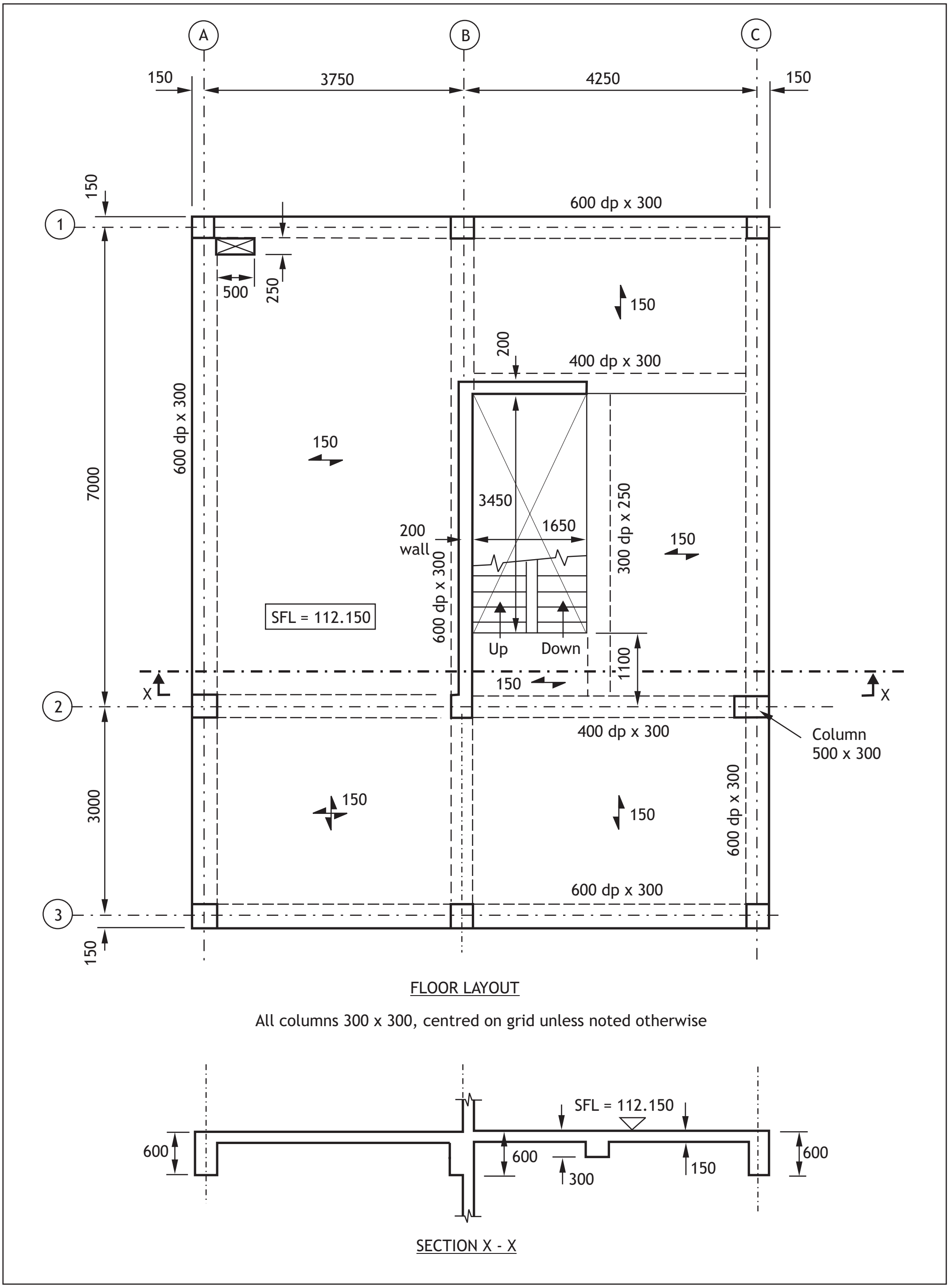


Figure 4.1
Example of general arrangement drawing for a concrete structure

4.2.1 Foundations

Table 4.1
Typical pilecap sizes (mm)

Diameter	Depth	Length, *L*	2-pile arrangement breadth, *B*	3-pile arrangement			
				Breadth, *B*	*X*	*Y*	*Z*
300	700	1500	600	1400	550	600	560
450	1000	2100	750	1900	700	750	760
600	1400	2700	900	2450	850	900	970
750	1800	3300	1050	3000	1000	1050	1170
900	2200	3900	1200	3550	1150	1200	1370
1200	3000	5100	1500	4590	1450	1500	1780
1500	3800	6300	1800	5660	1750	1800	2190

Note
Dimensions *L*, *B*, *X*, *Y* and *Z* are given in Figure 4.2.

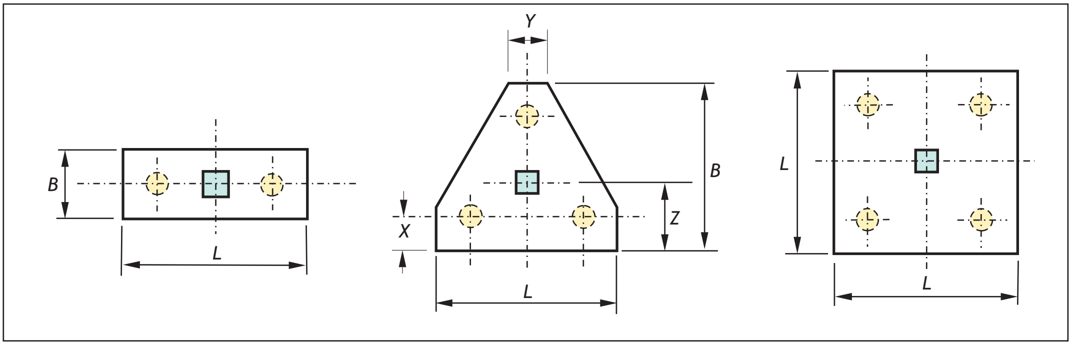


Figure 4.2
Typical pilecap layouts

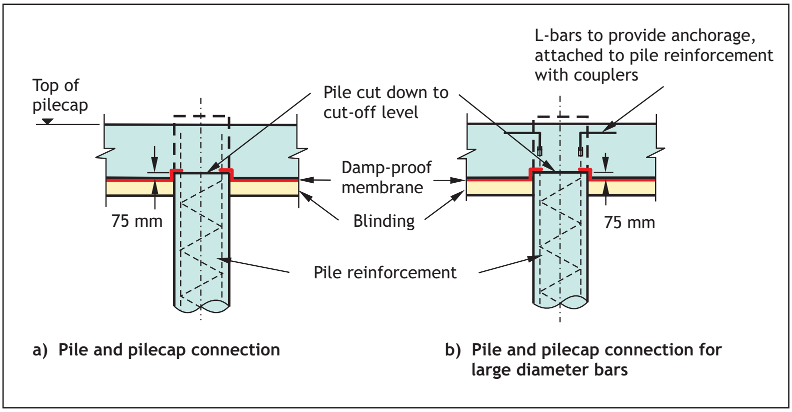


Figure 4.3
Pile and pilecap connections

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

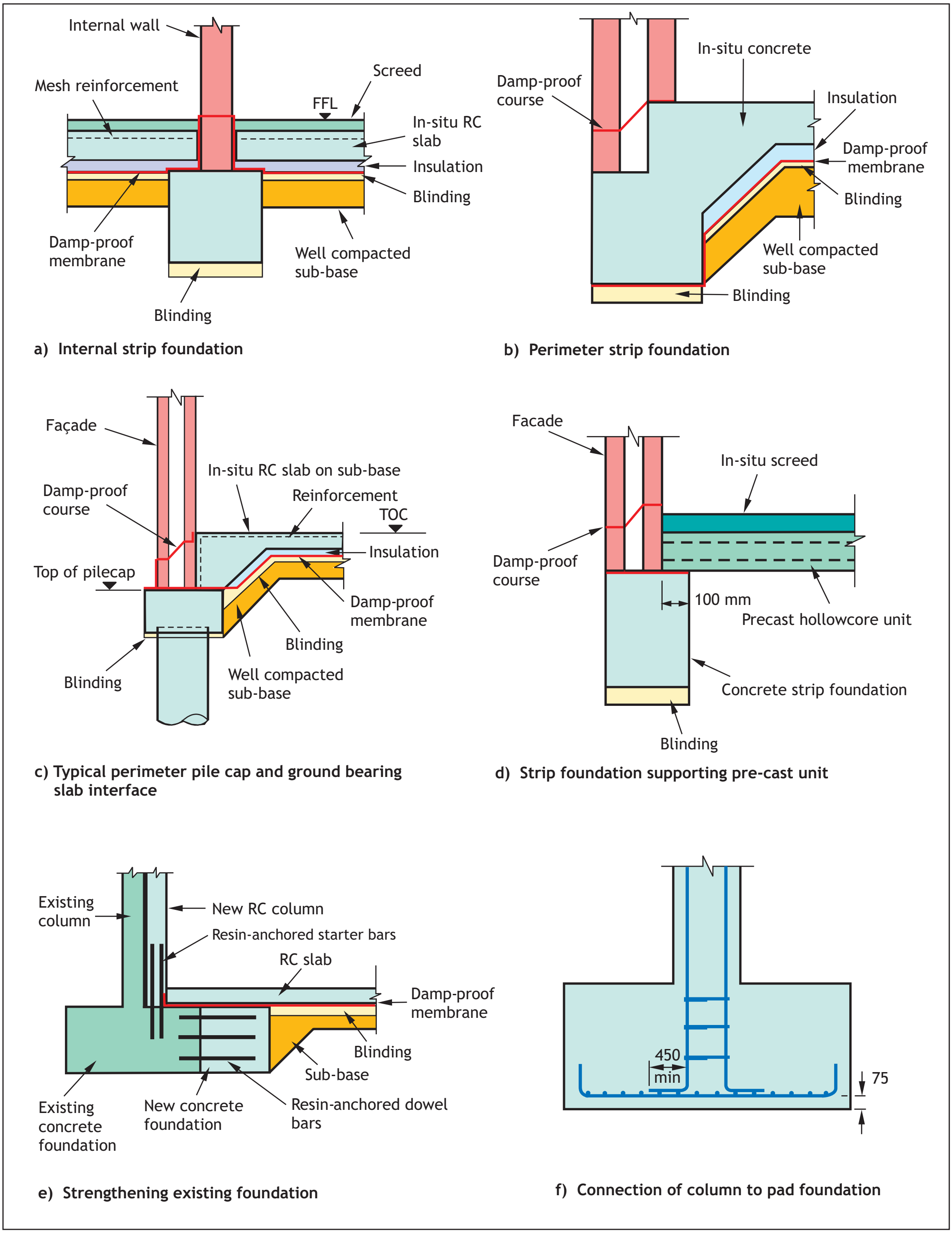
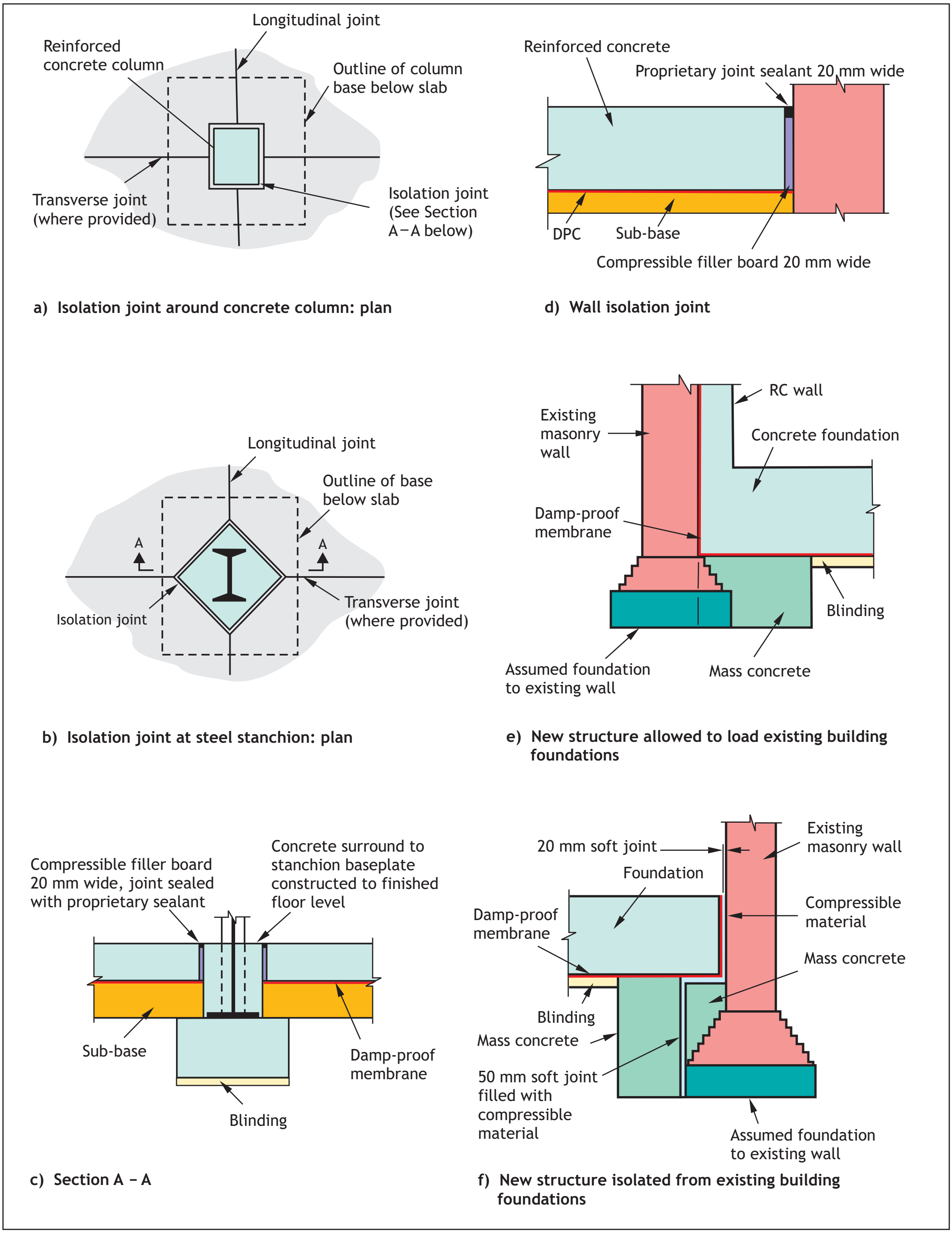


Figure 4.4
Shallow foundations 1

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.



