A **cement** and **concrete** industry publication

Economic Concrete Frame Elements to Eurocode 2

A pre-scheme handbook for the rapid sizing and selection of reinforced concrete frame elements in multi-storey buildings designed to Eurocode 2

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Foreword

This publication is based on design to Eurocode 2 and updates the original pre-scheme sizing handbook Economic Concrete Frame Elements which was based on BS 8110 and published in 1997.

Eurocode 2 brings economies over BS 8110 in some areas – up to 10% has been reported. While sizes of frame elements to BS 8110 would generally be safe, they would be sometimes unduly conservative and uneconomic in increasingly competitive markets. In addition, current British Standards for structural design are due to be withdrawn by 2010, with BS 8110 *Structural use of concrete* being made obsolete in 2008. Thus this new edition of *Economic concrete frame elements* has been produced by The Concrete Centre.

The new charts and data have been derived from design spreadsheets that carry out design to Eurocode 2 and, as appropriate, other Eurocodes, European and British Standards. The methodology behind the charts and data is fully explained and is, essentially, the same as that used for the previous version of this publication. However, the following should be noted:

- For continuous members, sizes are derived from analysis which, in the case of in-situ beams, includes the frame action of small columns.
- A new method for determining the sizes of perimeter columns is introduced. This takes account of both axial load and moment.
- Generally, in line with BS EN 1990 and its National Annex, loading is based on $1.25G_k + 1.5Q_k$ for residential and office areas and $1.35G_k + 1.5Q_k$ for storage areas.
- Much of the economy over the charts and data for BS 8110 comes from the treatment of loads and deflection by the Eurocodes – please refer to Deflection in Section 7.1.2.
- Ribbed slabs are an exception. Compared with BS 8110 greater depths are required.

Readers are advised to be conservative with their choices until such time as they become familiar with this publication and the workings of Eurocode 2.

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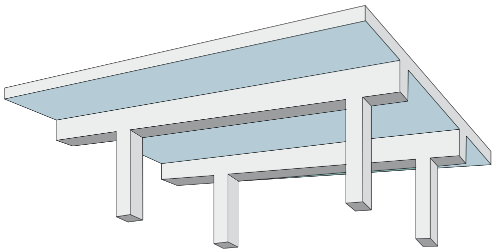

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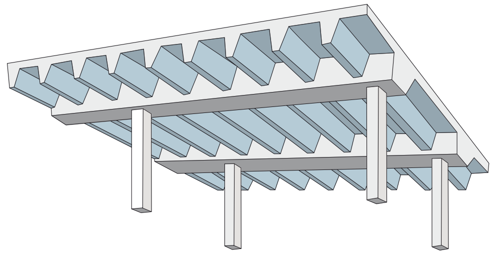

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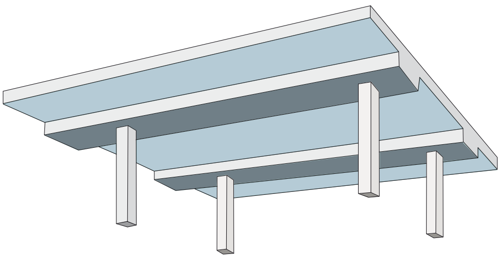

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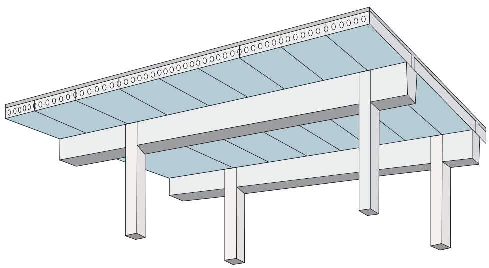

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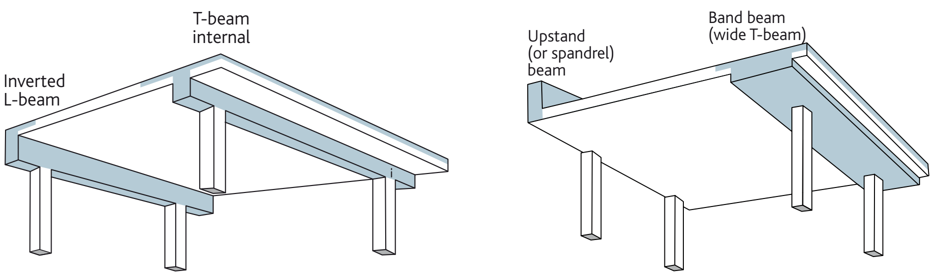

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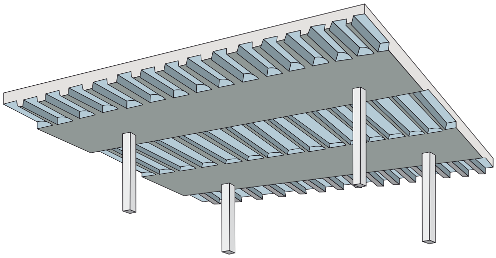

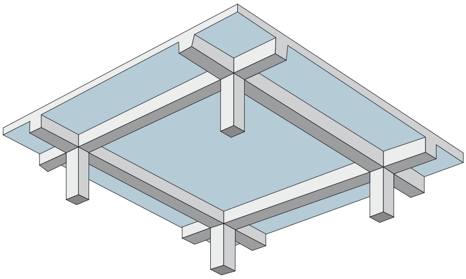

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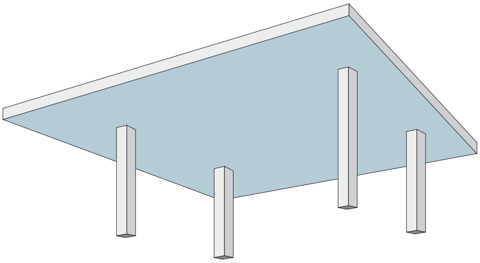

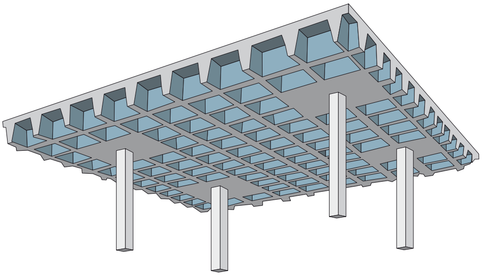

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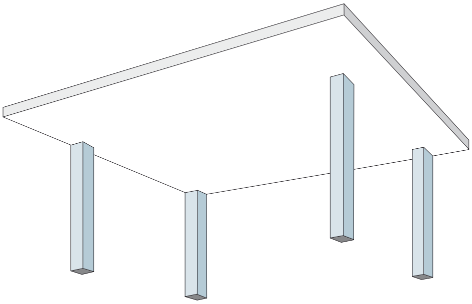

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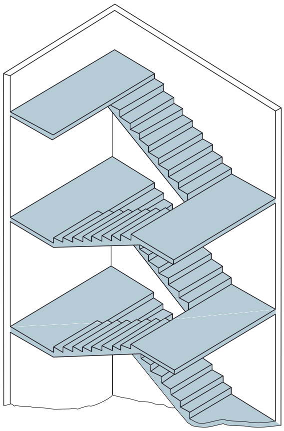


Symbols and abbreviations used in this publication

Symbol	Definition
A	Cross-sectional area; Accidental action
A_c	Cross-sectional area of concrete
A_{ps}	Cross-sectional area of prestressing reinforcement
A_s	Cross-sectional area of reinforcement
$A_{s,prov}$	Area of steel provided
$A_{s,req}$	Area of steel required
b	Overall width of a cross-section, or overall flange width in a T- or L-beam
b_e	Effective width of a flat slab (adjacent to perimeter column: used in determination of $M_{t,max}$)
b_w	Width of the web e.g. in rectangular, T-, I- or L-beams
b_{wmin}	Width of the web (double-tees)
c_{nom}	Nominal cover
d	Effective depth of a cross-section
E_{cm}	Mean secant modulus of elasticity of concrete
$E_{cm,i}$	Young's modulus (initial secant modulus at transfer of prestressing stresses to concrete)
$E_{cm(t)}$	Mean secant modulus of elasticity of concrete at transfer of prestress
EI	Stiffness, modulus of elasticity (E) x moment of inertia (I)
E_{ps}	Modulus of elasticity of Young's modulus for prestressing reinforcement
Exp.	Expression; Exposure class
e	Eccentricity
e_i	Eccentricity due to imperfections
erf	Elastic reaction factor
F_k	Characteristic value of an action
F_{rep}	Representative action. (= ψF_k where ψ = factor to convert characteristic value to representative value)
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
$f_{ck,i}$	Characteristic compressive cylinder strength of the topping at depropping
$f_{ck(t)}$	Characteristic compressive cylinder strength of concrete at transfer of prestress
f_{pk}	Characteristic yield strength of prestressing reinforcement
f_{yk}	Characteristic yield strength of reinforcement
G_k	Characteristic value of a permanent action (load)
G_{kc}	Characteristic self-weight of column
g_k	Characteristic value of a permanent action (load) per unit length or area
g_{kbm}	Adjustment in characteristic dead load in self-weight of beam to allow for thicknesses of slab ≠ 200 mm
g_{kc}	Characteristic dead load of cladding
g_{ko}	Characteristic dead load of other line loads
g_{ks}	Characteristic self-weight of slab
g_{ksdl}	Characteristic superimposed dead loads
h	Overall depth of a cross-section; Height
h_f	Depth of top flange (double-tees)
IL	Characteristic imposed load

Symbol	Definition
K	Effective length factor; Wobble factor
K_{φ}	Creep factor
l (or L)	Length; Span
L_0	Effective length of columns (or walls)
l_0	Distance between points of zero moment
l_s	Slab span perpendicular to beam
l_y, (l_z)	Span in the y (z) direction
M	Bending moment; Moment from 1st order analysis
M_{Ed}	Design moment
M_{0Ed}	Equivalent 1st order moment at about mid height of a column
$M_{t,max}$	Maximum transfer moment (between flat slab and edge support)
M_y (M_z)	Moment about the y-axis (z-axis) from 1st order analysis
NA	National Annex
N_{Ed}	Ultimate axial load(tension or compression at ULS)
n_{ll}	Ultimate line loads
n_s	Ultimate slab load
P/A	Prestress, MPa
$P\Delta$	Moment caused by a force at an eccentricity
PT	Post-tensioned concrete
Q_k	Characteristic value of a variable action (load)
q_k	Characteristic value of a variable action (load) per unit length or area
q_{ks}	Allowance for movable partitions treated as a characteristic variable action (load) per unit area
RC	Reinforced concrete
SDL	Superimposed dead loading
SLS	Serviceability limit state(s)
uaudl	Ultimate applied uniformly distributed load
ULS	Ultimate limit state(s)
V	Shear; Beam reaction
v_{Ed}	Shear stress; Punching shear stress at ULS
v_{Rd}	Allowable shear stress at ULS
w_{max}	Limiting calculated crack width
w_k	Crack width
α_n	Imposed load reduction factor
γ_C	Partial factor for concrete
γ_F	Partial factor for actions, F
γ_{fgk}	Partial factor for permanent actions (dead loads)
γ_{fqk}	Partial factor for imposed loads (variable actions)
γ_G	Partial factor for permanent actions, G
γ_S	Partial factor for steel
γ_Q	Partial factor for variable actions, Q
Δ	Change in
Δc_{dev}	Allowance made in design for deviation
ζ	Distribution coefficient
ε_c	Strain, e.g. shrinkage
μ	Coefficient of friction

Symbol	Definition
ξ	Reduction factor applied to G_k in BS EN 1990 Expression (6.10b)
ρ	Required tension reinforcement ratio, $A_{s,req} / A_c$
σ_s	Compressive concrete stress under the design load at SLS
σ_c	Tensile steel stress under the design load at SLS
φ	Creep factor
ϕ	Diameter (of reinforcement)
ψ	Factors defining representative values of variable actions
ψ_0	Combination value of ψ
ψ_1	Frequent value of ψ
ψ_2	Quasi-permanent value of ψ
	Single span
	Multiple span

1 Introduction

In conceiving a design for a multi-storey structure, there are, potentially, many options to be considered. The purpose of this publication is to help designers identify least-cost concrete options quickly. It does this by:

- Presenting feasible, economic concrete options for consideration
- Providing preliminary sizing of concrete frame elements in multi-storey structures
- Providing first estimates of reinforcement quantities
- Outlining the effects of using different types of concrete elements
- Helping ensure that the right concrete options are considered for scheme design

This handbook contains charts and data that present economic sizes for many types of concrete elements over a range of common loadings and spans. The main emphasis is on floor plates as these commonly represent 85% of superstructure costs. A short commentary on each type of element is given. This publication does not cover lateral stability; it presumes that stability will be provided by other means (e.g. by shear walls) and will be checked independently, nor does it cover foundations.

The charts and data work on loads as follows:

- **For slabs** – Economic depths are plotted against span for a range of characteristic imposed loads.
- **For beams** – Economic depths are plotted against span for a range of ultimate applied uniformly distributed loads, uaudl.

  Uaudl is the summation of ultimate loads from slabs (available from slab data), cladding, etc., with possible minor adjustment for beam self-weight and cladding.
- **For columns** – For internal columns a load:size chart is plotted. For perimeter columns, moment and moment:load charts are given.

  Data provided for beams and two-way slabs include ultimate axial loads to columns.

  Charts help to determine edge and corner column moments. Other charts give column sizes and reinforcement arrangements.

Thus a conceptual design can be built up by following load paths down the structure. For in-situ elements see Section 3, for precast elements see Section 4, for post-tensioned slabs and beams see Section 5. This publication will be the basis for an update of *CONCEPT* [1], a complementary computer-based conceptual design program available from The Concrete Centre, which produces a rapid and semi-automatic comparison of a number of concrete options.

Generally, the sizes given in this publication correspond to the minimum total cost of concrete, formwork, reinforcement, perimeter cladding and cost of supporting self-weight and imposed loads whilst complying with the requirements of Part 1 of BS EN 1992, Eurocode 2: *Design of concrete structures* [2, 3]. The charts and data are primarily intended for use by experienced engineers who are expected to make judgements as to how the information is used. The charts and data are based on idealised models. Engineers must assess the data in the light of their own experience and methods of working, their particular concerns, and the requirements of the project in hand.

This publication is intended as a handbook for the conceptual design of concrete structures in multi-storey buildings. It cannot, and should not, be used for actual structural scheme design, which should be undertaken by a properly experienced and qualified engineer. However, it should give other interested parties a ‘feel’ for the different options at a very early stage and will help designers choose the most viable options quickly and easily. These can be compared using *CONCEPT*.

2 Using the charts and data

2.1 General

The charts and data are intended to be used as shown below.

Determine general design criteria

See Sections 2.2 & 2.3

Establish layout, spans, loads, intended use, stability, aesthetics, service integration, programme and other issues. Identify worst case(s) of span and load.

Short-list feasible options

See Section 2.4

Envisage the structure as a whole. With rough sketches of typical structural bays, consider, and whenever possible, discuss likely alternative forms of construction (see Pictorial index, p. ii and the economic span ranges shown in Figure 2.2). Identify preferred structural solutions using in-situ (Section 3), precast (Section 4) and post-tensioned (Section 5) construction singly or in combinations..

For each short-listed option

Yes

Determine slab thickness

See Sections 2.5 & 8.2

Interpolate from the appropriate chart or data, using the maximum slab span and the relevant characteristic imposed load, i.e. interpolate between IL = 2.5, 5.0, 7.5 and 10.0 kN/m^2. NB: Generally 1.5 kN/m^2 is allowed for finishes and services.