a) Isolation joint around concrete column: plan

d) Wall isolation joint

b) Isolation joint at steel stanchion: plan

e) New structure allowed to load existing building foundations

c) Section A – A

f) New structure isolated from existing building foundations

Figure 4.5
Shallow foundations 2

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

4.2.2 Retaining walls

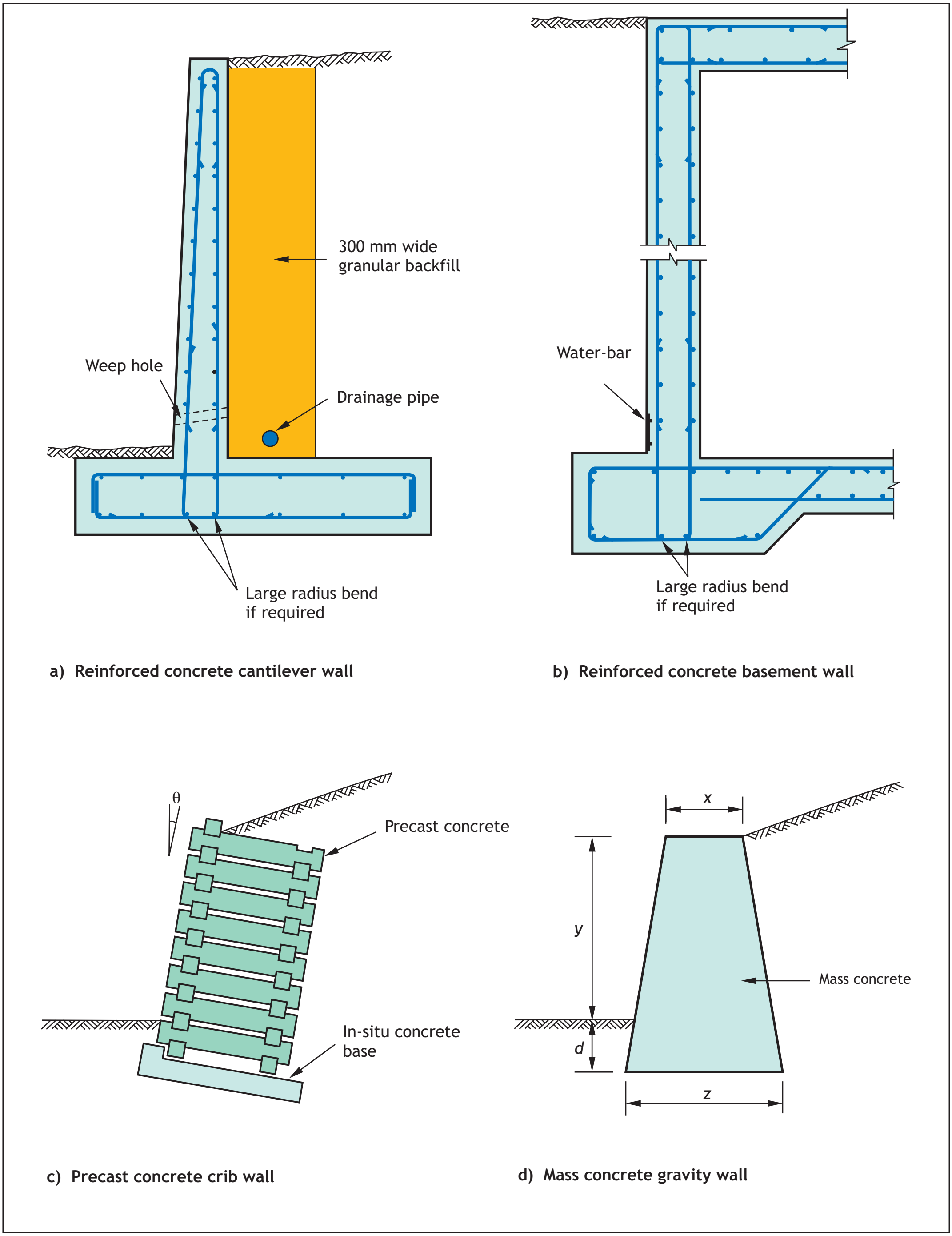


Figure 4.6
Retaining structures

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

4.2.3 Basement walls

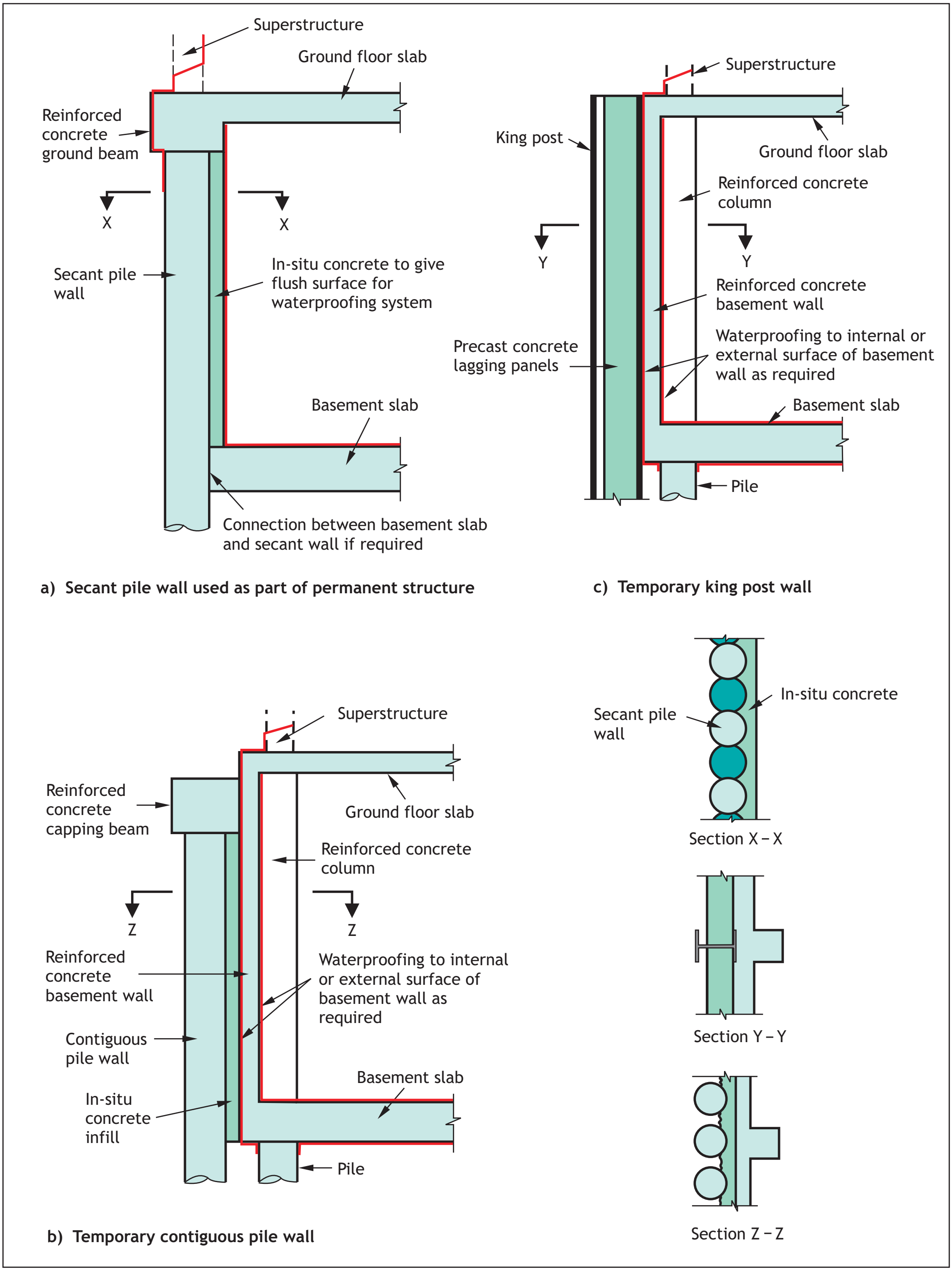
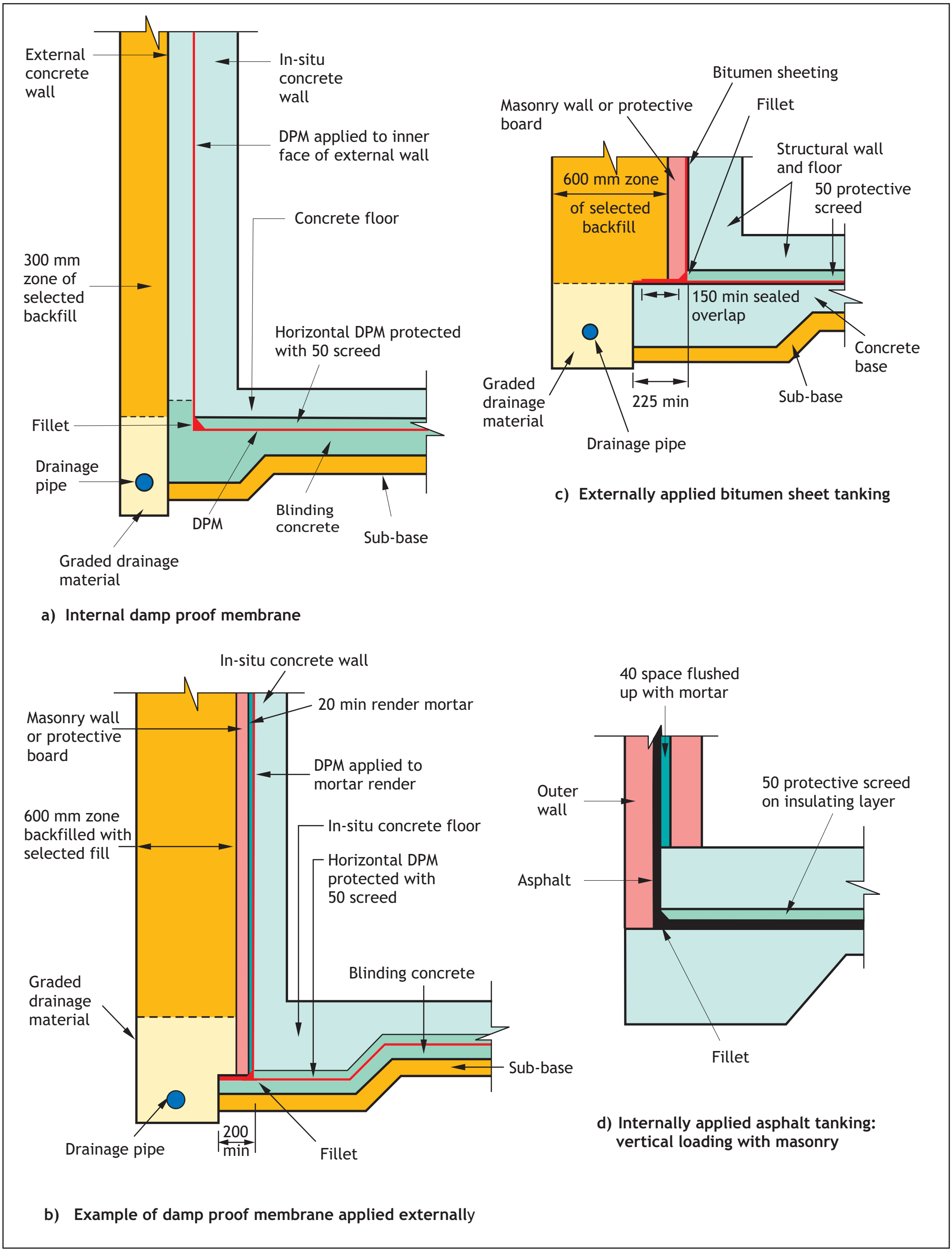


Figure 4.7
Basement walls

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

4.2.4 Waterproofing



a) Internal damp proof membrane

c) Externally applied bitumen sheet tanking

b) Example of damp proof membrane applied externally

d) Internally applied asphalt tanking: vertical loading with masonry

Figure 4.8
Tanked waterproofing options (Type A)

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

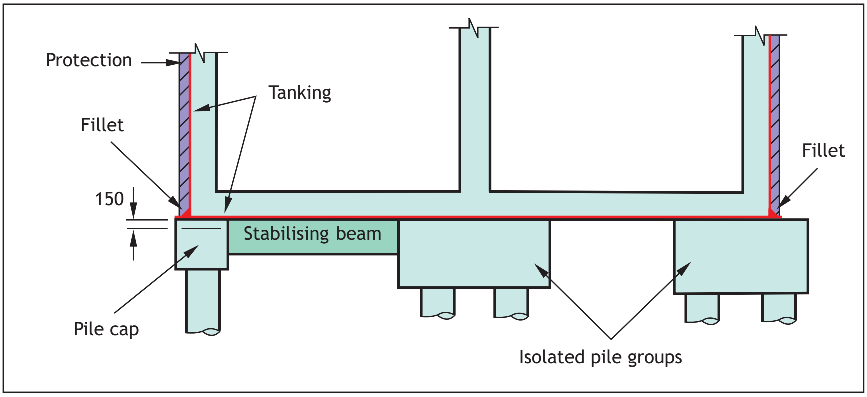


Figure 4.9
Tanked concrete basement carried on piles

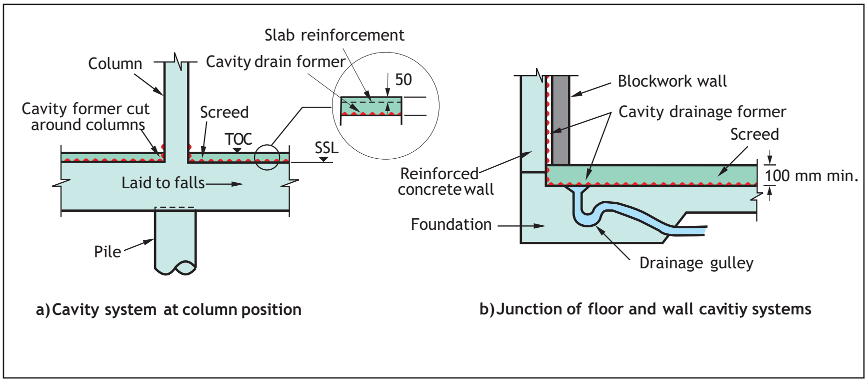


Figure 4.10
Typical cavity drainage (Type C)