Make note of ultimate line loads to supporting beams (i.e. characteristic line loads x load factors) or, in the case of flat slabs, troughed slabs, etc. ultimate axial loads to columns.

Determine beam sizes

See Sections 2.6 & 8.3

Estimate ultimate applied uniformly distributed load (uaudl) to beams by summing ultimate loads from slab(s), cladding and other line loads.

Choose the charts for the appropriate form and width of beam and determine depth by interpolating from the chart and/or data for the maximum beam span and the estimated ultimate applied uniformly distributed load (uaudl).

Note ultimate loads to supporting columns.

Determine column sizes

See Sections 2.7 & 8.4

Estimate total ultimate axial load (N_{Ed}) at lowest levels, e.g. multiply ultimate load per floor by the relevant number of storeys. Adjust if required, to account for elastic reaction factors, etc.

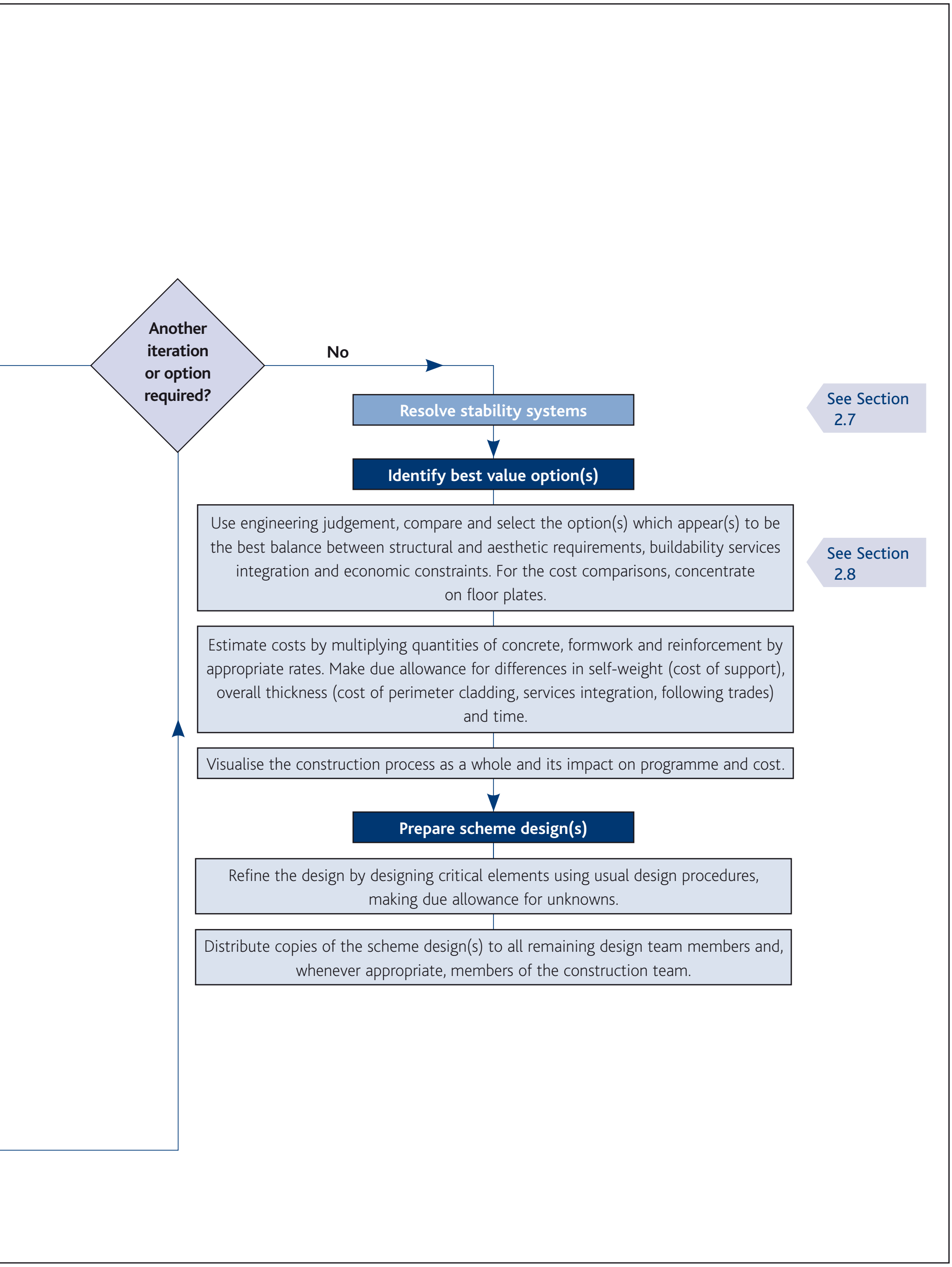
For internal columns interpolate square size of column from the appropriate chart and/or data using the estimated total ultimate axial load.

For perimeter columns, in addition to estimating N_{Ed}, estimate moment in column from charts according to assumed size of column and either:

- *Beam span in beam-and-slab construction* or
- *Slab span in flat slab construction.*

Use further charts to check adequacy and suitability of chosen column size for derived axial load and moment. Iterate as necessary.

Figure 2.1
Flowchart showing how to use this publication


Another iteration or option required?
No
Resolve stability systems
See Section 2.7
Identify best value option(s)
Use engineering judgement, compare and select the option(s) which appear(s) to be the best balance between structural and aesthetic requirements, buildability services integration and economic constraints. For the cost comparisons, concentrate on floor plates.
See Section 2.8
Estimate costs by multiplying quantities of concrete, formwork and reinforcement by appropriate rates. Make due allowance for differences in self-weight (cost of support), overall thickness (cost of perimeter cladding, services integration, following trades) and time.
Visualise the construction process as a whole and its impact on programme and cost.
Prepare scheme design(s)
Refine the design by designing critical elements using usual design procedures, making due allowance for unknowns.
Distribute copies of the scheme design(s) to all remaining design team members and, whenever appropriate, members of the construction team.

2.2 Basis and limitations on use

2.2.1 General

The charts and data in this publication are intended for use with the pre-scheme design of medium-rise multi-storey building frames and structures by experienced engineers who are expected to make judgements as to how the information is used. In producing the charts and data many assumptions have been made. These assumptions are more fully described in Section 7, *Derivation of charts and data*, and in the charts and data themselves. The charts and data are valid only if these assumptions and restrictions hold true.

2.2.2 Accuracy

The charts and data have been prepared using spreadsheets that produced optimised results based on theoretical overall costs (see Section 7.1.1). Increments of 1 mm depth were used to obtain smooth curves for the charts (nonetheless some manual smoothing was necessary). The use of 1 mm increments is not intended to instil some false sense of accuracy into the figures given. Rather, the user is expected to exercise engineering judgement and round up both loads and depths in line with his or her confidence in the design criteria being used and normal modular sizing. Thus, rather than using a 241 mm thick slab, it is intended that the user would actually choose a 250, 275 or 300 mm thick slab, confident in the knowledge that, provided loads and spans had been accurately assessed, a 241 mm slab would work. Going up to, say, a 300 mm thick slab might add 10% to the overall cost of structure and cladding, but this might be warranted in certain circumstances.

Note: The charted data is almost always close to minimum values, so it should never be rounded down.

2.2.3 Sensitivity

At pre-scheme design, it is unlikely that architectural layouts, finishes, services, and so forth, will have been finalised. Any options considered, indeed any structural scheme designs prepared, should therefore not be too sensitive to minor changes that are inevitable during the design development and construction phases.

2.2.4 Reinforcement densities

The data contain estimates of reinforcement densities (including tendons) for each element. The reinforcement data allow for calculated lap lengths and curtailment (but not wastage).

Estimates for elements may be aggregated to give very preliminary estimates of reinforcement quantities for comparative purposes only. They should be used with great caution (and definitely should not be used for contractual estimates of tonnages).

Many factors beyond the scope of this publication can affect reinforcement quantities on specific projects. These include non-rectangular layouts, large holes, actual covers used in design, detailing preferences (curtailment, laps, wastage), and the many unforeseen complications that inevitably occur. Different methods of analysis alone can account for 15% of reinforcement weight. Choosing to use a 275 mm deep slab rather than the 241 mm depth described above could reduce reinforcement tonnages by 7%.

Therefore, the densities given in the data are derived from simple rectangular layouts, using The Concrete Centre's interpretation of BS EN 1992[2, 3] (as described in Section 7), with allowances for curtailment and laps, but not for wastage.

2.2.5 Columns

The design of columns depends on many criteria. In this publication, only axial loads, and as far as possible moment, have been addressed. The sizes given (especially for perimeter columns) should, therefore, be regarded as tentative until proved by scheme design.

2.2.6 Stability

One of the main design criteria is stability. This handbook does not cover lateral stability, and presumes that stability will be provided by independent means (e.g. by shear walls).

2.3 General design criteria

2.3.1 Basic assumptions

Spans are defined as being from centreline of support to centreline of support. Although square bays are to be preferred on grounds of economy, architectural requirements will usually dictate the arrangement of floor layouts and the positioning of supporting walls and columns.

In terms of analysis, the following assumptions have been made for in-situ and post-tensioned elements:

- Slabs are supported on knife edge supports.
- Beams are supported by, and frame into, minimally sized supporting columns (250 mm square above and below).
- Flat slabs are supported by columns below only; column sizes as noted with the data.
- A maximum of 15% redistribution of moments at internal supports has been undertaken. (Beyond 15% the tables in BS EN 1992–1–2[3] become invalid.)
- Load arrangements are in accordance with the National Annex to BS EN 1992–1–1[2a] i.e. variable actions are applied on all or alternate spans.
- Loads are substantially uniformly distributed over single or multiple (three or more) spans.
- Variations in span length do not exceed 15% of the longest span.

Note: The more onerous of BS EN 1990 loading Expressions (6.10a) and (6.10b) is applied throughout.

Fixed values for ψ_2 (quasi-permanent proportion of imposed load) have been assumed. These values are detailed in Section 8.1.

Particular attention is drawn to the need to resolve lateral stability, and the layout of stair and service cores, which can have a dramatic effect on the position of vertical supports. Service core floors tend to have large holes, greater loads, but smaller spans than the main area of floor slab. Designs for the core and main floor should at least be compatible with each other.

2.3.2 Concrete grades

Concrete grade C30/37 has generally been used to generate data, apart from those for precast or prestressed members, where C40/50 was deemed more suitable. At the time of writing, BS 8500[4] specifies a grade C32/40 for certain exposure conditions, but the authors expect this to revert to the more standard C30/37 at the end of the overlap period between BS 8110[5] and Parts 1–1 and 2–1 of Eurocode 2[2, 3]. For exposure class XC1, lower concrete grades are permitted (down to C20/25), but the use of C30/37 will normally prove more economic.

2.3.3 Maximum spans

The charts and data should be interrogated at the maximum span of the member under consideration. Multiple-span continuous members are assumed to have equal spans with the end span being critical.

Often the spans will not be equal. The recommended use of the charts and data should therefore be restricted to spans that do not differ by more than 15% of the longest span. Nonetheless, the charts and data can be used beyond this limit, but with caution. Where end spans exceed inner spans by more than 15%, sizes should be increased to allow for, perhaps, 10% increase in moments. Conversely, where the outer spans are more than 15% shorter, sizes may be decreased. For in-situ elements, apart from slabs for use with 2400 mm wide beams, users may choose to multiply a maximum internal span by 0.92 to obtain an effective span at which to interrogate the relevant chart (based on the assumption of equal deflections in all spans, equal stiffness, *EI* and creep factor, φ).

2.3.4 Loads

Client requirements and occupancy or intended use usually dictate the imposed loads (IL) to be applied to floor slabs (BS EN 1991[6]). Finishes, services, cladding and layout of permanent partitions should be discussed with the other members of the design team in order that allowances (e.g. superimposed dead loads for slabs) can be determined. See Section 8.

In accordance with BS EN 1990 and its National Annex the worse case of Expressions (6.10a) and (6.10b) is used in the derivation of charts and data, i.e. for residential and office loads $n = 1.25g_k + 1.5q_k$; for storage loads (IL = 7.5 kN/m^2 and above) $n = 1.35g_k + 1.5q_k$.

To generate the tabulated data, it was necessary to assume values for ψ_2, the proportion of imposed loading considered to be permanent. For beams and columns, this value has conservatively been taken as 0.8. For slabs, ψ_2 has more realistically been assumed as 0.3 for an IL of 2.5 kN/m^2, 0.6 for ILs of 5.0 and 7.5 kN/m^2 and 0.8 for an IL of 10.0 kN/m^2. See Section 8.1.2 or see Table 2.1 in *Concise Eurocode 2*[7].

2.3.5 Intended use

Aspects such as provision for future flexibility, additional robustness, sound transmission, thermal mass, and so forth, need to be considered and can outweigh first cost economic considerations.

2.3.6 Stability

A means of achieving lateral stability (e.g. using core or shear walls or frame action) and robustness (e.g. by providing effective ties) must be resolved. Walls tend to slow up production, and sway frames should be considered for low-rise multi-storey buildings. This publication does not cover stability.

2.3.7 Fire resistance and exposure

The majority of the charts are intended for use on normal structures and are therefore based on 1 hour fire resistance and mild exposure (XC1).

Where the fire resistance and exposure conditions are other than normal, some guidance is given within the data. For other conditions and elements the reader should refer to Eurocode 2[2, 3] and, for precast elements, to manufacturers' recommendations.

Some relevant exposure conditions as defined in table 2.1 of Part 1–1 of Eurocode 2 are:

- XC1: concrete inside buildings with low air humidity; concrete permanently submerged in water.
- XC2: concrete surfaces subject to long-term water contact; many foundations.
- XC3: concrete inside buildings with moderate or high air humidity; external concrete sheltered from rain. XC3 also relates to internal voids and cores, such as in hollowcore units, unless the cores are sealed against ingress of moisture, in which case XC1 applies.
- XC4: concrete surfaces subject to water contact, not within exposure class XC2.
- XD1: concrete surfaces exposed to airborne chlorides. For chlorides and car parks refer to Section 4.1.4.

2.3.8 Aesthetic requirements

Aesthetic requirements should be discussed. If the structure is to be exposed, a realistic strategy to obtain the desired standard of finish should be formulated and agreed by the whole team. For example, ribbed slabs can be constructed in many ways: in-situ using polypropylene, GRP or expanded polystyrene moulds; precast as ribbed slabs or as double-tees or by using combinations of precast and in-situ concrete. Each method has implications on the standard of finish and cost.

2.3.9 Service integration

Services and structural design must be coordinated.

Horizontal distribution of services must be integrated with structural design. Allowances for ceiling voids, especially at beam locations, and/or floor service voids should be agreed. Above false ceilings, level soffits allow easy distribution of services. Although downstand beams may disrupt service runs they can create useful room for air-conditioning units, ducts and their crossovers.

Main vertical risers will usually require large holes, and special provisions should be made in core areas. Other holes may be required in other areas of the floor plate to accommodate pipes, cables, rain water outlets, lighting, air ducts, and so forth. These holes may significantly affect the design of slabs, e.g. flat slabs with holes adjacent to columns. In any event, procedures must be established to ensure that holes are structurally acceptable.

2.4 Feasible options

2.4.1 General principles

Concrete can be used in many different ways and often many different configurations are feasible. However, market forces, project requirements and site conditions affect the relative economics of each option. The chart in Figure 2.2 has been prepared to show the generally accepted economic ranges of various types of floor under normal conditions.

Minimum material content alone does not necessarily give the best value or most economic solution in overall terms. Issues such as buildability, repeatability, simplicity, aesthetics, thermal mass and, notably, speed must all be taken into account.