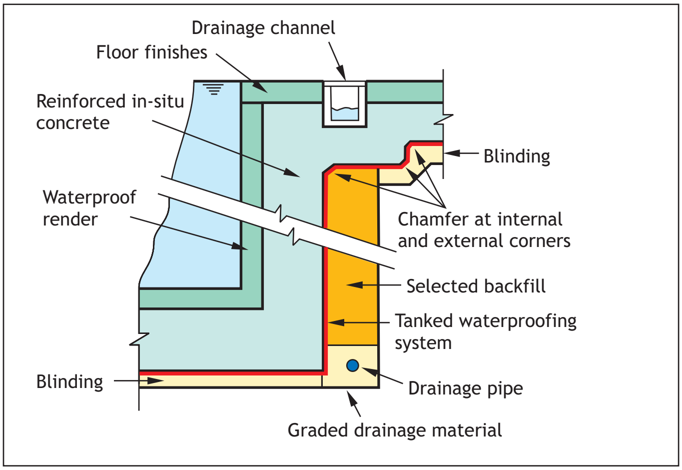


Figure 4.11
Swimming pool waterproofing

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

4.2.5 Cladding

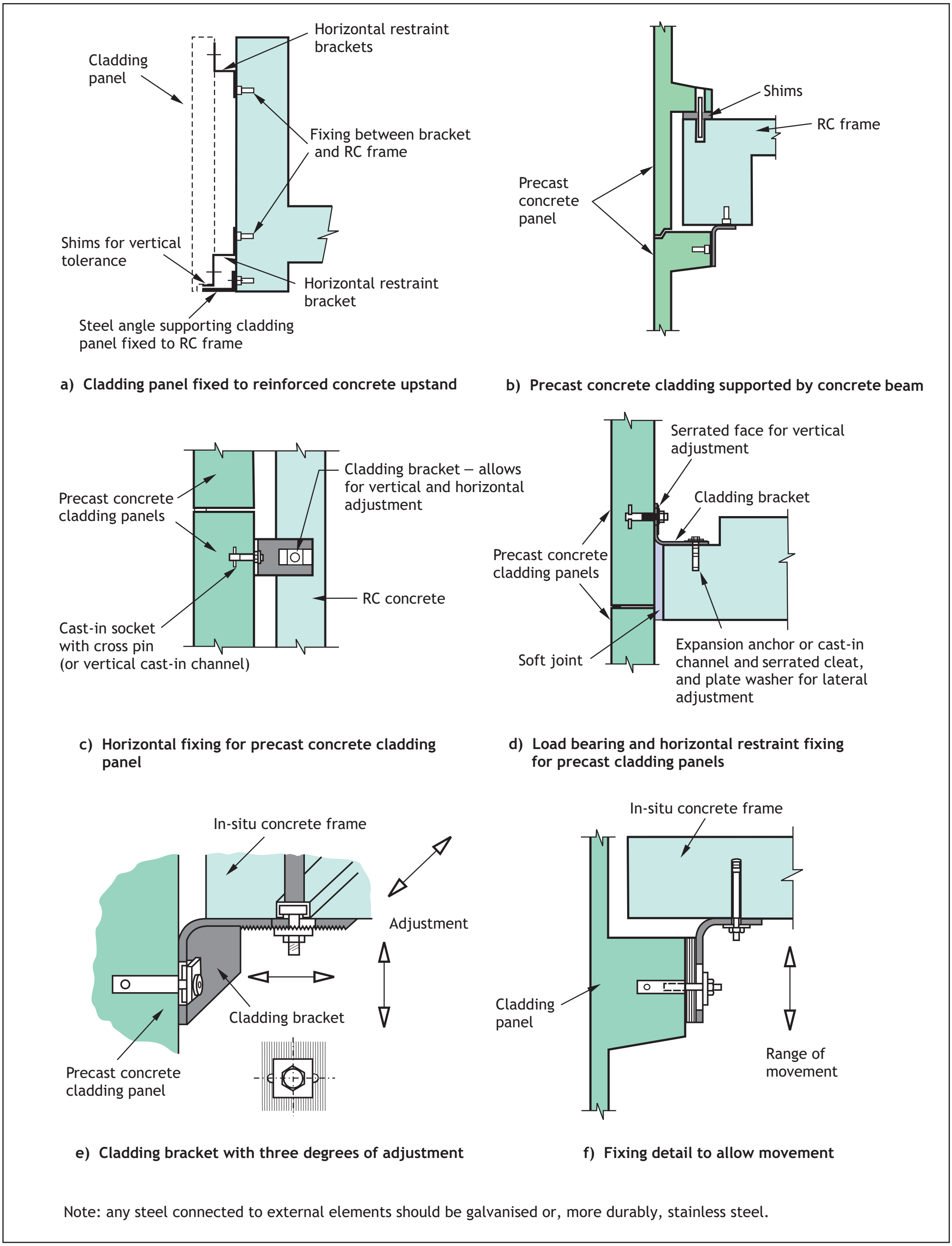


Figure 4.12
Typical fixing details for precast concrete panels

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

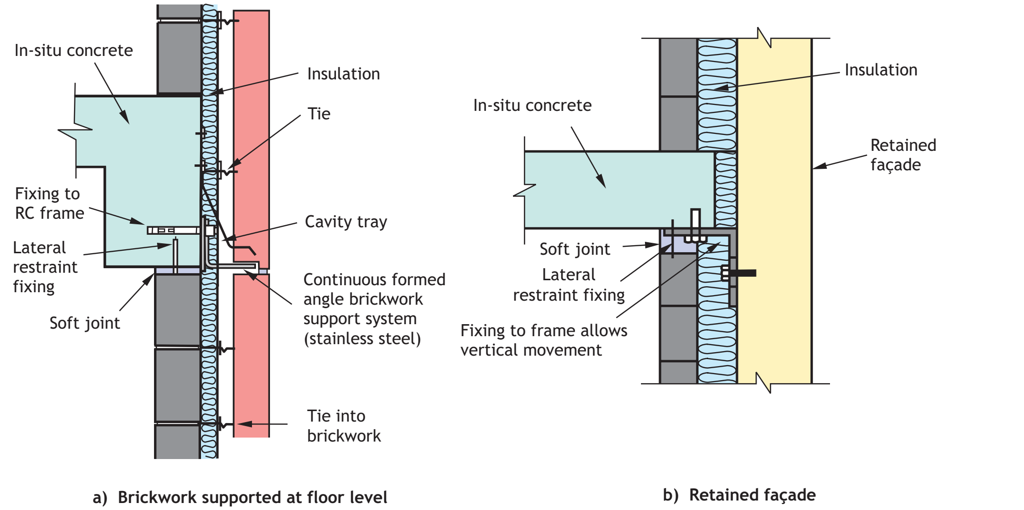


Figure 4.13
Fixing of masonry façades

4.2.6 Waterstops

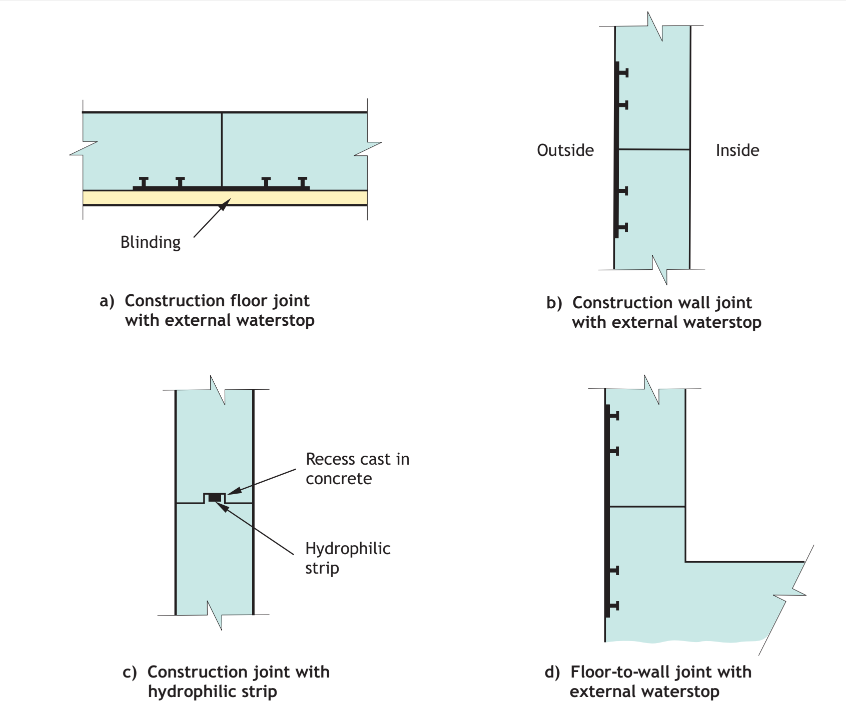


Figure 4.14
Typical applications of waterstops

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

4.2.7 Movement joints

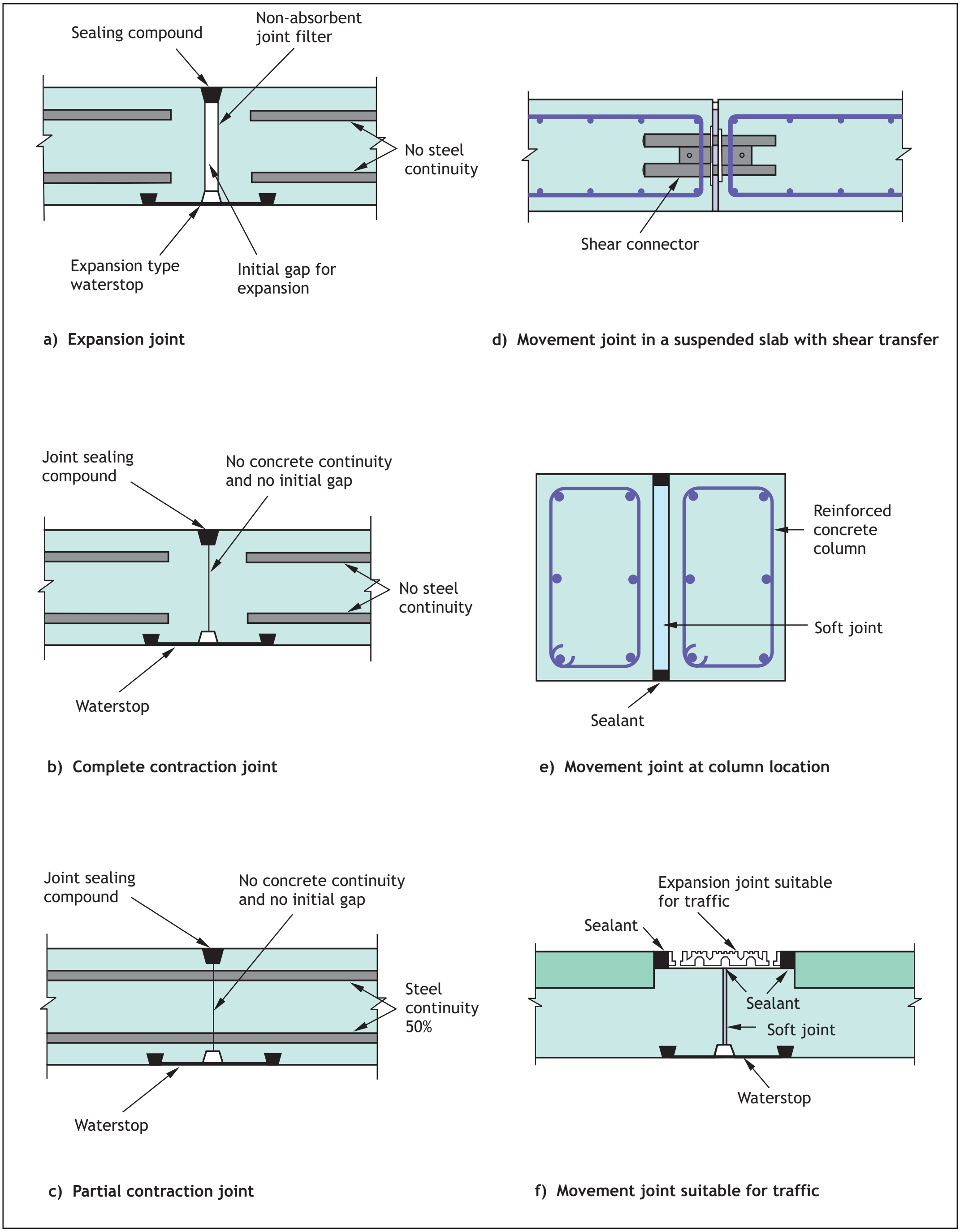


Figure 4.15
Movement joints

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

4.2.8 Superstructure

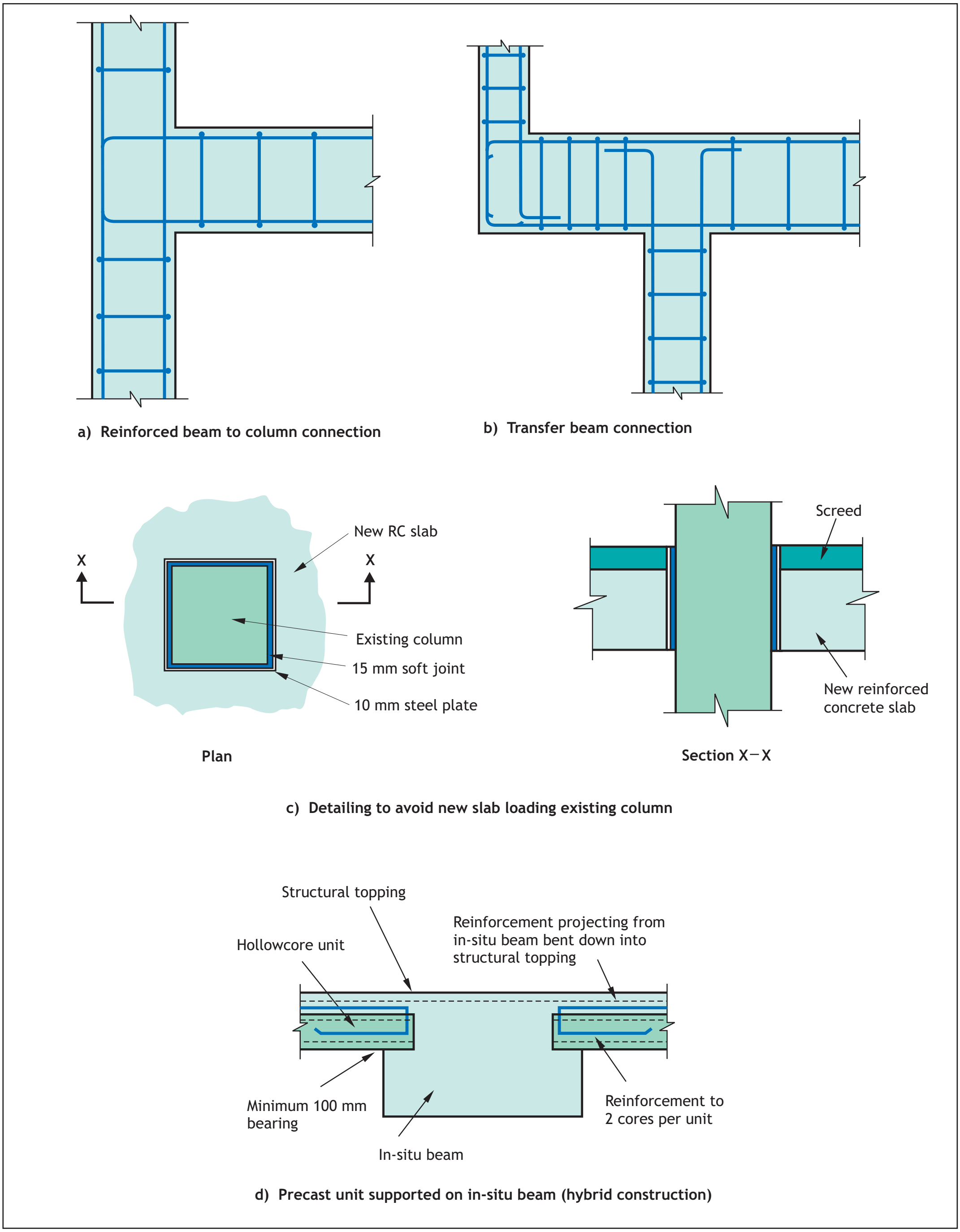


Figure 4.16
Superstructure details

Adapt these diagrams to suit - DON'T just reproduce them. Give dimensions where appropriate.

5 Method statement and programme (section 2e)

5.1 Method statement

The method statement should be written for the particular circumstances of the project; the examiners are looking for method statements that demonstrate an understanding of the construction process and how to build safely. The method statement and programme together are worth just 10% of the marks, so there is generally not much time available to complete this part of the question. Candidates tend to run short of time at the end of the examination and so this part is usually not well executed. Nevertheless there are marks available and every effort should be made to provide a clear method statement.

5.1.1 General contents of the method statement

The method statement should describe how the candidate considers that the specific building in the question can be safely constructed. The following outline may prove useful:

Preliminaries

The site set-up should be outlined; particular issues to consider are:

- Interfaces with the public
- Interfaces with adjacent properties
- Site logistics
- Handling of materials
- Plant and equipment to be provided, e.g. cranes

Sequence of work

- Identify the sequence of work and the dependencies between each activity
- Briefly describe each activity
- List the activities in a logical order
- Highlight key restraints and how they can be overcome
- Section 2.7.1 highlighted how health and safety risks should be minimised. Methods for reducing any remaining risks should be including here

5.1.2 Detailed considerations

The designer can influence the health and safety risks on a project by considering the issues at an early stage; the following list is an indication of areas where designers can have an influence:

- Reduce the requirement to **work at height**.
- Reduce the requirement to work in **excavations** and **confined spaces**, e.g. minimise the depths of foundations.
- Avoid the specification of materials that are **difficult to handle** e.g. long lengths of 40 mm diameter reinforcing bar.
- Clearly convey the lateral **stability requirements** for the permanent works.
- Avoid the specification of techniques that generate **noise**.
- Minimise the use of techniques that cause **vibration**.
- Consider the depth of the **groundwater** in the design.
- Reduce the risk of contact with **contaminants** e.g. specify thorough site investigation if there is a risk of contamination identified in a desk study.
- Identify the location of **utility services**.
- Consider **buoyancy** during construction.

- Construction close to **adjacent properties**.
- **Refurbishment** of existing properties.
- **Weight of elements** to be lifted.
- Types of **lifting equipment** envisaged for the project.
- **Temporary propping** requirements.

5.2 Programme

As with method statements, the candidate is usually under time pressure at this stage of the examination but, again, it is an opportunity to demonstrate your knowledge. The information provided here is really intended for initial programming only, **not** as a detailed guide on construction periods.

Typical construction rates can be obtained from Table 5.1 or Figure 5.1.

Table 5.1
Typical construction rates

Construction type	Construction rate
Establish site	2 weeks
Bored/CFA piled (600 mm dia. x 30 m long)	7 per day
Driven precast piles	From 200 m/day hard ground to 1000 m/day soft ground

Note
Rates given are generic and intended only for guidance.

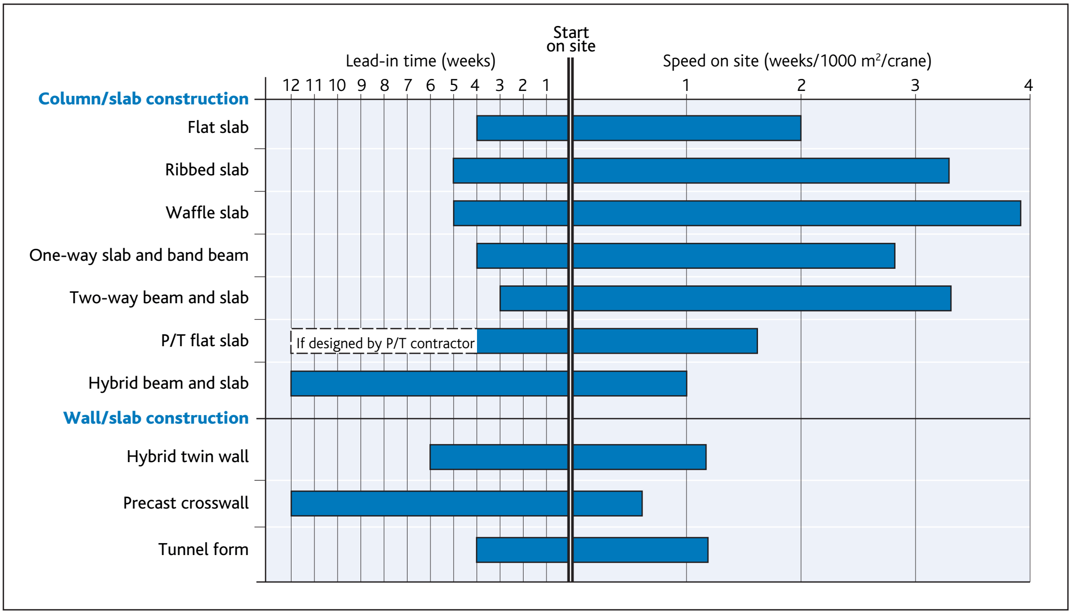


Figure 5.1
Lead-in times and speed of construction

5.2.1 Example programmes

Figures 5.2 to 5.4 show the layout and construction programmes for the structural options for a commercial building that have been taken from *Commercial buildings - cost model study* [9].

The building is three storeys high and located on a green-field site, it has been assumed that one crane has been used for this construction.

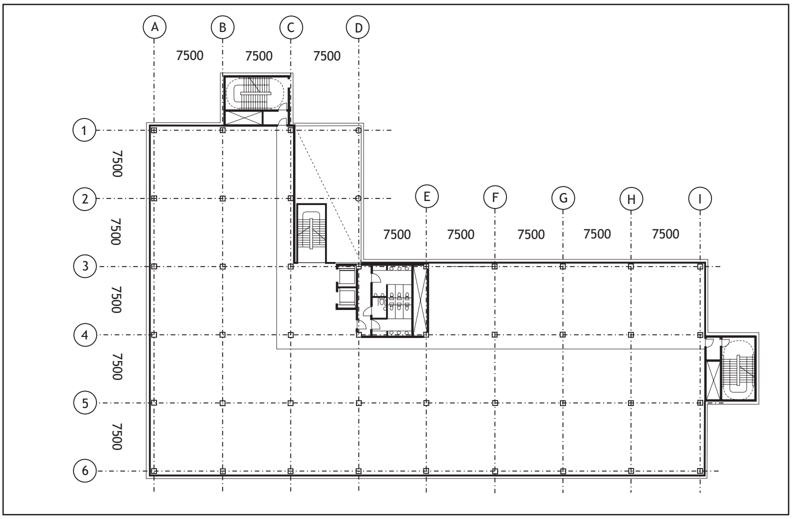


Figure 5.2
Plan of 3-storey commercial building

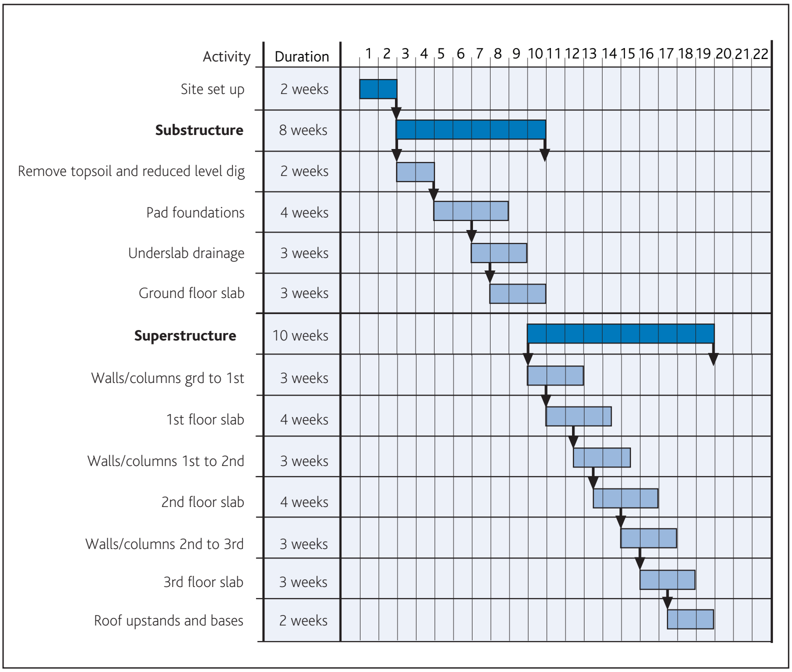


Figure 5.3
Construction programme for a flat slab solution

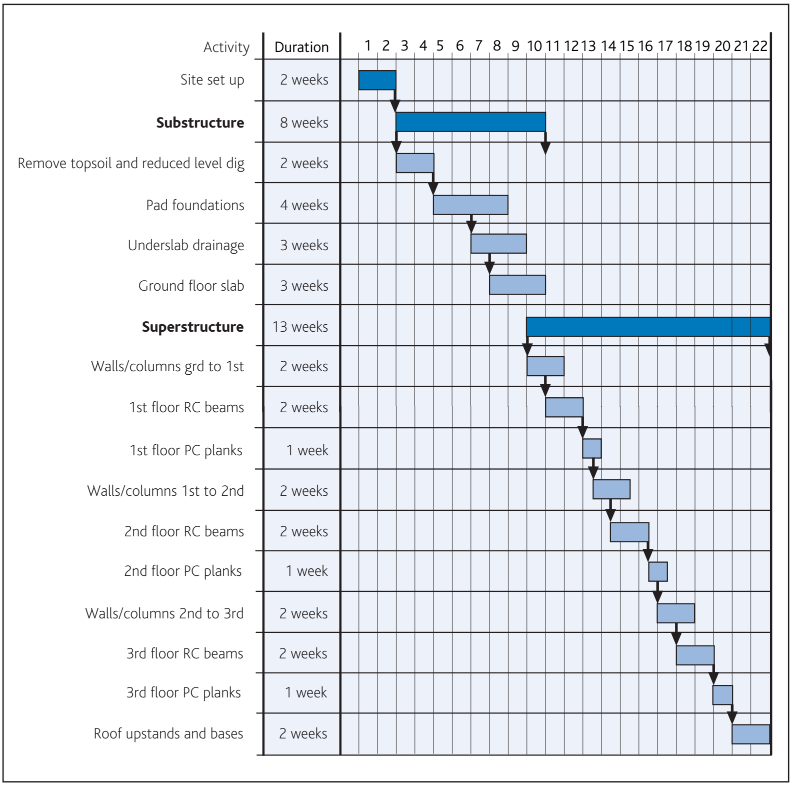


Figure 5.4
Construction programme for a hybrid concrete solution

Appendix A: Robustness requirements for precast concrete frames

The requirement to provide a robust precast concrete frame can be met by the deemed-to-satisfy method given in BS 8110 (Cl. 2.2.2.2(d)) of using a fully-tied solution. This method relies on empirical rules (Cl. 3.12.3) to provide suitable strength and ductility in the frame. The following ties should be provided (see also Figure A.1):

- Floor ties – connecting floors over an internal support.
- Perimeter floor ties – connecting floors to a perimeter support.
- Internal ties – over an intermediate support perpendicular to the span of the floor.
- Peripheral ties – around the perimeter of the floor.
- Vertical ties – connecting vertical walls or columns to provide continuity.

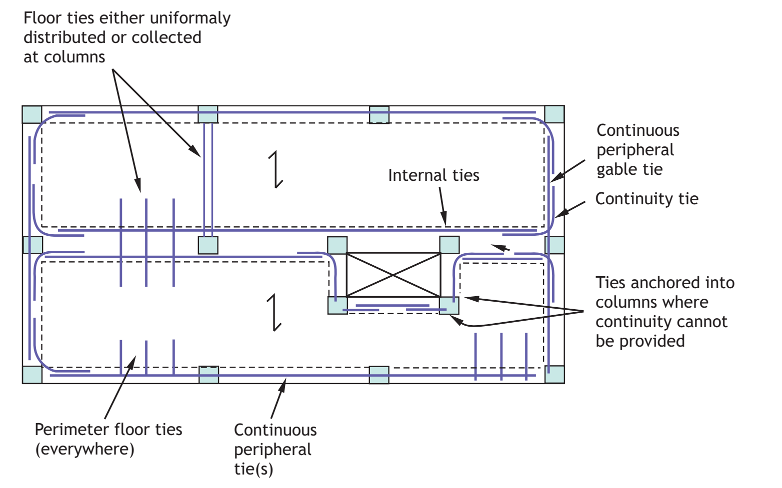


Figure A.1
Floor ties for a precast concrete frame [27]

The ties can be either reinforcement (e.g. H12) or helical prestressing strand (design strength 1580 N/mm^2), which is laid taught but not prestressed. The bars or strand should be adequately lapped and embedded in in-situ concrete with a minimum dimension of at least $\phi + 2H_{agg} + 10$ mm (i.e. usually at least 50 mm). 10 mm aggregate is often used to minimise the size of the concrete infill. The ties may be reinforcement already provided to serve another purpose and the design forces are in lieu of the normal design forces.

A.1 Horizontal ties

Typical horizontal tie details are shown in Figures A.2 to A.9. The following should be noted:

- The opening up of adjacent cores in hollowcore units should be avoided.
- The recommended maximum length of an open core in a hollowcore unit is 600 mm.

A.2 Vertical ties

Vertical ties should resist the ultimate load of the floor supported by the column at that level.

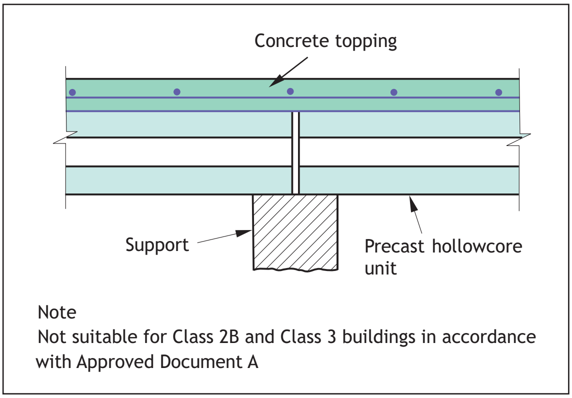


Figure A.2
Internal floor ties within concrete topping

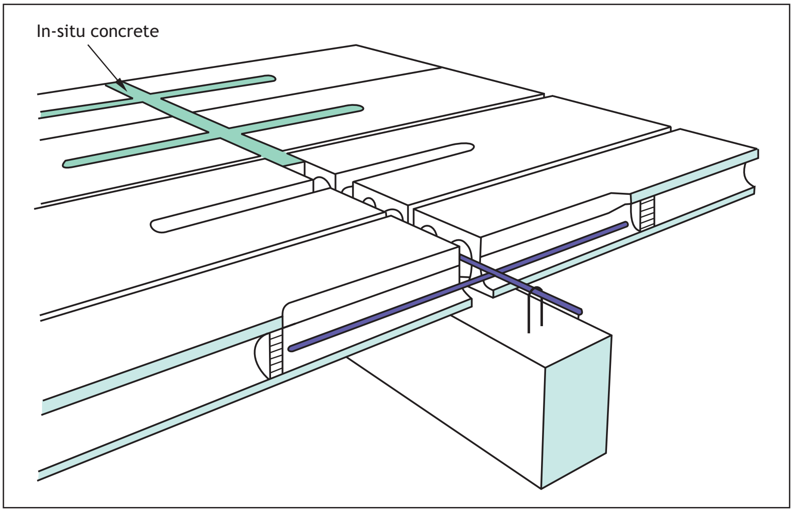


Figure A.3
Internal floor ties within hollowcore units [27]

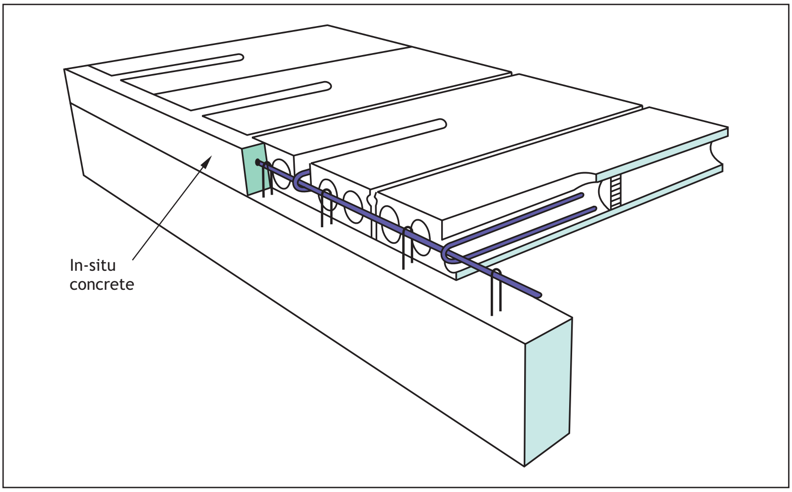


Figure A.4
Perimeter floor ties within hollowcore units [27]

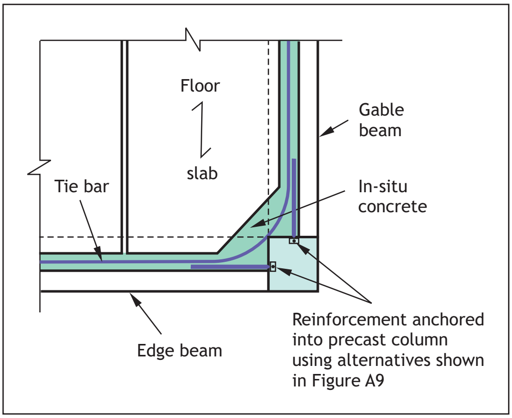


Figure A.5
Detailing at corner columns [27]

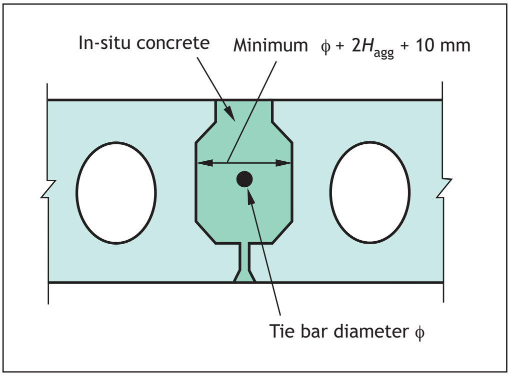


Figure A.6
Position of floor tie within longitudinal joints of hollowcore units

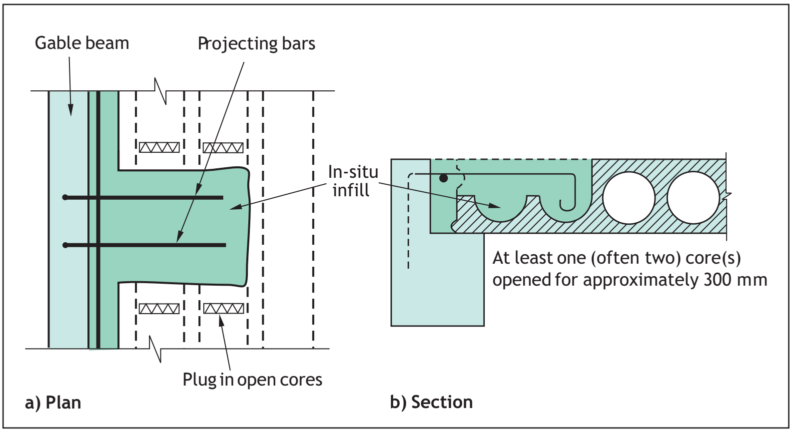


Figure A.7
Perimeter ties where hollowcore units span parallel to edge beam [27]

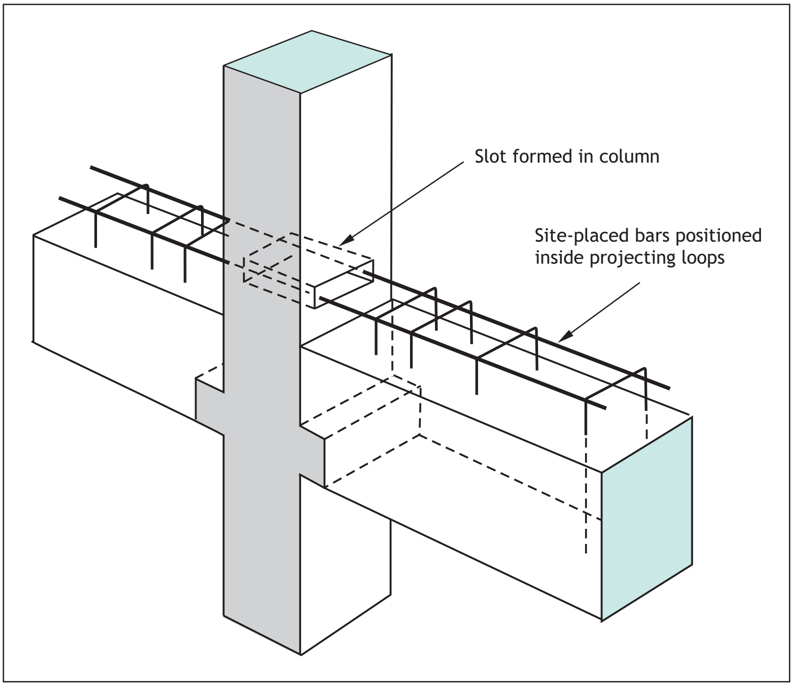


Figure A.8
Internal ties taken through precast column [27]

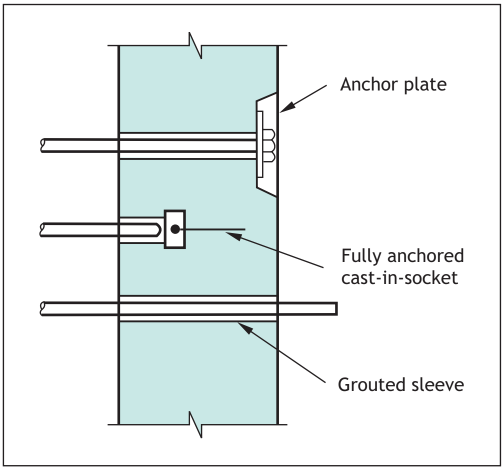


Figure A.9
Alternative arrangements for internal ties at precast column position

Appendix B: Selected tables from BS 8110

Table B1
Form and area of shear reinforcement in beams (table 3.7 of BS 8110)

Value of v (N/mm^2)	Form of shear reinforcement to be provided	Area of shear reinforcement to be provided
Less than $0.5v_c$ throughout the beam	See Note 1	—
$0.5v_c < v < (v_c + 0.4)$	Minimum links for whole length of beam	$A_{sv} \geq 0.4b_v s_v/0.87f_{yv}$ (see Note 2)
$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or 5 N/mm^2	Links or links combined with bent-up bars. Not more than 50% of the shear resistance provided by the steel may be in the form of bent-up bars (see Note 3)	Where links only provided: $A_{sv} \geq b_v s_v(v - v_c)/0.87f_{yv}$. Where links and bent-up bars provided: see Cl 3.4.5.6 of BS 8110

Notes

1 While minimum links should be provided in all beams of structural importance, it will be satisfactory to omit them in members of minor structural importance such as lintels or where the maximum design shear stress is less than half v_c.

2 Minimum links provide a design shear resistance of 0.4 N/mm^2.

3 See Cl.3.4.5.5 of BS 8110 for guidance on spacing of links and bent-up bars.

Table B2
Values of design concrete shear strength, v_c (N/mm^2) (table 3.8 of BS 8110)

$\frac{100A_s}{b_v d}$	Effective depth (mm)							
	125	150	175	200	225	250	300	400
≤ 0.15	0.45	0.43	0.41	0.40	0.39	0.38	0.36	0.34
0.25	0.53	0.51	0.49	0.47	0.46	0.45	0.43	0.40
0.50	0.67	0.64	0.62	0.60	0.58	0.56	0.54	0.50
0.75	0.77	0.73	0.71	0.68	0.66	0.65	0.62	0.57
1.00	0.84	0.81	0.78	0.75	0.73	0.71	0.68	0.63
1.50	0.97	0.92	0.89	0.86	0.83	0.81	0.78	0.72
2.00	1.06	1.02	0.98	0.95	0.92	0.89	0.86	0.80
≥ 3.00	1.22	1.16	1.12	1.08	1.05	1.02	0.98	0.91

For characteristic concrete strengths greater than 25 N/mm^2, the values in this table may be multiplied by $(f_{cu}/25)^{1/2}$. The value of f_{cu} should not be taken as greater than 40.

Table B3
Basic span/effective depth ratio for rectangular or flanged beams (table 3.9 of BS 8110)

Support conditions	Rectangular section	Flanged beams with $b_w/b \leq 0.3$
Cantilever	7	5.6
Simply supported	20	16.0
Continuous	26	20.8

Table B4
Modification factor for tension reinforcement (table 3.10 of BS 8110)

Service stress	M/bd^2								
	0.5	0.75	1.0	1.5	2.0	3.0	4.0	5.0	6.0
100	2.00	2.00	2.00	1.86	1.63	1.36	1.19	1.08	1.01
150	2.00	2.00	1.98	1.69	1.49	1.25	1.11	1.01	0.94
167 (f_y= 250)	2.00	2.00	1.91	1.63	1.44	1.21	1.08	0.99	0.92
200	2.00	1.95	1.76	1.51	1.35	1.14	1.02	0.94	0.88
250	1.90	1.70	1.55	1.34	1.20	1.04	0.94	0.87	0.82
300	1.60	1.44	1.33	1.16	1.06	0.93	0.85	0.80	0.76
333 (f_y= 500)	1.41	1.28	1.18	1.05	0.96	0.86	0.79	0.75	0.72

Note
The design service stress in the tension reinforcement in a member may be estimated from the equation:

$$f_s = \frac{2\, f_y\, A_{s,\ req}}{3\, A_{s,\ prov}\, \beta_b}$$

Table B5
Modification factor for compression reinforcement (table 3.11 of BS 8110)

$100\, A_{s,\ prov}/bd$	Factor
0	1.00
0.15	1.05
0.25	1.08
0.35	1.10
0.50	1.14
0.75	1.20
1.00	1.25
1.50	1.33
2.00	1.40
2.50	1.45
≥3.0	1.50

Table B6
Form and area of shear reinforcement in solid slabs (table 3.16 of BS 8110)

Value of v (N/mm^2)	Form of shear reinforcement to be provided	Area of shear reinforcement to be provided
$v < v_c$	None required	None
$v_c < v < (v_c + 0.4)$	Minimum links in areas where $v > v_c$	$A_{sv} \geq 0.4\, b\, s_v/0.87 f_{yv}$
$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or 5 N/mm^2	Links and/or bent-up bars in any combination (but the spacing between links or bent-up bars need not be less than d)	Where links only provided: $A_{sv} \geq b s_v(v - v_c)/0.87 f_{yv}$ Where bent-up bars only provided: $A_{sb} \geq b_{sb}(v - v_c)/\{0.87 f_{yv}\, (\cos\alpha + \sin\alpha \times \cot\beta)\}$ (see Cl 3.4.5.7 of BS 8110)

Note
In slabs less than 200 mm deep, it is difficult to bend and fix shear reinforcement so that its effectiveness can be assured. It is therefore not advisable to use shear reinforcement in such slabs.

Table B7
Minimum percentages of reinforcement (table 3.25 of BS 8110)

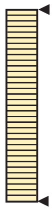
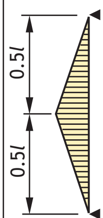
Situation	Definition of percentage	Minimum percentage	
		f_y = 250 N/mm^2	f_y = 500 N/mm^2
Tension reinforcement			
Sections subjected mainly to pure tension	$100A_s/A_c$	0.80	0.45
Sections subjected to flexure:			
Flanged beams, web in tension:			
$b_w/b < 0.4$	$100A_s/b_wh$	0.32	0.18
$b_w/b \geq 0.4$	$100A_s/b_wh$	0.24	0.13
Flanged beams, flange in tension:			
T-beam	$100A_s/b_wh$	0.48	0.26
L-beam	$100A_s/b_wh$	0.36	0.20
Rectangular section (in solid slabs this minimum should be provided in both directions)	$100A_s/A_c$	0.24	0.13
Compression reinforcement (where such reinforcement is required for the ultimate limit state)			
General rule	$100A_{sc}/A_{cc}$	0.40	0.40
Simplified rules for particular cases:			
Rectangular column or wall	$100A_{sc}/A_c$	0.40	0.40
Flanged beam:			
Flange in compression	$100A_{sc}/bh_f$	0.40	0.40
Web in compression	$100A_{sc}/b_wh$	0.20	0.20
Rectangular beam	$100A_{sc}/A_c$	0.20	0.20
Transverse reinforcement in flanges or flanged beams (provided over full effective flange width near top surface to resist horizontal shear)			
	$100A_{st}/h_f l$	0.15	0.15

Table B8
Clear distances between bars according to percentage redistribution (mm) (table 3.28 of BS 8110)

Specified characteristic strength of reinforcement, f_y	% redistribution to or from section considered						
	–30	–20	–10	0	10	20	30
250	200	225	255	280	300	300	300
500	110	125	140	155	170	185	200

Appendix C: Design aids

Table C.1
Design moment factors for continuous beams with approximately equal spans

Load	Dead load (all spans loaded)	Imposed load (maximum of alternate spans loaded or all spans loaded)
Uniformly distributed 	0.125 0.070 0.070	0.125 0.096 0.096
	0.100 0.100 0.080 0.025 0.080	0.117 0.117 0.101 0.075 0.101
	0.107 0.071 0.107 0.077 0.036 0.036 0.077	0.121 0.107 0.121 0.099 0.081 0.081 0.099
	0.105 0.079 0.079 0.105 0.078 0.033 0.046 0.033 0.078	0.120 0.111 0.111 0.120 0.100 0.079 0.086 0.079 0.100
	0.156 0.095 0.095	0.156 0.129 0.129
	0.125 0.125 0.108 0.042 0.108	0.146 0.146 0.136 0.104 0.136
	0.134 0.089 0.134 0.104 0.056 0.056 0.104	0.151 0.134 0.151 0.134 0.111 0.111 0.134
	0.132 0.099 0.099 0.132 0.105 0.051 0.068 0.051 0.105	0.150 0.139 0.139 0.150 0.135 0.109 0.117 0.109 0.135
Concentrated at mid span 	0.188 0.156 0.156	0.188 0.203 0.203
	0.150 0.150 0.175 0.100 0.175	0.175 0.175 0.213 0.175 0.213
	0.161 0.107 0.161 0.170 0.116 0.116 0.170	0.181 0.161 0.181 0.210 0.183 0.183 0.210
	0.158 0.118 0.118 0.158 0.171 0.112 0.132 0.112 0.171	0.179 0.167 0.167 0.179 0.211 0.181 0.191 0.181 0.211
Concentrated at third points 	0.167 0.111 0.111	0.167 0.139 0.139
	0.133 0.133 0.122 0.033 0.122	0.156 0.156 0.144 0.100 0.144
	0.143 0.095 0.143 0.119 0.056 0.056 0.119	0.160 0.144 0.160 0.143 0.111 0.111 0.143
	0.140 0.105 0.105 0.140 0.120 0.050 0.061 0.050 0.120	0.159 0.148 0.148 0.159 0.144 0.108 0.115 0.108 0.144

Notes

1 Bending moment = (coefficient) x (total load on span) x (span).

2 Bending moment coefficient:

A above line apply to negative bending moment at supports.