Whilst a superstructure may only represent 10% of new build costs, it has a critical influence on the whole construction process and ensuing programme. Time-related costs, especially those for multi-storey structures, have a dramatic effect on the relative economics of particular types of construction.

2.4.2 Concrete options

Certain techniques tend to suit particular building sectors. The following guidance is given but is subject to the requirements of a particular project, market forces and so forth.

Commercial

Up to about 8 or 9 m span in-situ flat slabs are popular as they provide speed and flexibility at minimum cost. Up to 12 or 13 m spans post-tensioned flat slabs are economical. For longer spans up to 18 m, one-way post-tensioned slabs on post-tensioned band beams provide an office solution that avoids the constraint of integrating services and structure. Ribbed slabs provide minimum weight solutions and defined areas for penetrations. One-way slabs and beams provide very robust solutions. The use of precast concrete alone or in association with in-situ concrete, can speed construction on site.

Residential
Flat slab construction offers the thinnest possible structural solution minimising cladding costs whilst comfortably meeting acoustic requirements. Increasingly these slabs are being post-tensioned, so making them 25% thinner than conventional flat slabs.

For hotels and student accommodation, tunnel form construction and precast crosswall are economic and fast to build. They take advantage of the cellular architecture by treating the separating walls as structure, thereby minimising or eliminating the time to erect the internal partitions. Both tunnel form and crosswall can include with openings for two- and three-bedroom apartments.

Retail
Adaptability is an important design issue in this sector. The ability to meet tenant demands may mean being able to accommodate large voids (e.g. escalators) and high imposed loads (e.g. partitions). Some design teams opt for in-situ slabs with judicious over-provision of reinforcement, incorporation of knockout panels or designing slabs as simply supported on two-way beams to allow for future non-continuity. Hybrid concrete construction, using the best of in-situ and precast concrete, can offer this flexibility too.

Schools
Concrete offers the inherent benefits of thermal mass, noise attenuation, robustness and fire resistance to this sector. The requirement to adapt classroom sizes often leads to the use of in-situ slabs (flat slab, ribbed slab or one-way slab) or precast floor planks on beams. Crosswall solutions with large openings (75% of classroom width) have also been used to provide the flexibility to join classrooms together.

Hospitals and laboratories
In the most heavily serviced buildings the flat soffits of flat slabs provide infinite flexibility during design and, more importantly, operation of services distribution. Flat slabs are also the most economic form of construction to meet vibration criteria.

Car parks
In-situ, hybrid and wholly precast solutions are popular. On-site post-tensioning and/or the use of prestressed precast units allow clear spans to be achieved economically.

2.4.3 Types of concrete frame construction

Briefly, the main differences between types of construction are summarised below, and their economic ranges are illustrated in Figure 2.2.

In-situ
- **One-way slabs (solid or ribbed)** – Economic over a wide range of spans, but supporting downstand beams affect overall economics, speed of construction and service distribution.
- **Flat slabs** – With flat soffits, quick and easy to construct and usually most economic, but holes, deflection and punching shear require detailed consideration.
- **Troughed slabs** – Slightly increased depths, formwork costs and programme durations offset by lighter weight, longer spans and greater adaptability.
- **Band beam-and-slab** – Very useful for long spans in rectangular panels – popular for car parks.
- **Two-way slabs** – Robust with large span and load capacities, these are popular for retail premises and warehouses, but downstand beams disrupt construction and services.
- **Waffle slabs** – May be slow, but can be useful for larger spans and aesthetics.

Precast
- **Precast and composite slabs** – Widely available and economic across a wide range of spans and loads. Speed and quality on site may be offset by lead-in times.

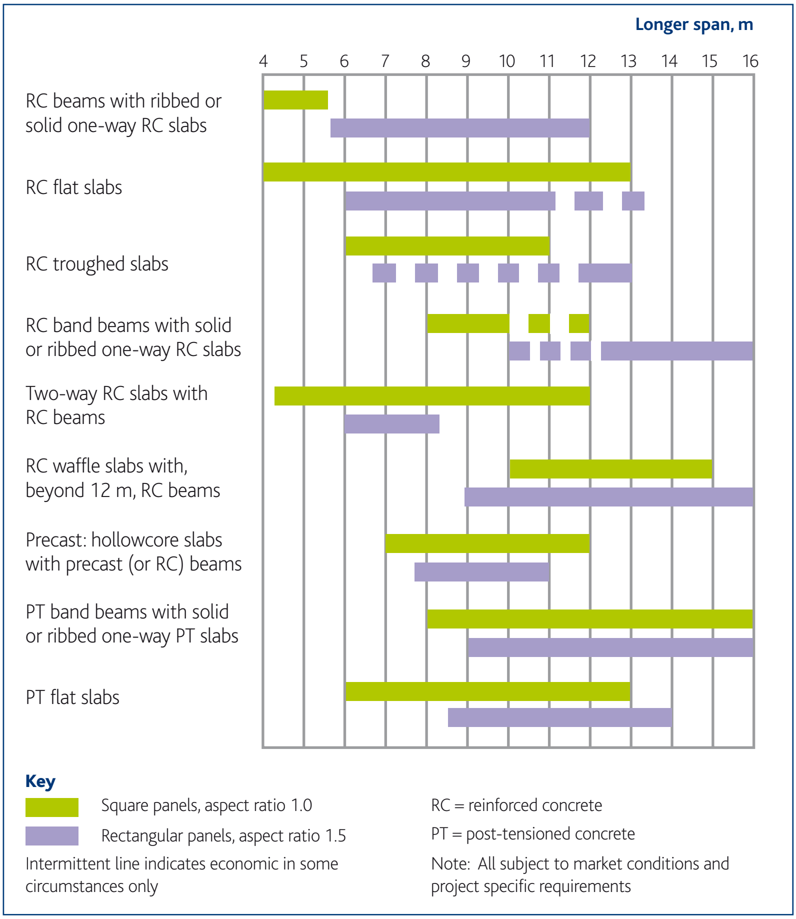
Post-tensioned
- **Post-tensioned slabs and beams** – Extend the economic span range of in-situ slabs and beams, especially useful where depth is critical.

Other forms

- **Hybrid forms of construction** – combinations of the above.
- **Tunnel-form or crosswall construction** – Can be very efficient technique for hotel or multi-storey domestic construction, as this method allows multiple uses and quick turnaround of formwork.

Whilst the charts and data have been grouped into in-situ, precast and composite, and post-tensioned concrete construction, the load information is interchangeable. In other words, hybrid options[8] such as precast floor units onto in-situ beams can be investigated by sizing the precast units and applying the appropriate ultimate load to the appropriate width and type of beam.

Figure 2.2
Concrete floor construction: typical economic span ranges



2.5 Determine slab thickness

Determine economic thickness from the appropriate chart(s) or data using the maximum span and appropriate characteristic imposed load (IL). The slab charts work on characteristic imposed load and illustrate thicknesses given in the data. The data includes ultimate loads to supporting beams (or columns), estimates of reinforcement and other information. The user is expected to interpolate between values of imposed load given, and to round up both the depth and ultimate loads to supports in line with his or her confidence in the design criteria used and normal modular sizing.

The design imposed load should be determined from BS EN 1991, Eurocode 1: *Actions on structures* [6], the intended use of the building and the client's requirements, and should then be agreed with the client. The slab charts highlight the following characteristic imposed loads:

- 2.5 kN/m^2 – general office loading, car parking.
- 5.0 kN/m^2 – high specification office loading, file rooms, areas of assembly.
- 7.5 kN/m^2 – plant rooms and storage loadings.
- 10.0 kN/m^2 – storage loading.

For each value of imposed load, a relatively conservative value of ψ_2 has been used in serviceability checks. The appropriateness of the value used should be checked and if necessary, adjustments should be made to the slab depth (see Section 8.1).