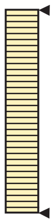
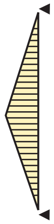
B below line apply to positive bending moment in span.

3 Coefficients apply when all spans are equal (variations in the span length must not exceed 15% of the longest).

4 Loads on each loaded span are equal.

5 Moment of inertia same throughout all spans.

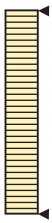
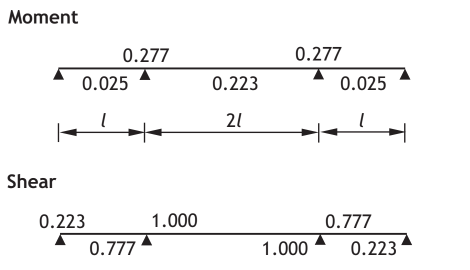
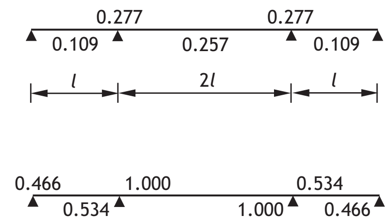
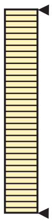
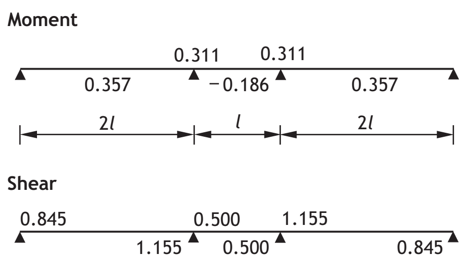
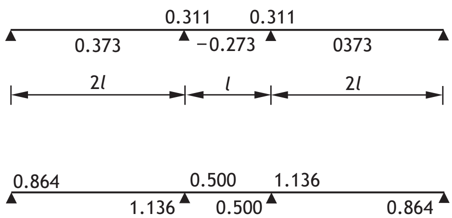
Table C.2
Design shear force factors for continuous beams with approximately equal spans

Load	Dead load (all spans loaded)	Imposed load (maximum of alternate spans loaded or all spans loaded)
Uniformly distributed 	0.375 0.625 0.625 0.375	0.438 0.625 0.625 0.438
	0.400 0.500 0.600 0.600 0.500 0.400	0.450 0.583 0.617 0.617 0.583 0.450
	0.393 0.536 0.464 0.607 0.607 0.464 0.536 0.393	0.446 0.603 0.571 0.621 0.621 0.571 0.603 0.446
	0.395 0.526 0.500 0.474 0.605 0.605 0.474 0.500 0.526 0.395	0.447 0.598 0.591 0.576 0.620 0.620 0.576 0.591 0.598 0.447
Triangularly distributed 	0.344 0.656 0.656 0.344	0.422 0.656 0.656 0.422
	0.375 0.500 0.625 0.625 0.500 0.375	0.437 0.605 0.646 0.646 0.605 0.437
	0.366 0.545 0.455 0.634 0.634 0.455 0.545 0.366	0.433 0.628 0.589 0.651 0.651 0.589 0.628 0.433
	0.369 0.532 0.500 0.468 0.631 0.631 0.468 0.500 0.532 0.369	0.434 0.622 0.614 0.595 0.649 0.649 0.595 0.614 0.622 0.434
Concentrated at midspan 	0.313 0.688 0.688 0.313	0.406 0.688 0.688 0.406
	0.350 0.500 0.650 0.650 0.500 0.350	0.425 0.625 0.675 0.675 0.625 0.425
	0.339 0.554 0.446 0.661 0.661 0.446 0.554 0.339	0.420 0.654 0.607 0.681 0.681 0.607 0.654 0.420
	0.342 0.540 0.500 0.460 0.658 0.658 0.460 0.500 0.540 0.342	0.421 0.647 0.636 0.615 0.679 0.679 0.615 0.636 0.647 0.421
Concentrated at third points 	0.333 0.667 0.667 0.333	0.417 0.667 0.667 0.417
	0.367 0.500 0.633 0.633 0.500 0.367	0.433 0.611 0.656 0.656 0.611 0.433
	0.357 0.548 0.452 0.643 0.643 0.452 0.548 0.357	0.429 0.637 0.595 0.661 0.661 0.595 0.637 0.429
	0.360 0.535 0.500 0.465 0.640 0.640 0.465 0.500 0.535 0.360	0.430 0.631 0.621 0.602 0.659 0.659 0.602 0.621 0.631 0.480

Note
Shear force = (coefficient) x (total load on one span).

Table C.3

Design moment and shear force factors for spans of unequal length

Load	Dead load (all spans loaded)	Imposed load (maximum of alternate spans loaded or all spans loaded)
Uniformly distributed 		
Uniformly distributed 		

Notes

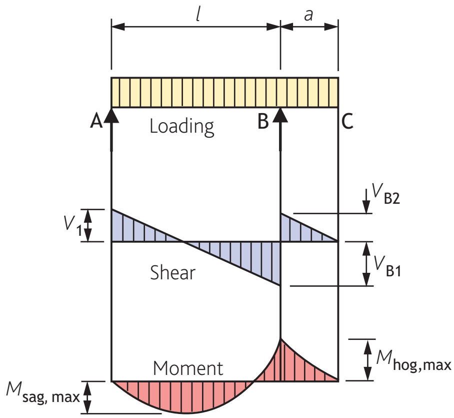
1 Bending moment = (coefficient) x (load per unit length) x (l^2).

2 Shear force = (coefficient) x (load per unit length) x (l).

3 Bending moment coefficient:

A above line apply to negative bending moment at supports.

B below line apply to positive bending moment in span (negative value indicates negative bending moment).



$$R_A = V_A = \frac{w}{2l}(l^2 - a^2)$$

$$R_B = V_{B1} + V_{B2} = \frac{w}{2l}(l + a)^2$$

$$V_{B1} = wa$$

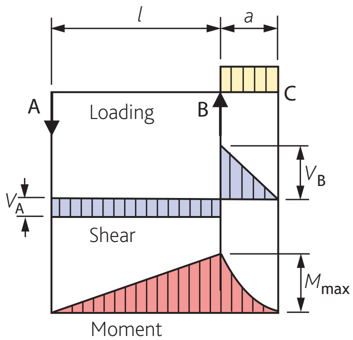
$$V_{B2} = \frac{w}{2l}(l^2 + a^2)$$

$$M_{sag,\,max} = \frac{w}{8l^2}(l + a)^2(l - a)^2$$

$$M_{hog,\,max} = \frac{wa^2}{2}$$

$$\delta_C = \frac{wa}{24EI}(4a^2 l - l^3 - 3a^3)$$

a) Beam overhanging one support - uniformly distributed load



$$R_A = V_A = \frac{wa^2}{2l}$$

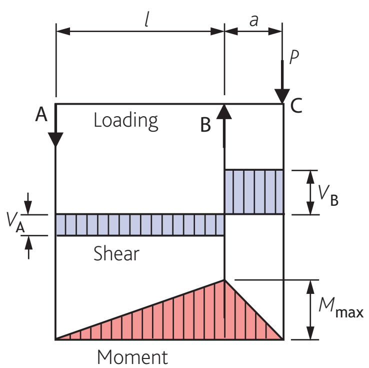
$$R_B = V_A + V_B = \frac{wa}{2l}(2l + a)$$

$$V_B = wa$$

$$M_{max} = \frac{wa^2}{2}$$

$$\delta_C = \frac{wa^3}{24EI}(4l + 3a)$$

b) Beam overhanging one support - uniformly distributed load on overhang



$$R_A = V_A = \frac{Pa}{l}$$

$$R_B = V_A + V_B = \frac{P}{l}(l + a)$$

$$V_B = P$$

$$M_{max} = Pa$$

$$\delta_C = \frac{Pa^2}{3EI}(l + a)$$

c) Beam overhanging one support - concentrated load at end of overhang

Figure C.1
Bending moments, shear forces and deflections for cantilevers

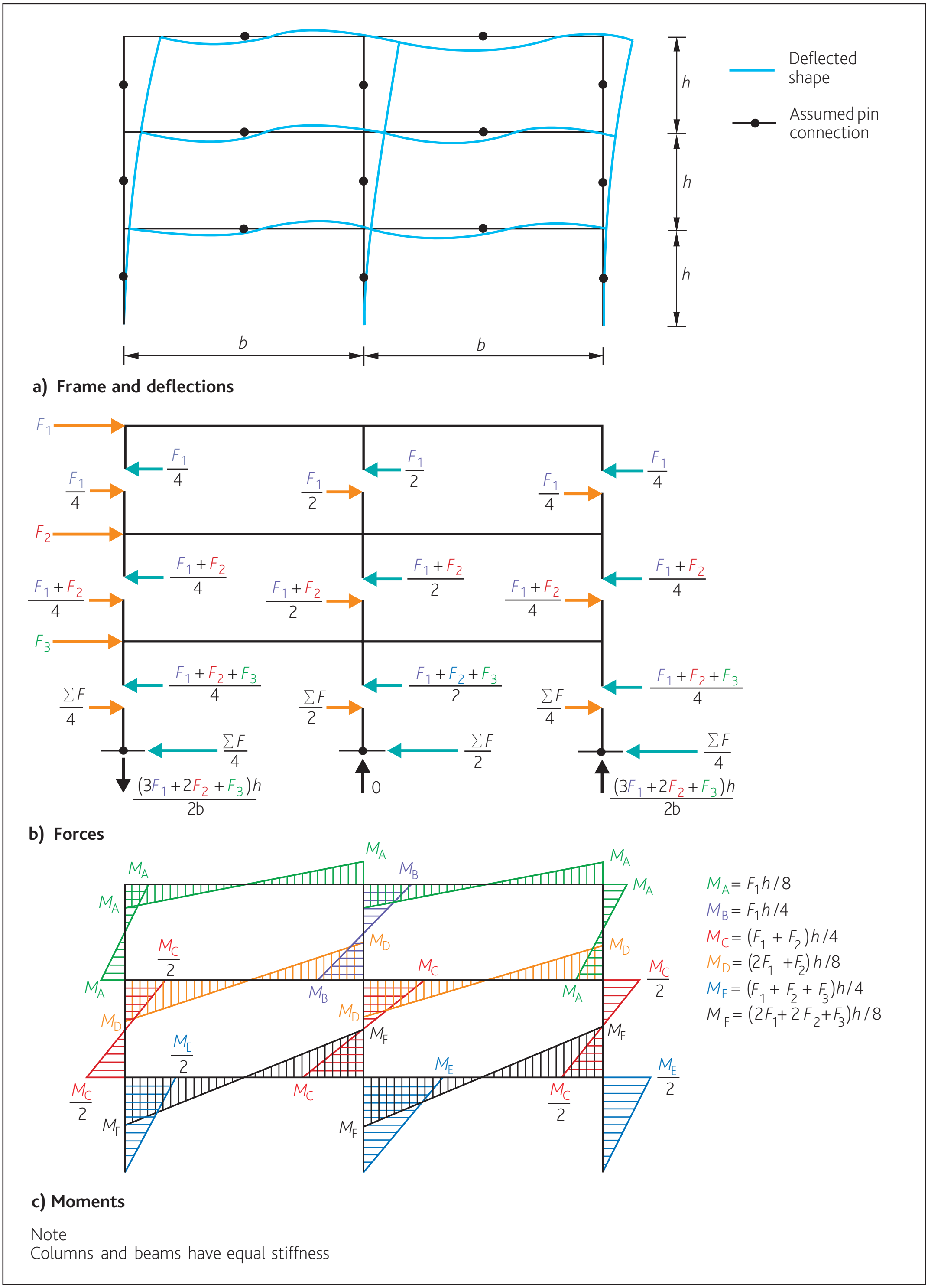


Figure C.2
Approximate moments for a multi-storey frame

Table C.4
Values of design concrete shear stress, v_c, for f_{cu} = 30 N/mm^2

$\frac{100 A_s}{b_v d}$	Effective depth (mm)											
	125	150	175	200	225	250	275	300	325	350	375	400
0.15	0.48	0.46	0.44	0.42	0.41	0.40	0.39	0.38	0.38	0.37	0.36	0.36
0.20	0.53	0.50	0.48	0.47	0.45	0.44	0.43	0.42	0.41	0.41	0.40	0.39
0.25	0.57	0.54	0.52	0.50	0.49	0.48	0.46	0.45	0.45	0.44	0.43	0.42
0.30	0.60	0.57	0.55	0.53	0.52	0.51	0.49	0.48	0.47	0.46	0.46	0.45
0.35	0.63	0.60	0.58	0.56	0.55	0.53	0.52	0.51	0.50	0.49	0.48	0.47
0.40	0.66	0.63	0.61	0.59	0.57	0.56	0.54	0.53	0.52	0.51	0.50	0.49
0.45	0.69	0.66	0.63	0.61	0.59	0.58	0.57	0.55	0.54	0.53	0.52	0.51
0.50	0.71	0.68	0.66	0.63	0.62	0.60	0.59	0.57	0.56	0.55	0.54	0.53
0.55	0.74	0.70	0.68	0.65	0.64	0.62	0.60	0.59	0.58	0.57	0.56	0.55
0.60	0.76	0.72	0.70	0.67	0.65	0.64	0.62	0.61	0.60	0.59	0.58	0.57
0.65	0.78	0.74	0.72	0.69	0.67	0.65	0.64	0.63	0.61	0.60	0.59	0.58
0.70	0.80	0.76	0.73	0.71	0.69	0.67	0.65	0.64	0.63	0.62	0.61	0.60
0.75	0.82	0.78	0.75	0.73	0.70	0.69	0.67	0.66	0.64	0.63	0.62	0.61
0.80	0.83	0.80	0.77	0.74	0.72	0.70	0.68	0.67	0.66	0.64	0.63	0.62
0.85	0.85	0.81	0.78	0.76	0.73	0.72	0.70	0.68	0.67	0.66	0.65	0.64
0.90	0.87	0.83	0.80	0.77	0.75	0.73	0.71	0.70	0.68	0.67	0.66	0.65
0.95	0.88	0.84	0.81	0.79	0.76	0.74	0.73	0.71	0.70	0.68	0.67	0.66
1.00	0.90	0.86	0.83	0.80	0.78	0.76	0.74	0.72	0.71	0.69	0.68	0.67
1.05	0.91	0.87	0.84	0.81	0.79	0.77	0.75	0.73	0.72	0.71	0.69	0.68
1.10	0.93	0.89	0.85	0.82	0.80	0.78	0.76	0.74	0.73	0.72	0.70	0.69
1.15	0.94	0.90	0.87	0.84	0.81	0.79	0.77	0.76	0.74	0.73	0.72	0.70
1.20	0.95	0.91	0.88	0.85	0.82	0.80	0.78	0.77	0.75	0.74	0.73	0.71
1.25	0.97	0.92	0.89	0.86	0.84	0.81	0.79	0.78	0.76	0.75	0.74	0.72
1.30	0.98	0.94	0.90	0.87	0.85	0.82	0.80	0.79	0.77	0.76	0.74	0.73
1.35	0.99	0.95	0.91	0.88	0.86	0.83	0.82	0.80	0.78	0.77	0.75	0.74
1.40	1.00	0.96	0.92	0.89	0.87	0.84	0.83	0.81	0.79	0.78	0.76	0.75
1.45	1.02	0.97	0.93	0.90	0.88	0.85	0.83	0.82	0.80	0.79	0.77	0.76
1.50	1.03	0.98	0.95	0.91	0.89	0.86	0.84	0.83	0.81	0.79	0.78	0.77
1.60	1.05	1.00	0.97	0.93	0.91	0.88	0.86	0.84	0.83	0.81	0.80	0.79
1.70	1.07	1.02	0.99	0.95	0.93	0.90	0.88	0.86	0.84	0.83	0.81	0.80
1.80	1.09	1.04	1.00	0.97	0.94	0.92	0.90	0.88	0.86	0.84	0.83	0.82
1.90	1.11	1.06	1.02	0.99	0.96	0.94	0.91	0.89	0.88	0.86	0.85	0.83
2.00	1.13	1.08	1.04	1.01	0.98	0.95	0.93	0.91	0.89	0.87	0.86	0.85
2.10	1.15	1.10	1.06	1.02	0.99	0.97	0.94	0.92	0.91	0.89	0.87	0.86
2.20	1.17	1.12	1.07	1.04	1.01	0.98	0.96	0.94	0.92	0.90	0.89	0.87
2.30	1.19	1.13	1.09	1.05	1.02	1.00	0.97	0.95	0.93	0.92	0.90	0.89
2.40	1.20	1.15	1.11	1.07	1.04	1.01	0.99	0.97	0.95	0.93	0.91	0.90
2.50	1.22	1.16	1.12	1.08	1.05	1.03	1.00	0.98	0.96	0.94	0.93	0.91
2.60	1.24	1.18	1.14	1.10	1.07	1.04	1.01	0.99	0.97	0.95	0.94	0.92
2.70	1.25	1.20	1.15	1.11	1.08	1.05	1.03	1.00	0.99	0.97	0.95	0.94
2.80	1.27	1.21	1.16	1.13	1.09	1.06	1.04	1.02	1.00	0.98	0.96	0.95
2.90	1.28	1.22	1.18	1.14	1.11	1.08	1.05	1.03	1.01	0.99	0.97	0.96
3.00	1.30	1.24	1.19	1.15	1.12	1.09	1.06	1.04	1.02	1.00	0.98	0.97

Table C.5

Values of design concrete shear stress, v_c, for f_{cu} = 35 N/mm^2

$\frac{100 A_s}{b_v d}$	Effective depth (mm)											
	125	150	175	200	225	250	275	300	325	350	375	400
0.15	0.50	0.48	0.46	0.45	0.43	0.42	0.41	0.40	0.40	0.39	0.38	0.38
0.20	0.55	0.53	0.51	0.49	0.48	0.47	0.45	0.44	0.44	0.43	0.42	0.41
0.25	0.60	0.57	0.55	0.53	0.51	0.50	0.49	0.48	0.47	0.46	0.45	0.45
0.30	0.63	0.60	0.58	0.56	0.55	0.53	0.52	0.51	0.50	0.49	0.48	0.47
0.35	0.67	0.64	0.61	0.59	0.58	0.56	0.55	0.54	0.52	0.52	0.51	0.50
0.40	0.70	0.67	0.64	0.62	0.60	0.59	0.57	0.56	0.55	0.54	0.53	0.52
0.45	0.72	0.69	0.67	0.64	0.63	0.61	0.59	0.58	0.57	0.56	0.55	0.54
0.50	0.75	0.72	0.69	0.67	0.65	0.63	0.62	0.60	0.59	0.58	0.57	0.56
0.55	0.77	0.74	0.71	0.69	0.67	0.65	0.64	0.62	0.61	0.60	0.59	0.58
0.60	0.80	0.76	0.73	0.71	0.69	0.67	0.65	0.64	0.63	0.62	0.61	0.60
0.65	0.82	0.78	0.75	0.73	0.71	0.69	0.67	0.66	0.65	0.63	0.62	0.61
0.70	0.84	0.80	0.77	0.75	0.72	0.71	0.69	0.67	0.66	0.65	0.64	0.63
0.75	0.86	0.82	0.79	0.76	0.74	0.72	0.71	0.69	0.68	0.66	0.65	0.64
0.80	0.88	0.84	0.81	0.78	0.76	0.74	0.72	0.71	0.69	0.68	0.67	0.66
0.85	0.90	0.86	0.82	0.80	0.77	0.75	0.74	0.72	0.71	0.69	0.68	0.67
0.90	0.91	0.87	0.84	0.81	0.79	0.77	0.75	0.73	0.72	0.71	0.69	0.68
0.95	0.93	0.89	0.85	0.83	0.80	0.78	0.76	0.75	0.73	0.72	0.71	0.70
1.00	0.95	0.90	0.87	0.84	0.82	0.80	0.78	0.76	0.74	0.73	0.72	0.71
1.05	0.96	0.92	0.88	0.85	0.83	0.81	0.79	0.77	0.76	0.74	0.73	0.72
1.10	0.98	0.93	0.90	0.87	0.84	0.82	0.80	0.78	0.77	0.75	0.74	0.73
1.15	0.99	0.95	0.91	0.88	0.86	0.83	0.81	0.80	0.78	0.77	0.75	0.74
1.20	1.00	0.96	0.92	0.89	0.87	0.84	0.83	0.81	0.79	0.78	0.76	0.75
1.25	1.02	0.97	0.94	0.91	0.88	0.86	0.84	0.82	0.80	0.79	0.77	0.76
1.30	1.03	0.99	0.95	0.92	0.89	0.87	0.85	0.83	0.81	0.80	0.78	0.77
1.35	1.05	1.00	0.96	0.93	0.90	0.88	0.86	0.84	0.82	0.81	0.79	0.78
1.40	1.06	1.01	0.97	0.94	0.91	0.89	0.87	0.85	0.83	0.82	0.80	0.79
1.45	1.07	1.02	0.98	0.95	0.92	0.90	0.88	0.86	0.84	0.83	0.81	0.80
1.50	1.08	1.03	1.00	0.96	0.93	0.91	0.89	0.87	0.85	0.84	0.82	0.81
1.60	1.11	1.06	1.02	0.98	0.95	0.93	0.91	0.89	0.87	0.85	0.84	0.83
1.70	1.13	1.08	1.04	1.00	0.97	0.95	0.93	0.91	0.89	0.87	0.86	0.84
1.80	1.15	1.10	1.06	1.02	0.99	0.97	0.94	0.92	0.91	0.89	0.87	0.86
1.90	1.17	1.12	1.08	1.04	1.01	0.98	0.96	0.94	0.92	0.91	0.89	0.88
2.00	1.19	1.14	1.10	1.06	1.03	1.00	0.98	0.96	0.94	0.92	0.91	0.89
2.10	1.21	1.16	1.11	1.08	1.05	1.02	0.99	0.97	0.95	0.94	0.92	0.91
2.20	1.23	1.18	1.13	1.09	1.06	1.03	1.01	0.99	0.97	0.95	0.93	0.92
2.30	1.25	1.19	1.15	1.11	1.08	1.05	1.02	1.00	0.98	0.96	0.95	0.93
2.40	1.27	1.21	1.16	1.13	1.09	1.06	1.04	1.02	1.00	0.98	0.96	0.95
2.50	1.28	1.23	1.18	1.14	1.11	1.08	1.05	1.03	1.01	0.99	0.98	0.96
2.60	1.30	1.24	1.20	1.16	1.12	1.09	1.07	1.04	1.02	1.01	0.99	0.97
2.70	1.32	1.26	1.21	1.17	1.14	1.11	1.08	1.06	1.04	1.02	1.00	0.98
2.80	1.33	1.27	1.23	1.19	1.15	1.12	1.09	1.07	1.05	1.03	1.01	1.00
2.90	1.35	1.29	1.24	1.20	1.16	1.13	1.11	1.08	1.06	1.04	1.02	1.01
3.00	1.36	1.30	1.25	1.21	1.18	1.15	1.12	1.10	1.07	1.05	1.04	1.02

Table C.6
Values of design concrete shear stress, v_c, for f_{cu} = 40 N/mm^2

$\frac{100 A_s}{b_v d}$	Effective depth (mm)											
	125	150	175	200	225	250	275	300	325	350	375	400
0.15	0.53	0.50	0.48	0.47	0.45	0.44	0.43	0.42	0.41	0.41	0.40	0.39
0.20	0.58	0.55	0.53	0.51	0.50	0.49	0.47	0.46	0.46	0.45	0.44	0.43
0.25	0.62	0.60	0.57	0.55	0.54	0.52	0.51	0.50	0.49	0.48	0.47	0.47
0.30	0.66	0.63	0.61	0.59	0.57	0.56	0.54	0.53	0.52	0.51	0.50	0.49
0.35	0.70	0.67	0.64	0.62	0.60	0.59	0.57	0.56	0.55	0.54	0.53	0.52
0.40	0.73	0.70	0.67	0.65	0.63	0.61	0.60	0.59	0.57	0.56	0.55	0.54
0.45	0.76	0.72	0.70	0.67	0.65	0.64	0.62	0.61	0.60	0.59	0.58	0.57
0.50	0.78	0.75	0.72	0.70	0.68	0.66	0.64	0.63	0.62	0.61	0.60	0.59
0.55	0.81	0.77	0.74	0.72	0.70	0.68	0.67	0.65	0.64	0.63	0.62	0.61
0.60	0.83	0.80	0.77	0.74	0.72	0.70	0.68	0.67	0.66	0.64	0.63	0.62
0.65	0.86	0.82	0.79	0.76	0.74	0.72	0.70	0.69	0.67	0.66	0.65	0.64
0.70	0.88	0.84	0.81	0.78	0.76	0.74	0.72	0.71	0.69	0.68	0.67	0.66
0.75	0.90	0.86	0.83	0.80	0.78	0.76	0.74	0.72	0.71	0.69	0.68	0.67
0.80	0.92	0.88	0.84	0.82	0.79	0.77	0.75	0.74	0.72	0.71	0.70	0.69
0.85	0.94	0.89	0.86	0.83	0.81	0.79	0.77	0.75	0.74	0.72	0.71	0.70
0.90	0.95	0.91	0.88	0.85	0.82	0.80	0.78	0.77	0.75	0.74	0.73	0.71
0.95	0.97	0.93	0.89	0.86	0.84	0.82	0.80	0.78	0.77	0.75	0.74	0.73
1.00	0.99	0.94	0.91	0.88	0.85	0.83	0.81	0.79	0.78	0.76	0.75	0.74
1.05	1.00	0.96	0.92	0.89	0.87	0.84	0.83	0.81	0.79	0.78	0.76	0.75
1.10	1.02	0.98	0.94	0.91	0.88	0.86	0.84	0.82	0.80	0.79	0.78	0.76
1.15	1.04	0.99	0.95	0.92	0.89	0.87	0.85	0.83	0.82	0.80	0.79	0.77
1.20	1.05	1.00	0.97	0.93	0.91	0.88	0.86	0.84	0.83	0.81	0.80	0.79
1.25	1.06	1.02	0.98	0.95	0.92	0.90	0.87	0.86	0.84	0.82	0.81	0.80
1.30	1.08	1.03	0.99	0.96	0.93	0.91	0.89	0.87	0.85	0.83	0.82	0.81
1.35	1.09	1.04	1.00	0.97	0.94	0.92	0.90	0.88	0.86	0.84	0.83	0.82
1.40	1.11	1.06	1.02	0.98	0.95	0.93	0.91	0.89	0.87	0.85	0.84	0.83
1.45	1.12	1.07	1.03	0.99	0.97	0.94	0.92	0.90	0.88	0.87	0.85	0.84
1.50	1.13	1.08	1.04	1.01	0.98	0.95	0.93	0.91	0.89	0.87	0.86	0.85
1.60	1.16	1.10	1.06	1.03	1.00	0.97	0.95	0.93	0.91	0.89	0.88	0.86
1.70	1.18	1.13	1.08	1.05	1.02	0.99	0.97	0.95	0.93	0.91	0.90	0.88
1.80	1.20	1.15	1.11	1.07	1.04	1.01	0.99	0.97	0.95	0.93	0.91	0.90
1.90	1.22	1.17	1.13	1.09	1.06	1.03	1.01	0.98	0.96	0.95	0.93	0.92
2.00	1.25	1.19	1.15	1.11	1.08	1.05	1.02	1.00	0.98	0.96	0.95	0.93
2.10	1.27	1.21	1.16	1.13	1.09	1.06	1.04	1.02	1.00	0.98	0.96	0.95
2.20	1.29	1.23	1.18	1.14	1.11	1.08	1.06	1.03	1.01	0.99	0.98	0.96
2.30	1.31	1.25	1.20	1.16	1.13	1.10	1.07	1.05	1.03	1.01	0.99	0.98
2.40	1.32	1.26	1.22	1.18	1.14	1.11	1.09	1.06	1.04	1.02	1.01	0.99
2.50	1.34	1.28	1.23	1.19	1.16	1.13	1.10	1.08	1.06	1.04	1.02	1.00
2.60	1.36	1.30	1.25	1.21	1.17	1.14	1.12	1.09	1.07	1.05	1.03	1.02
2.70	1.38	1.32	1.27	1.22	1.19	1.16	1.13	1.11	1.08	1.06	1.05	1.03
2.80	1.39	1.33	1.28	1.24	1.20	1.17	1.14	1.12	1.10	1.08	1.06	1.04
2.90	1.41	1.35	1.30	1.25	1.22	1.19	1.16	1.13	1.11	1.09	1.07	1.05
3.00	1.43	1.36	1.31	1.27	1.23	1.20	1.17	1.15	1.12	1.10	1.08	1.07

Table C.7
Modification factor for tension reinforcement

f_s	M/bd^2																	
	0.50	0.60	0.70	0.80	0.90	1.00	1.10	1.20	1.30	1.40	1.50	1.60	1.70	1.80	1.90	2.00	2.20	2.40
100	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.98	1.92	1.86	1.81	1.76	1.71	1.67	1.63	1.56	1.50
110	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.94	1.88	1.82	1.77	1.73	1.68	1.64	1.60	1.54	1.48
120	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.97	1.90	1.84	1.79	1.74	1.69	1.65	1.61	1.58	1.51	1.45
130	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.93	1.86	1.81	1.75	1.71	1.66	1.62	1.58	1.55	1.48	1.43
140	2.00	2.00	2.00	2.00	2.00	2.00	1.95	1.89	1.83	1.77	1.72	1.67	1.63	1.59	1.55	1.52	1.46	1.40
145	2.00	2.00	2.00	2.00	2.00	2.00	1.93	1.87	1.81	1.75	1.70	1.66	1.61	1.57	1.54	1.50	1.44	1.39
150	2.00	2.00	2.00	2.00	2.00	1.98	1.91	1.85	1.79	1.73	1.69	1.64	1.60	1.56	1.52	1.49	1.43	1.38
155	2.00	2.00	2.00	2.00	2.00	1.96	1.89	1.83	1.77	1.72	1.67	1.62	1.58	1.54	1.51	1.48	1.42	1.36
160	2.00	2.00	2.00	2.00	2.00	1.94	1.87	1.81	1.75	1.70	1.65	1.61	1.57	1.53	1.49	1.46	1.40	1.35
165	2.00	2.00	2.00	2.00	1.99	1.92	1.85	1.79	1.73	1.68	1.63	1.59	1.55	1.51	1.48	1.45	1.39	1.34
170	2.00	2.00	2.00	2.00	1.97	1.90	1.83	1.77	1.71	1.66	1.62	1.57	1.53	1.50	1.46	1.43	1.38	1.33
175	2.00	2.00	2.00	2.00	1.95	1.87	1.81	1.75	1.69	1.64	1.60	1.56	1.52	1.48	1.45	1.42	1.36	1.31
180	2.00	2.00	2.00	2.00	1.93	1.85	1.79	1.73	1.68	1.63	1.58	1.54	1.50	1.47	1.43	1.40	1.35	1.30
185	2.00	2.00	2.00	1.98	1.90	1.83	1.77	1.71	1.66	1.61	1.56	1.52	1.49	1.45	1.42	1.39	1.33	1.29
190	2.00	2.00	2.00	1.96	1.88	1.81	1.75	1.69	1.64	1.59	1.55	1.51	1.47	1.44	1.40	1.37	1.32	1.27
195	2.00	2.00	2.00	1.93	1.86	1.79	1.73	1.67	1.62	1.57	1.53	1.49	1.45	1.42	1.39	1.36	1.31	1.26
200	2.00	2.00	1.99	1.91	1.83	1.76	1.70	1.65	1.60	1.55	1.51	1.47	1.44	1.40	1.37	1.35	1.29	1.25
205	2.00	2.00	1.97	1.88	1.81	1.74	1.68	1.63	1.58	1.54	1.49	1.46	1.42	1.39	1.36	1.33	1.28	1.24
210	2.00	2.00	1.94	1.86	1.79	1.72	1.66	1.61	1.56	1.52	1.48	1.44	1.41	1.37	1.34	1.32	1.27	1.22
215	2.00	2.00	1.91	1.83	1.76	1.70	1.64	1.59	1.54	1.50	1.46	1.42	1.39	1.36	1.33	1.30	1.25	1.21
220	2.00	1.98	1.89	1.81	1.74	1.68	1.62	1.57	1.52	1.48	1.44	1.41	1.37	1.34	1.31	1.29	1.24	1.20
225	2.00	1.95	1.86	1.79	1.72	1.66	1.60	1.55	1.50	1.46	1.43	1.39	1.36	1.33	1.30	1.27	1.23	1.19
230	2.00	1.92	1.84	1.76	1.69	1.63	1.58	1.53	1.49	1.44	1.41	1.37	1.34	1.31	1.29	1.26	1.21	1.17
235	1.99	1.89	1.81	1.74	1.67	1.61	1.56	1.51	1.47	1.43	1.39	1.36	1.33	1.30	1.27	1.25	1.20	1.16
240	1.96	1.87	1.78	1.71	1.65	1.59	1.54	1.49	1.45	1.41	1.37	1.34	1.31	1.28	1.26	1.23	1.19	1.15
245	1.93	1.84	1.76	1.69	1.62	1.57	1.52	1.47	1.43	1.39	1.36	1.32	1.29	1.27	1.24	1.22	1.17	1.14
250	1.90	1.81	1.73	1.66	1.60	1.55	1.50	1.45	1.41	1.37	1.34	1.31	1.28	1.25	1.23	1.20	1.16	1.12
255	1.87	1.78	1.71	1.64	1.58	1.52	1.48	1.43	1.39	1.35	1.32	1.29	1.26	1.24	1.21	1.19	1.15	1.11
260	1.84	1.76	1.68	1.61	1.55	1.50	1.45	1.41	1.37	1.34	1.30	1.27	1.25	1.22	1.20	1.17	1.13	1.10
265	1.81	1.73	1.65	1.59	1.53	1.48	1.43	1.39	1.35	1.32	1.29	1.26	1.23	1.20	1.18	1.16	1.12	1.09
270	1.78	1.70	1.63	1.56	1.51	1.46	1.41	1.37	1.33	1.30	1.27	1.24	1.21	1.19	1.17	1.14	1.11	1.07
275	1.75	1.67	1.60	1.54	1.49	1.44	1.39	1.35	1.32	1.28	1.25	1.22	1.20	1.17	1.15	1.13	1.09	1.06
280	1.72	1.64	1.58	1.52	1.46	1.41	1.37	1.33	1.30	1.26	1.23	1.21	1.18	1.16	1.14	1.12	1.08	1.05
285	1.69	1.62	1.55	1.49	1.44	1.39	1.35	1.31	1.28	1.25	1.22	1.19	1.17	1.14	1.12	1.10	1.07	1.03
290	1.66	1.59	1.52	1.47	1.42	1.37	1.33	1.29	1.26	1.23	1.20	1.17	1.15	1.13	1.11	1.09	1.05	1.02
295	1.63	1.56	1.50	1.44	1.39	1.35	1.31	1.27	1.24	1.21	1.18	1.16	1.13	1.11	1.09	1.07	1.04	1.01
300	1.60	1.53	1.47	1.42	1.37	1.33	1.29	1.25	1.22	1.19	1.16	1.14	1.12	1.10	1.08	1.06	1.03	1.00
305	1.57	1.51	1.45	1.39	1.35	1.30	1.27	1.23	1.20	1.17	1.15	1.12	1.10	1.08	1.06	1.04	1.01	0.98
310	1.54	1.48	1.42	1.37	1.32	1.28	1.25	1.21	1.18	1.16	1.13	1.11	1.09	1.07	1.05	1.03	1.00	0.97
315	1.51	1.45	1.39	1.34	1.30	1.26	1.23	1.19	1.16	1.14	1.11	1.09	1.07	1.05	1.03	1.02	0.99	0.96
320	1.48	1.42	1.37	1.32	1.28	1.24	1.20	1.17	1.14	1.12	1.10	1.07	1.05	1.03	1.02	1.00	0.97	0.95
325	1.45	1.39	1.34	1.30	1.25	1.22	1.18	1.15	1.13	1.10	1.08	1.06	1.04	1.02	1.00	0.99	0.96	0.93
330	1.43	1.37	1.32	1.27	1.23	1.19	1.16	1.13	1.11	1.08	1.06	1.04	1.02	1.00	0.99	0.97	0.95	0.92
333	1.41	1.35	1.30	1.26	1.22	1.18	1.15	1.12	1.10	1.07	1.05	1.03	1.01	0.99	0.98	0.96	0.94	0.91