Except for precast double-tees, the charts and data assume 1.50 kN/m^2 for superimposed dead loading (SDL). If the design superimposed dead loading differs from 1.50 kN/m^2, the characteristic imposed load used for interrogating the charts and data should be adjusted to an equivalent imposed load, which can be estimated from Table 2.1. See also Section 8.2.4.

It should be noted that most types of slabs require beam support. However, flat slabs in general do not. Charts and data for flat slabs work on characteristic imposed load but give ultimate axial loads to supporting columns. Troughed slabs and waffle slabs (designed as two-way slabs with integral beams and level soffits) incorporate beams and the information given assumes beams of specified widths within the overall depth of the slab. These charts and data, again, work on characteristic imposed load, but give ultimate loads to supporting columns. The designs for these slabs assumed a perimeter cladding load of 10 kN/m.

The data include some information on economic thicknesses of two-way slabs with rectangular panels. The user may, with caution, interpolate from this information. With flat slabs, rectangular panels make little difference, so depths should be based on the longer span.

Table 2.1
Equivalent imposed loads, kN/m^2

Imposed load kN/m^2	Superimposed dead load kN/m^2					
	0.0	**1.0**	**2.0**	**3.0**	**4.0**	**5.0**
2.5	1.25	2.08	2.92	3.75	4.58	5.42
5.0	3.75	4.58	5.42	6.25	7.08	7.92
7.5	6.25	7.08	7.92	8.75	9.58	10.40
10.0	8.75	9.58	10.40	11.30	12.10	n/a

Note
The values in this table have been derived from 1.25(SDL – 1.5)/1.5 + IL

2.6 Determine beam sizes

2.6.1 General

For assumed web widths, determine economic depths from appropriate charts using maximum spans and appropriate ultimate applied uniformly distributed loads (uaudl) expressed in kN/m.

The beam charts work on ultimate applied uniformly distributed loads (uaudl). The user must calculate or estimate this line load for each beam considered. This load includes the ultimate reaction from slabs and ultimate applied line loads such as cladding or partitions that are to be carried by the beam. Self-weight of beams is allowed for within the beam charts and data (see Section 8.3).

For internal beams, the uaudl load usually results from supporting slabs alone. The load can be estimated by interpolating from the slab's data and, if necessary, adjusting the load to suit actual, rather than assumed, circumstances by applying an elastic reaction factor (see Section 8.3.2).

Perimeter beams typically support end spans of slabs and perimeter cladding. Again, slab loads can be interpolated from the data for slabs. Ultimate cladding loads and any adjustments required for beam self-weight should be estimated and added to the slab loads (see Section 8.3.3).

The data includes ultimate loads to supports, reinforcement and other information. The user can interpolate between values given in the charts and data, and is expected to adjust and round up both the loads and depth in line with his or her confidence in the design criteria used and normal modular sizing.

Beams supporting two-way slabs

In broad outline the same principles can be applied to beams supporting two-way slabs. Triangular or trapezoidal slab reactions may be represented by equivalent UDLs over the central ¾ of each span (see Section 8.3.4).

Point loads

Whilst this publication is intended for investigating uniformly distributed loads, central point loads can be investigated, with caution, by assuming an equivalent ultimate applied uniformly distributed load of twice the ultimate applied point load/span, in kN/m.

2.6.2 In-situ beams

The charts for in-situ reinforced beams cover a range of web widths and ultimate applied uniformly distributed loads (uaudl), and are divided into:

- Rectangular beams: e.g. isolated or upstand beams, beams with no flange, beams not homogeneous with supported slabs.
- Inverted L-beams: e.g. perimeter beams with top flange one side of the web.
- T-beams: e.g. internal beams with top flange both sides of the web.

The user must determine which is appropriate. For instance, a T-beam that is likely to have large holes in the flange at mid-span can be de-rated from a T- to an L-beam or even to a rectangular beam.

2.6.3 Precast beams

The charts and data for precast reinforced beams cover a range of web widths and ultimate applied uniformly distributed loads (uaudl). They are divided into:

- Rectangular beams: i.e. isolated or upstand beams.
- L-beams: e.g. perimeter beams supporting hollowcore floor units.
- (Inverted) T-beams: e.g. internal beams supporting hollowcore floor units.

The charts assume that the beams are simply supported and non-composite, i.e. no flange action or benefit from temporary propping is assumed. The user must determine which form of beam is appropriate. The depth of hollowcore or other units is recessed within the depth of the beam; therefore there is no requirement to add the depth of the slab to the depth of the recessed precast beam.

2.6.4 Post-tensioned beams

Section 5.3.1 presents charts and data for 1000 mm wide rectangular beams with no flange action. Other rectangular post-tensioned beam widths can be investigated on a pro-rata basis, i.e. ultimate load per metre width of web (see Section 8.3.5). Additionally, data are presented for 2400 mm wide T-beams assuming full flange action.

2.7 Determine column sizes

2.7.1 General

The charts are divided into:

- Internal columns.
- Edge and (external) corner columns for beam-and-slab construction.
- Edge and (external) corner columns for flat slab construction.

The square size of internal column required can be interpolated from the appropriate chart(s) using the total ultimate axial load, N_{Ed}, typically at the lowest level. In the case of perimeter (edge and corner) columns, both the ultimate 1st order moment, M, and the ultimate axial load, N_{Ed}, are required to determine the column size. Sizing charts allow different sizes to be identified for different percentages of reinforcement content.

The total ultimate axial load, N_{Ed}, is the summation of beam (or two-way floor system) reactions and the cladding and column self-weight from the top level to the level under consideration (usually bottom). Ideally, this load should be calculated from first principles (see Section 8.4). In accordance with BS EN 1991[6], imposed loads might be reduced. However, to do so is generally unwarranted in pre-scheme designs of low-rise structures. Sufficient accuracy can be obtained by approximating the load as follows:

$$N_{Ed} = \begin{pmatrix} \text{ult. load from beams per level or ult. load from two-way slab systems per level} \\ + \text{ ult. load from cladding per storey} \\ + \text{ ult. self-weight of beam per level} \end{pmatrix} \text{x no. of floors}$$

For in-situ edge and corner columns, moment derivation charts are provided adjacent to moment:load sizing charts. The moment derivation charts allow column design moments, M, to be estimated for a range of column sizes. For relative simplicity the charts work using 1st order design moments, M, (see Sections 3.3.2 and 7.1.5).

For beam-and-slab construction, M is determined from the beam span and its ultimate applied uniformly distributed load (uaudl). For flat slab construction, M is determined from the slab span and appropriate imposed load (IL). In each case, the moment is then used with the appropriate moment:load sizing chart opposite to confirm the size and to estimate the reinforcement content. The charts assume a quoted ratio of M_y to M_z and that the columns are not slender. A method for determining moments in precast columns is given in Section 4.3.3.

Table 2.2
Moment derivation and moment:load sizing charts for perimeter columns

Column type	Beam-and-slab construction		Flat slab construction	
	Moment	Sizing	Moment	Sizing
Edge column	Figure 3.37	Figure 3.38	Figure 3.41	Figure 3.42
Corner column	Figure 3.39	Figure 3.40	Figure 3.43	Figure 3.44

2.7.2 Schemes using beams

Beam reactions can be read or interpolated from the data for beams. Reactions in two orthogonal directions should be considered, for example perimeter columns may provide end support for an internal beam and internal support for a perimeter beam. Usually the weight of cladding should have been allowed for in the loads on perimeter beams (see Section 8.3). If not, or if other loads are envisaged, due allowance must be made.

2.7.3 Schemes using two-way floor systems

Two-way floor systems (i.e. flat slabs, troughed slabs and waffle slabs) either do not require beams or else include prescribed beams. Their data include ultimate loads or reactions to supporting columns.

2.7.4 Roof loads

Other than in areas of mechanical plant, roof loadings seldom exceed floor loadings. For the purposes of estimating column loads, it is usually conservative to assume that loads from concrete roofs may be equated to those from a normal floor. Loads from a lightweight roof can be taken as a proportion of a normal floor. Around perimeters, an adjustment should be made for the usual difference in height of cladding at roof level.