Table C.7 continued...

f_s	M/bd^2																	
	2.60	2.80	3.00	3.20	3.40	3.60	3.80	4.00	4.20	4.40	4.60	4.80	5.00	5.20	5.40	5.60	5.80	6.00
100	1.45	1.40	1.36	1.32	1.28	1.25	1.22	1.19	1.17	1.14	1.12	1.10	1.08	1.07	1.05	1.03	1.02	1.01
110	1.42	1.38	1.33	1.30	1.26	1.23	1.20	1.17	1.15	1.13	1.11	1.09	1.07	1.05	1.04	1.02	1.01	0.99
120	1.40	1.35	1.31	1.28	1.24	1.21	1.18	1.16	1.13	1.11	1.09	1.07	1.05	1.04	1.02	1.01	0.99	0.98
130	1.38	1.33	1.29	1.26	1.22	1.19	1.17	1.14	1.12	1.10	1.08	1.06	1.04	1.02	1.01	0.99	0.98	0.97
140	1.35	1.31	1.27	1.23	1.20	1.17	1.15	1.12	1.10	1.08	1.06	1.04	1.03	1.01	1.00	0.98	0.97	0.96
145	1.34	1.30	1.26	1.22	1.19	1.16	1.14	1.11	1.09	1.07	1.05	1.04	1.02	1.00	0.99	0.98	0.96	0.95
150	1.33	1.29	1.25	1.21	1.18	1.16	1.13	1.11	1.08	1.06	1.05	1.03	1.01	1.00	0.98	0.97	0.96	0.94
155	1.32	1.28	1.24	1.20	1.17	1.15	1.12	1.10	1.08	1.06	1.04	1.02	1.00	0.99	0.98	0.96	0.95	0.94
160	1.30	1.26	1.23	1.19	1.16	1.14	1.11	1.09	1.07	1.05	1.03	1.01	1.00	0.98	0.97	0.96	0.94	0.93
165	1.29	1.25	1.22	1.18	1.15	1.13	1.10	1.08	1.06	1.04	1.02	1.01	0.99	0.98	0.96	0.95	0.94	0.93
170	1.28	1.24	1.21	1.17	1.14	1.12	1.09	1.07	1.05	1.03	1.02	1.00	0.98	0.97	0.96	0.94	0.93	0.92
175	1.27	1.23	1.20	1.16	1.14	1.11	1.09	1.06	1.04	1.02	1.01	0.99	0.98	0.96	0.95	0.94	0.93	0.91
180	1.26	1.22	1.18	1.15	1.13	1.10	1.08	1.06	1.04	1.02	1.00	0.98	0.97	0.96	0.94	0.93	0.92	0.91
185	1.25	1.21	1.17	1.14	1.12	1.09	1.07	1.05	1.03	1.01	0.99	0.98	0.96	0.95	0.94	0.92	0.91	0.90
190	1.23	1.20	1.16	1.13	1.11	1.08	1.06	1.04	1.02	1.00	0.98	0.97	0.96	0.94	0.93	0.92	0.91	0.90
195	1.22	1.19	1.15	1.12	1.10	1.07	1.05	1.03	1.01	0.99	0.98	0.96	0.95	0.94	0.92	0.91	0.90	0.89
200	1.21	1.17	1.14	1.11	1.09	1.06	1.04	1.02	1.00	0.99	0.97	0.95	0.94	0.93	0.92	0.91	0.89	0.88
205	1.20	1.16	1.13	1.10	1.08	1.05	1.03	1.01	0.99	0.98	0.96	0.95	0.93	0.92	0.91	0.90	0.89	0.88
210	1.19	1.15	1.12	1.09	1.07	1.04	1.02	1.00	0.99	0.97	0.95	0.94	0.93	0.91	0.90	0.89	0.88	0.87
215	1.17	1.14	1.11	1.08	1.06	1.04	1.01	1.00	0.98	0.96	0.95	0.93	0.92	0.91	0.90	0.89	0.88	0.87
220	1.16	1.13	1.10	1.07	1.05	1.03	1.01	0.99	0.97	0.95	0.94	0.93	0.91	0.90	0.89	0.88	0.87	0.86
225	1.15	1.12	1.09	1.06	1.04	1.02	1.00	0.98	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.87	0.86	0.85
230	1.14	1.11	1.08	1.05	1.03	1.01	0.99	0.97	0.95	0.94	0.92	0.91	0.90	0.89	0.88	0.87	0.86	0.85
235	1.13	1.10	1.07	1.04	1.02	1.00	0.98	0.96	0.95	0.93	0.92	0.90	0.89	0.88	0.87	0.86	0.85	0.84
240	1.11	1.08	1.06	1.03	1.01	0.99	0.97	0.95	0.94	0.92	0.91	0.90	0.88	0.87	0.86	0.85	0.84	0.84
245	1.10	1.07	1.05	1.02	1.00	0.98	0.96	0.94	0.93	0.91	0.90	0.89	0.88	0.87	0.86	0.85	0.84	0.83
250	1.09	1.06	1.04	1.01	0.99	0.97	0.95	0.94	0.92	0.91	0.89	0.88	0.87	0.86	0.85	0.84	0.83	0.82
255	1.08	1.05	1.02	1.00	0.98	0.96	0.94	0.93	0.91	0.90	0.89	0.87	0.86	0.85	0.84	0.83	0.83	0.82
260	1.07	1.04	1.01	0.99	0.97	0.95	0.93	0.92	0.90	0.89	0.88	0.87	0.86	0.85	0.84	0.83	0.82	0.81
265	1.05	1.03	1.00	0.98	0.96	0.94	0.93	0.91	0.90	0.88	0.87	0.86	0.85	0.84	0.83	0.82	0.81	0.81
270	1.04	1.02	0.99	0.97	0.95	0.93	0.92	0.90	0.89	0.88	0.86	0.85	0.84	0.83	0.82	0.82	0.81	0.80
275	1.03	1.00	0.98	0.96	0.94	0.92	0.91	0.89	0.88	0.87	0.86	0.85	0.84	0.83	0.82	0.81	0.80	0.79
280	1.02	0.99	0.97	0.95	0.93	0.91	0.90	0.89	0.87	0.86	0.85	0.84	0.83	0.82	0.81	0.80	0.80	0.79
285	1.01	0.98	0.96	0.94	0.92	0.91	0.89	0.88	0.86	0.85	0.84	0.83	0.82	0.81	0.80	0.80	0.79	0.78
290	1.00	0.97	0.95	0.93	0.91	0.90	0.88	0.87	0.86	0.84	0.83	0.82	0.81	0.81	0.80	0.79	0.78	0.78
295	0.98	0.96	0.94	0.92	0.90	0.89	0.87	0.86	0.85	0.84	0.83	0.82	0.81	0.80	0.79	0.78	0.78	0.77
300	0.97	0.95	0.93	0.91	0.89	0.88	0.86	0.85	0.84	0.83	0.82	0.81	0.80	0.79	0.78	0.78	0.77	0.76
305	0.96	0.94	0.92	0.90	0.88	0.87	0.85	0.84	0.83	0.82	0.81	0.80	0.79	0.78	0.78	0.77	0.76	0.76
310	0.95	0.93	0.91	0.89	0.87	0.86	0.85	0.83	0.82	0.81	0.80	0.79	0.79	0.78	0.77	0.76	0.76	0.75
315	0.94	0.91	0.90	0.88	0.86	0.85	0.84	0.83	0.81	0.80	0.80	0.79	0.78	0.77	0.76	0.76	0.75	0.75
320	0.92	0.90	0.89	0.87	0.85	0.84	0.83	0.82	0.81	0.80	0.79	0.78	0.77	0.76	0.76	0.75	0.75	0.74
325	0.91	0.89	0.87	0.86	0.84	0.83	0.82	0.81	0.80	0.79	0.78	0.77	0.76	0.76	0.75	0.74	0.74	0.73
330	0.90	0.88	0.86	0.85	0.83	0.82	0.81	0.80	0.79	0.78	0.77	0.76	0.76	0.75	0.74	0.74	0.73	0.73
333	0.89	0.87	0.86	0.84	0.83	0.82	0.81	0.79	0.79	0.78	0.77	0.76	0.75	0.75	0.74	0.73	0.73	0.72