2.8 Resolve stability and robustness

The charts and data are for braced frames, so the means of achieving lateral stability must be determined. This may be by providing shear walls, by using frame action in in-situ structures or by using bracing. The use of ties, especially in precast structures, must also be considered.

2.9 Identify best value options

Having determined sizes of elements, the quantities of concrete and formwork can be calculated and reinforcement estimated. By applying rates for each material, a rudimentary cost comparison of the feasible options can be made. Concrete, formwork and reinforcement in floor plates constitute up to 90% of superstructure costs. Due allowances for market conditions, site constraints, differences in timescales, cladding and foundation costs should be included when determining best value and the most appropriate option(s) for further study.

As part of this process, visualize the construction process. Imagine how the structure will be constructed. Consider buildability and the principles of value engineering. Consider timescales, the flow of labour, plant and materials. Whilst a superstructure may represent only 10% of new build costs, it has a critical influence on the construction process and ensuing programme. Consider the impact of the superstructure options on service integration, also types, sizes and programme durations of foundations and substructures (see Section 9).

2.10 Prepare scheme designs

Once preferred options have been identified, full scheme design should be undertaken by a suitably experienced engineer to confirm and refine sizes and reinforcement estimates. These designs should be forwarded to the remaining members of the design team, for example the architect for coordination and dimensional control, and the cost consultant for budget costing.

The final choice of frame type should be a joint decision between client, design team, and whenever possible, contractor.

2.11 Examples

2.11.1 In-situ slabs

Estimate the thickness of a continuous multiple span one-way solid slab spanning 7.0 m supporting an imposed load of 2.5 kN/m^2, and a superimposed dead load of 3.2 kN/m^2, as shown in Figure 2.3

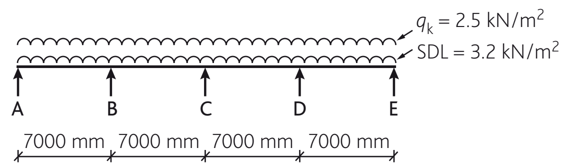


Figure 2.3
Continuous slab in a domestic structure

The **Concrete** Centre™	Project details Examples of using ECFE: In-situ slabs	Calculated by chg	Job no. CCIP – 025
		Checked by rmw	Sheet no. 1
		Client TCC	Date Oct 08

From Table 2.1, equivalent imposed load for IL = 2.5 kN/m^2 and SDL = 3.2 kN/m^2 is estimated to be 3.9 kN/m^2.

From Figure 3.1, interpolating between lines for IL = 2.5 kN/m^2 and IL = 5.0 kN/m^2, depth required is estimated to be 215 mm.

Alternatively, interpolating from one-way solid slab data (Table 3.1b), multiple span, at 3.9 kN/m^2, between 2.5 kN/m^2 (195 mm) and 5 kN/m^2 (216 mm), then:

Thickness = 195 + (216 – 195) x (3.9 – 2.5) / (5.0 – 2.5)
= 195 + 21 x 0.56
= 207 mm

Say, 210 mm thick solid slab.

2.11.2 Internal beams

Estimate the size of internal continuous beams spanning 8.0 m required to support the solid slab in Example 2.11.1 above.

The Concrete Centre™	Project details Examples of using ECFE: Internal beams	Calculated by chg	Job no. CCIP – 025
		Checked by rmw	Sheet no. 1
		Client TCC	Date Oct 08

Interpolating internal support reaction from one-way solid slab data (Table 3.1b), multiple span, at 3.9 kN/m^2, between 2.5 kN/m^2 (82 kN/m) and 5 kN/m^2 (113 kN/m), then:

Load = 82 + (3.9 – 2.5) x (113 – 82) / (5.0 – 2.5)
= 100 kN/m

Applying an elastic reaction factor of 1.1 (see Section 8.3.2), then:

Load to beam = 100 x 1.1
= 110 kN/m

Interpolating from the chart for, say, a T-beam with a 900 mm web, multiple span (Figure 3.31) at 8 m span and between loads of 100 kN/m (404 mm) and 200 kN/m (459 mm), then:

Depth = 404 + (459 – 404) x (110 – 100) / (200 – 100)
= 404 + 5
= 409 mm

Say, 900 mm wide by 425 mm deep internal beams.

2.11.3 Perimeter beams

Estimate the perimeter beam sizes for the slab in the examples above. Perimeter curtain wall cladding adds 3.0 kN/m (characteristic) per storey.

The Concrete Centre™	Project details Examples of using ECFE: Perimeter beams	Calculated by chg	Job no. CCIP – 025
		Checked by rmw	Sheet no. 1
		Client TCC	Date Oct 08

a) For perimeter beam perpendicular to slab span

Interpolating end support reaction from one-way solid slab data (Table 3.1b), multiple span, at 3.9 kN/m^2, between 2.5 kN/m^2 (41 kN/m) and 5 kN/m^2 (56 kN/m), then:

Load from slab = 41 + (3.9 – 2.5) x (56 – 41) / (5.0 – 2.5)
= 50 kN/m

Load from cladding = 3 x 1.25
= 3.8 kN/m

(Note the use of Exp. (6.10b) is assumed, so γ_G = 1.25 (See Section 8.1)

Total load = 50 + 3.8
= 53.8, say, 54 kN/m

Beam size: interpolating from L-beam chart and data, multiple span, say, 450 mm web width (Figure 3.20), at 54 kN/m over 8 m. At 50 kN/m suggested depth is 404 mm; at 100 kN/m suggested depth is 469 mm, then:

Depth required = 404 + (54 – 50) / (100 – 50) x (469 – 404)
= 409 mm

b) For perimeter beams parallel to slab span

Allow, say, 1 m of slab, then:

Load from slab = (0.21 x 25 + 3.2) x 1.25 + 2.5 x 1.5
= 14.3 kN/m

Load from cladding = 3.8 kN/m

Total load = 18.1 kN/m (ult.)

Beam size: reading from L-beam chart and data, multiple span, say, 300 mm web width (Figure 3.19 and Table 3.19), at 25 kN/m over 7 m, suggested depth is 307 mm.

For edges perpendicular to slab span, use 450 x 410 mm deep edge beams;

for edges parallel to slab span, 300 x 310 mm deep edge beams can be used.

For simplicity, use say, 450 x 425 mm deep edge beams all round.

2.11.4 Columns

Estimate the column sizes for the above examples assuming a three-storey structure as illustrated in Figure 2.4 with a floor-to-floor height of 3.5 m.

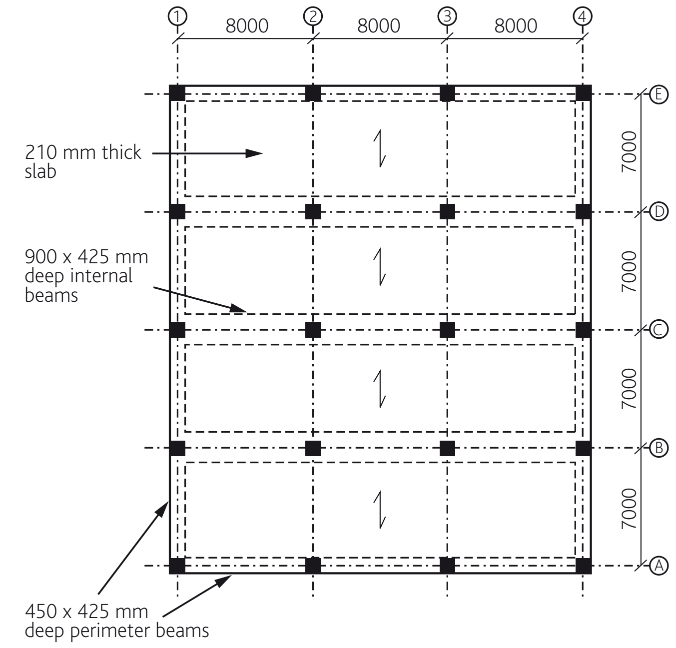


Figure 2.4
Floor arrangement

Method

For internal columns estimate the ultimate axial load, N_{Ed}, then size from chart or data.