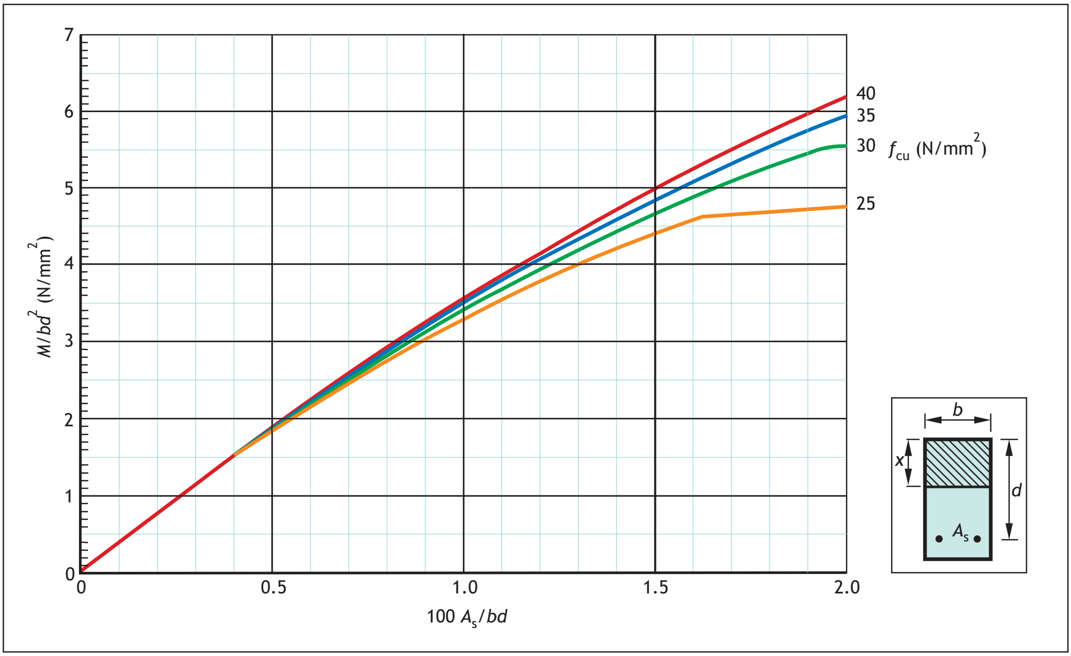


Figure C.3
Design chart for singly reinforced beam

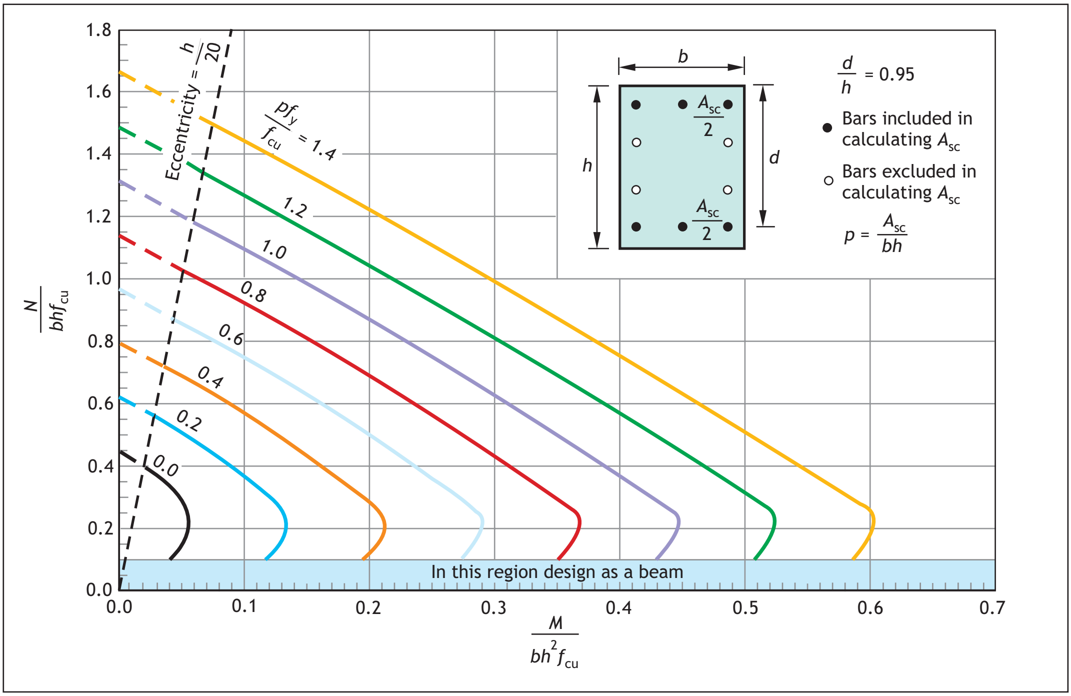


Figure C.4a
Column design chart for rectangular column d/h = 0.95

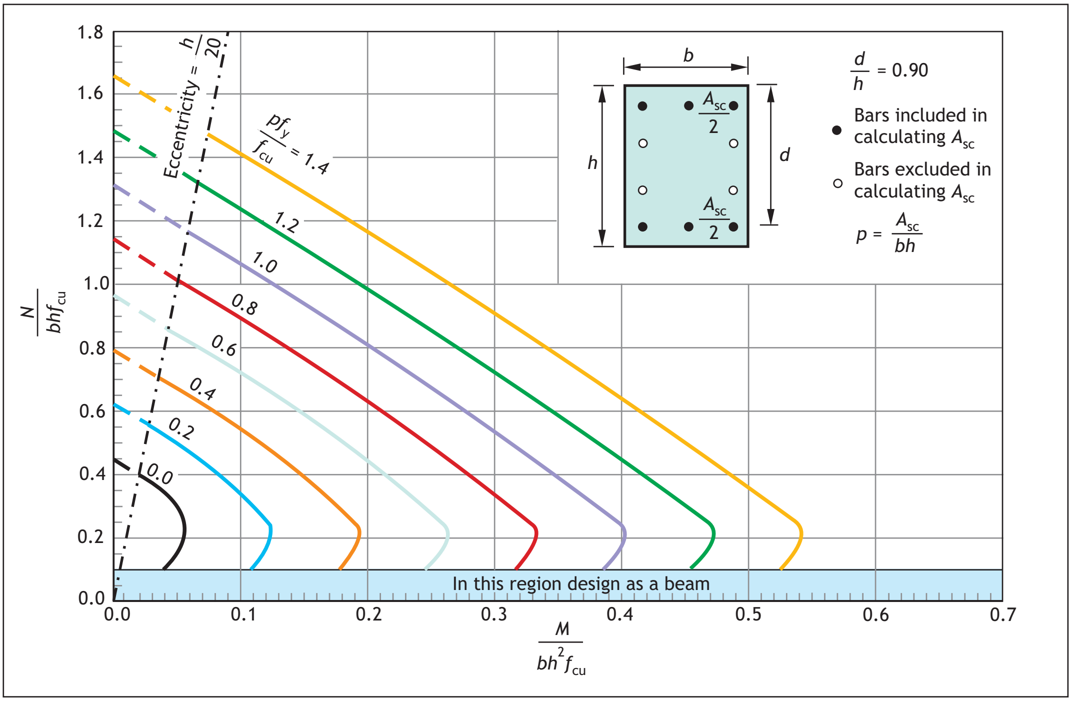


Figure C.4b
Column design chart for rectangular column *d*/*h* = 0.90

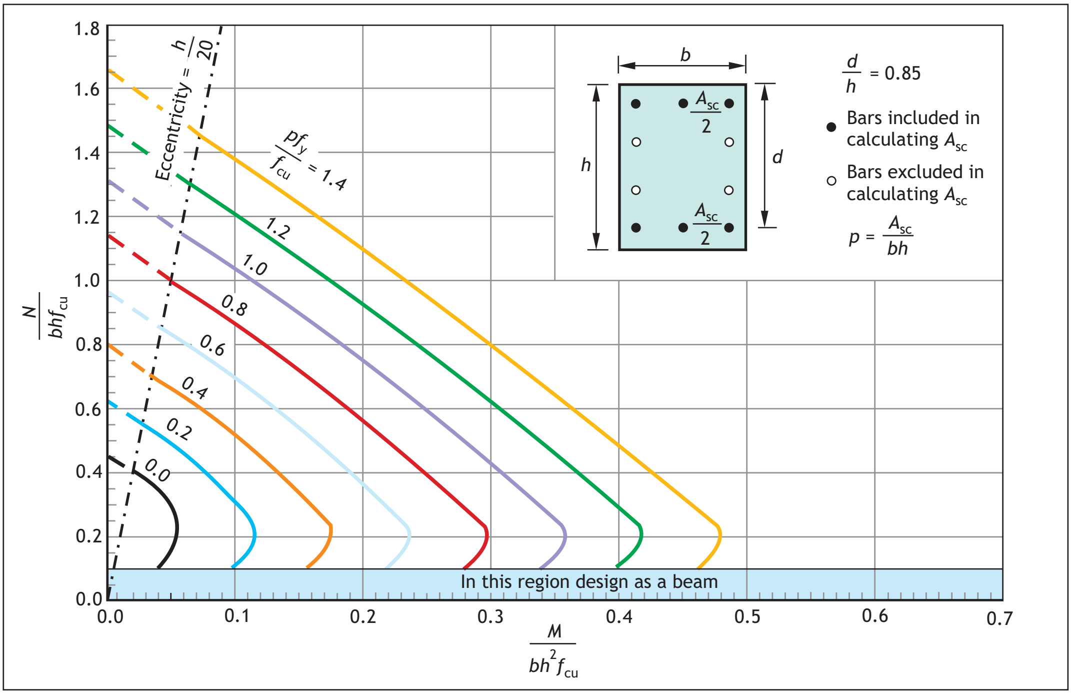


Figure C.4c
Column design chart for rectangular column *d*/*h* = 0.85

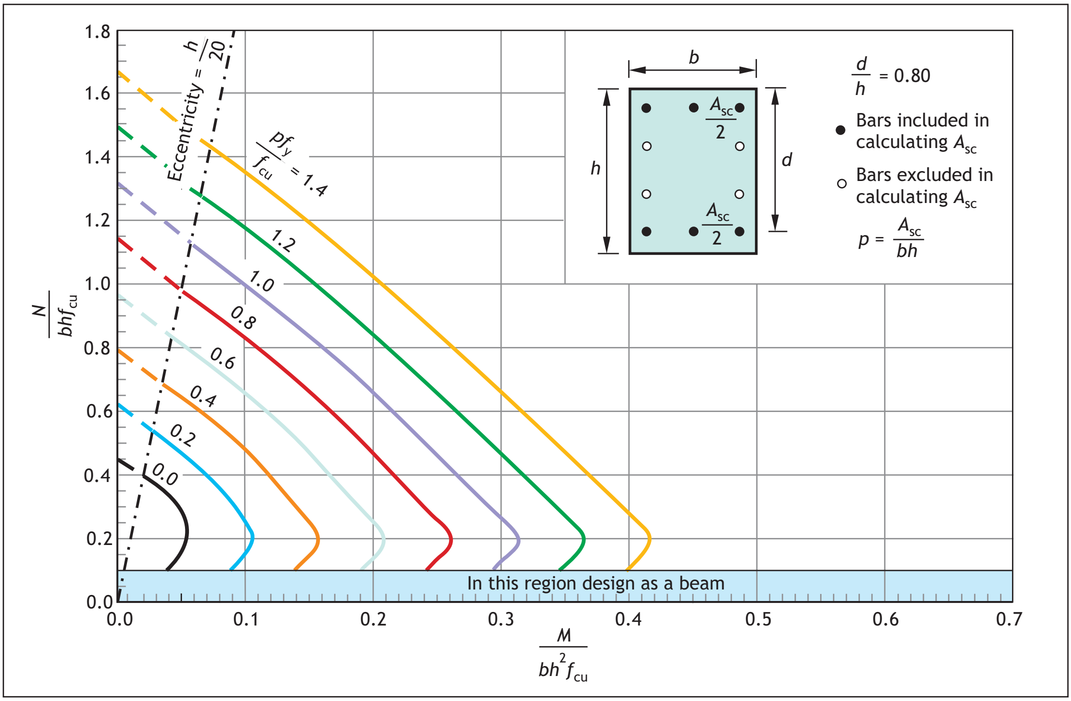


Figure C.4d
Column design chart for rectangular column *d*/*h* = 0.80

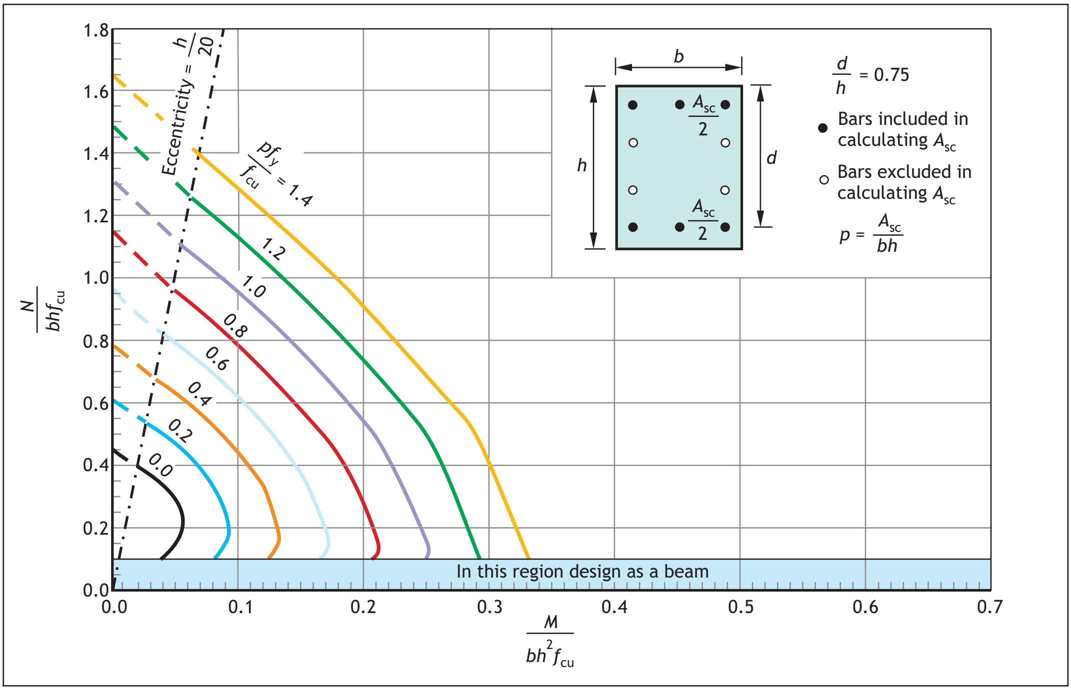


Figure C.4e
Column design chart for rectangular column *d*/*h* = 0.75

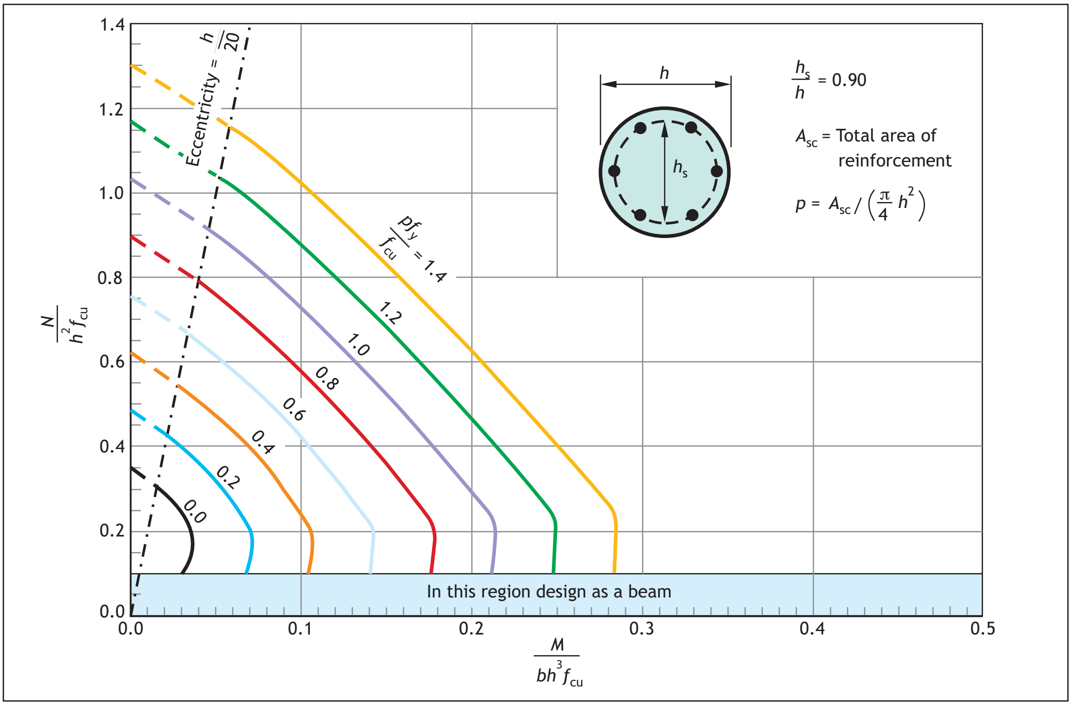


Figure C.5a
Column design chart for circular column *d/h* = 0.90

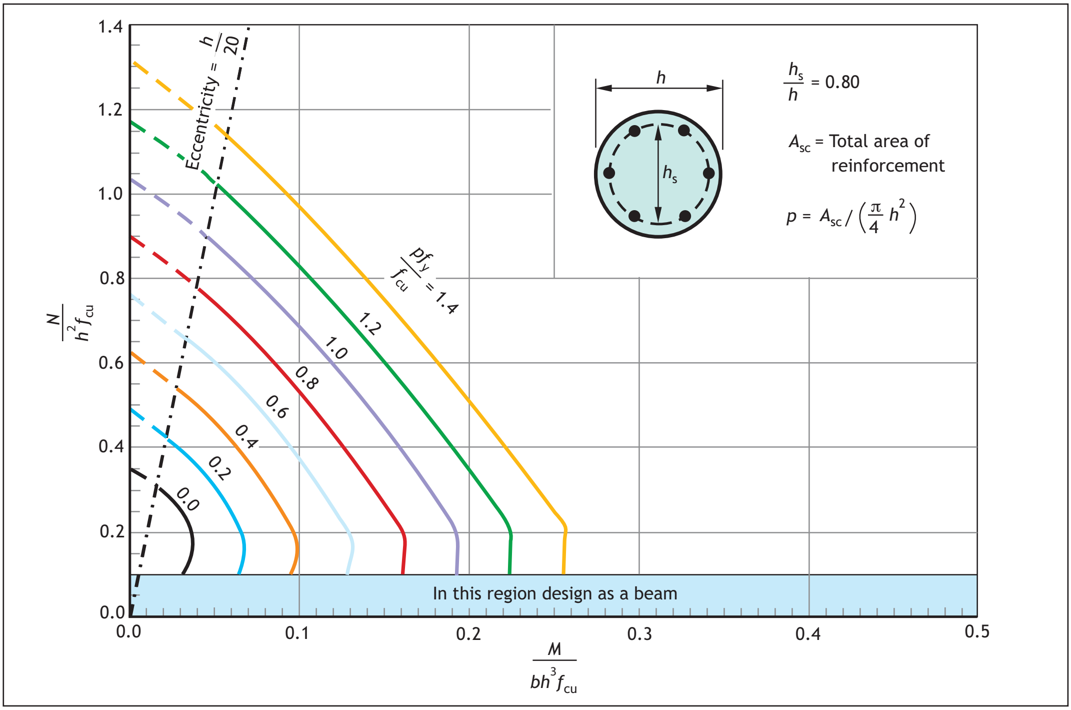


Figure C.5b
Column design chart for circular column *d/h* = 0.80

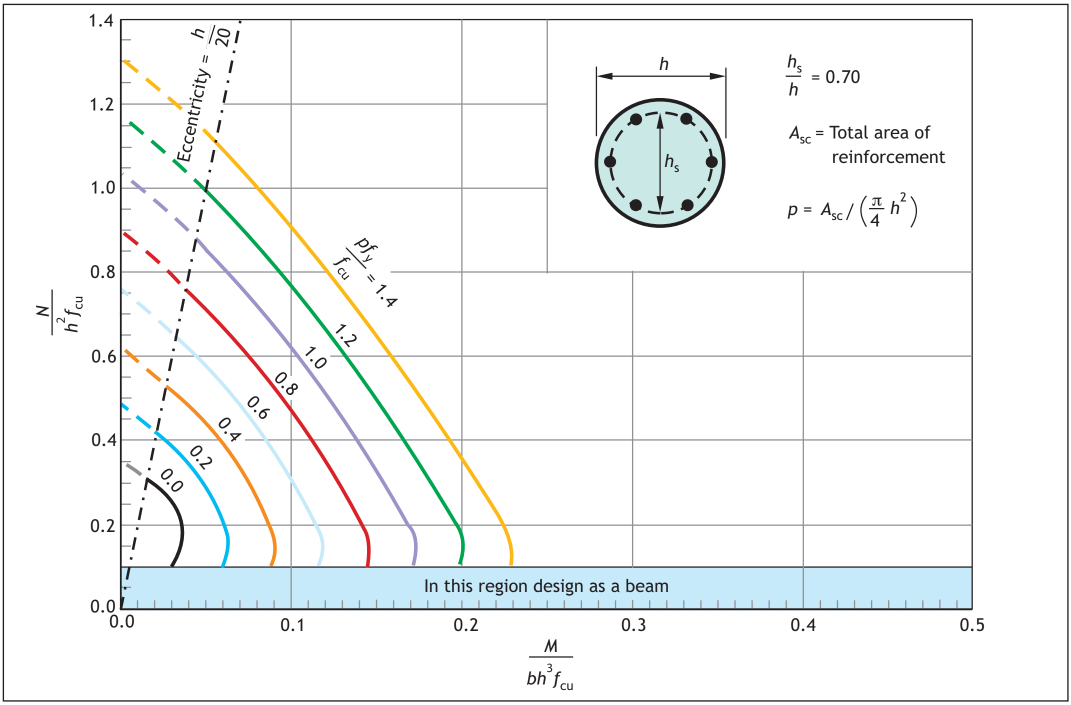


Figure C.5c
Column design chart for circular column *d/h* = 0.70

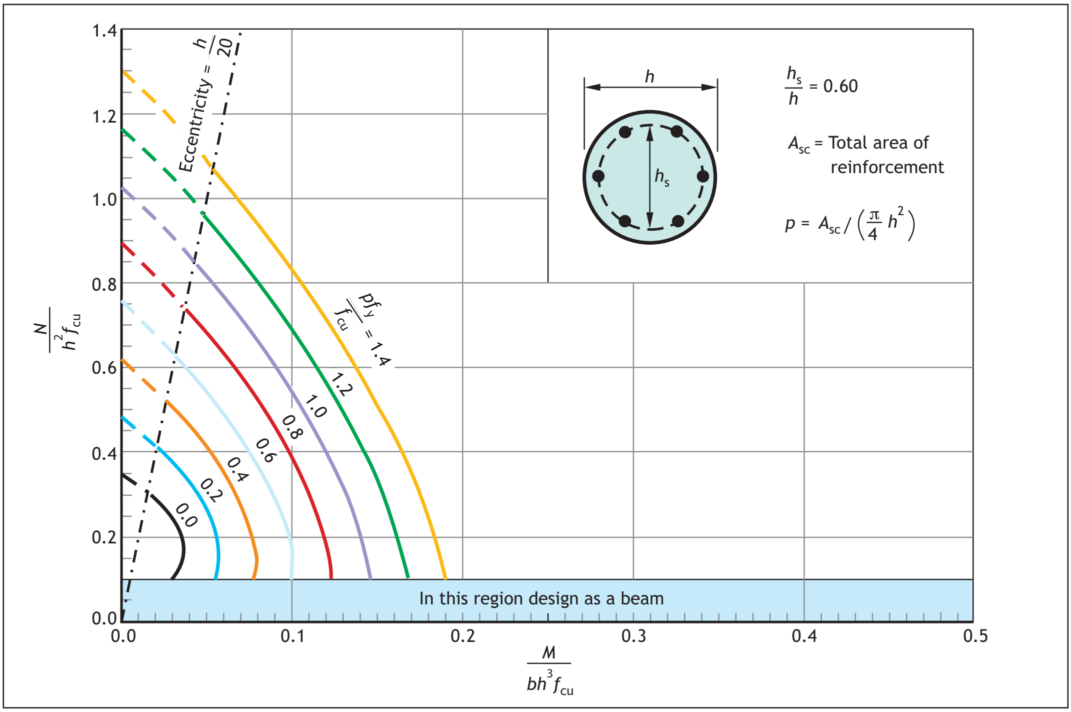


Figure C.5d
Column design chart for circular column *d/h* = 0.60

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Index

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	251	302	352	402	452	503
10	78.5	157	236	314	393	471	550	628	707	785
12	113	226	339	452	565	679	792	905	1020	1130
16	201	402	603	804	1010	1210	1410	1610	1810	2010
20	314	628	942	1260	1570	1880	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 spacings of bars (mm^2)

Bar size (mm)	Spacing of bars (mm^2)									
	75	100	125	150	175	200	225	250	275	300
8	670	503	402	335	287	251	223	201	183	168
10	1050	785	628	524	449	393	349	314	286	262
12	1510	1130	905	754	646	565	503	452	411	377
16	2680	2010	1610	1340	1150	1010	894	804	731	670
20	4190	3140	2510	2090	1800	1570	1400	1260	1140	1050
25	6540	4910	3930	3270	2800	2450	2180	1960	1780	1640
32	10700	8040	6430	5360	4600	4020	3570	3220	2920	2680
40	16800	12600	10100	8380	7180	6280	5590	5030	4570	4190

Mass of groups of bars (kg per metre run)

Bar size (mm)	Number of bars									
	1	2	3	4	5	6	7	8	9	10
8	0.395	0.789	1.184	1.578	1.973	2.368	2.762	3.157	3.551	3.946
10	0.617	1.233	1.850	2.466	3.083	3.699	4.316	4.932	5.549	6.165
12	0.888	1.776	2.663	3.551	4.439	5.327	6.215	7.103	7.990	8.878
16	1.578	3.157	4.735	6.313	7.892	9.470	11.048	12.627	14.205	15.783
20	2.466	4.932	7.398	9.865	12.331	14.797	17.263	19.729	22.195	24.662
25	3.853	7.707	11.560	15.413	19.267	23.120	26.974	30.827	34.680	38.534
32	6.313	12.627	18.940	25.253	31.567	37.880	44.193	50.507	56.820	63.133
40	9.865	19.729	29.594	39.458	49.323	59.188	69.052	78.917	88.781	98.646

Mass for various spacings of bars (kg per m^2)

Bar size (mm)	Spacing of bars (mm)									
	75	100	125	150	175	200	225	250	275	300
8	5.261	3.946	3.157	2.631	2.255	1.973	1.754	1.578	1.435	1.315
10	8.221	6.165	4.932	4.110	3.523	3.083	2.740	2.466	2.242	2.055
12	11.838	8.878	7.103	5.919	5.073	4.439	3.946	3.551	3.228	2.959
16	21.044	15.783	12.627	10.522	9.019	7.892	7.015	6.313	5.739	5.261
20	32.882	24.662	19.729	16.441	14.092	12.331	10.961	9.865	8.968	8.221
25	51.378	38.534	30.827	25.689	22.019	19.267	17.126	15.413	14.012	12.845
32	84.178	63.133	50.507	42.089	36.076	31.567	28.059	25.253	22.958	21.044
40	131.528	98.646	78.917	65.764	56.369	49.323	43.843	39.458	35.871	32.882

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Concrete Buildings Scheme Design Manual

This handbook is intended to be a quick reference guide for candidates taking the Institution of Structural Engineer's Chartered Membership Examination. It will also form an essential reference source in the design office.

It gives guidance on every section of the examination, including selecting viable options, appraising the designs, initial sizing, design calculations and producing drawings, details, method statement and programmes, and is packed with useful design data and charts.

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