For edge and corner columns follow the procedure below:

1. Estimate the ultimate axial load, N_{Ed}, from beam (or slab) reactions and column self-weight.
2 Estimate (1st order) design moment, M, by assuming a column size, then estimate moment by using the appropriate moment derivation chart.
3. From the moment:load chart for the assumed size, axial load and moment, estimate the required reinforcement.
4. Confirm column size or iterate as necessary.

The Concrete Centre™	Project details: Examples of using ECFE: Columns	Calculated by: chg	Job no.: CCIP – 025
		Checked by: rmw	Sheet no.: 1
		Client: TCC	Date: Oct 08

a) Beam reactions

Internal beam reactions

The internal beams are T-beams 900 wide and 425 mm deep, carrying a uaudl of 110 kN/m spanning 8 m.

From data (see Table 3.31) for 100 kN/m and 8 m span, internal support reaction = 868 kN x 110/100 (adjustment for 110 kN/m load) x 1.10 (adjustment made for elastic reactions; see Section 8.3.2) = 1050 kN*.

*Alternatively, this load may be calculated as follows:

Span x uaudl (see 2.11.2)	=	8 x 1.1 x 110 = 968 kN
Self-weight	=	0.9 x (0.425 – 0.21) x 8 x 25 x 1.25 x 1.1 = 53 kN.
Total	=	1021 kN.

End support reaction = 434 kN x 110/100 = 477 kN.

Reactions for edge beams perpendicular to slab span
These edge beams are L-beams, 450 mm wide by 425 mm deep, carrying a uaudl of 54 kN/m, with a span of 8 m.

By interpolating from data (Table 3.20) and applying an elastic reaction factor,
internal support reaction = 434 kN x 54/50 x 1.10
= 516 kN.

End support reaction = 217 x 54/50
= 234 kN.

Reactions for edge beam parallel to slab span
These edge beams are L-beams 450 mm wide by 425 mm deep, carrying a uaudl of 18 kN/m (including cladding) over 7 m spans. As no tabulated data is available, calculate reactions.

Self-weight of beam = 0.45 x 0.425 x 25 x 1.25
= 6 kN/m.

Therefore internal support reaction = (18 + 6) x 7 x 1.1
= 185 kN.

End support reaction = (18 + 6) x 7 / 2
= 84 kN.

Figure 2.4 shows the floor arrangement and beam reactions. The same exercise could be done for the roof and ground floor. But in this example it is assumed that roof loads equate to suspended slab loads and that the ground floor is supported by the ground.

b) Self-weight of columns

Assume 450 mm square columns and 3.5 m storey height (3.075 m from floor to soffit).

From Table 8.11 in Section 8.4.2 allow, say, 20 kN/storey or calculate:
0.45 x 0.45 x 3.1 x 25 x 1.25 = 19.6 kN.

But use, say, 25 kN per floor.

Total ultimate axial loads, N_{Ed}, in the columns

Internal: (1050 + 0 + 25) kN x 3 storeys = 3225 kN, say, 3250 kN.

Edge parallel to slab span: (185 + 477 + 25) x 3 = 2061 kN, say, 2100 kN.

Edge perpendicular to slab span: (516 + 0 + 25) x 3 = 1608 kN, say, 1650 kN.

Corner: (234 + 84 + 25) x 3 = 1029 kN, say, 1050 kN.

c) Sizing columns (see Figure 2.5)

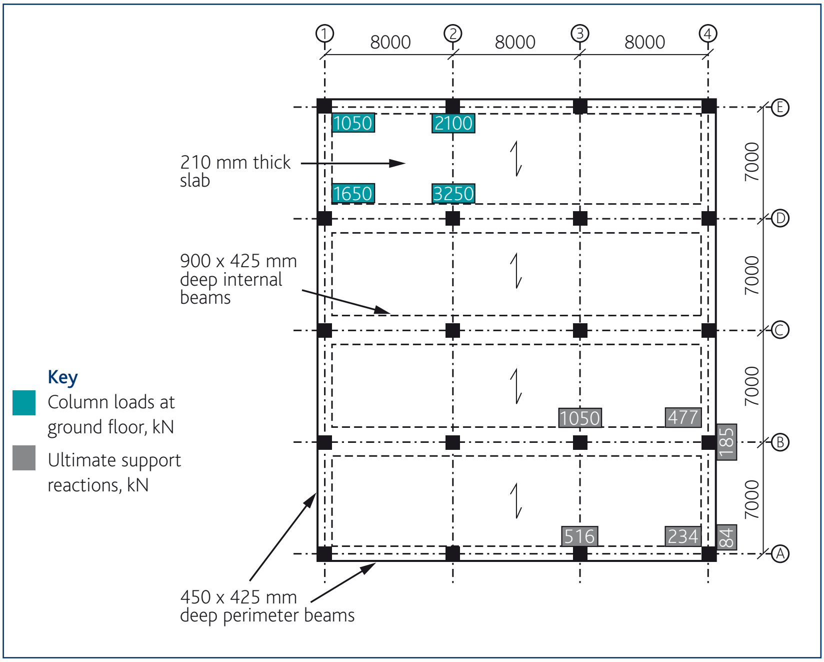


Figure 2.5
Floor arrangement, column loads and beam reactions

Internal column

From Figure 3.35, for a load of 3250 kN.

A 400 mm square column would require approximately 1.8% reinforcement.

A 375 mm square column would require approximately 2.5% reinforcement.

Try 400 mm square with 1.8% reinforcement provided by 4 no. H32s, approximately 228 kg/m^3 (from Figure 3.45).

Edge column for 1650 kN over 3 storeys (Grids 1 & 4)

As internal beam frames into column, use beam and column data.

From Figure 3.37 for beam of internal span of 8 m supporting a uaudl of 110 kN/m, for a 400 mm square column (Figure 3.37c).

Column moment $\approx$ 235 kNm.

From Table 3.36, increase in moment for a 3.5 m storey height rather than one of 3.75 m = 5%.

Therefore column moment = 1.05 x 235 = 247 kNm.

For a 400 mm square column supporting 1650 kN and 247 kNm, from Figure 3.38c, assuming columns above and below.

Reinforcement required = 3.0%.

For a 500 mm square column (Figure 3.37d), column moment $\approx$ 300 kNm.

From Table 3.36, increase in column moment = 3%.
Therefore column moment = 1.03 x 300 = 309 kNm.

Interpolating from Figure 3.38d for a 500 mm square column supporting 1650 kN and 309 kNm.

Reinforcement required = 0.6%.

Out of preference use a 400 mm square with 3.2% reinforcement provided by (from Figure 3.45) 4 no. H32s plus 4 no. H25s approximately 476 kg/m^3.

Edge column for 2100 kN over 3 storeys (Grids A & E)

Despite the presence of an edge beam, the slab will tend to frame into the column, therefore treat as flat slab with average slab span = say 7.5 m and IL = 3.9 kN/m^2 in two directions as before.

Try 400 mm square column as other edge.

Interpolating Figure 3.41c for a 400 mm square column for 3.9 kN/m^2.

Column moment = 110 kNm.

From Table 3.38, assuming columns above and below, increase in column moment = 2%.
Therefore column moment = 1.02 x 110 = 112 kNm.

Interpolating Figure 3.42c for a 400 mm square column supporting 1650 kN and 110 kNm.
Reinforcement required = 0.3% (nominal).

From Figure 3.45, use 400 mm square with, say, 4 no. H25s (1.2%: 137 kg/m^3).

Corner columns for 1050 kN over 3 storeys

From Figure 3.39c for an 8 m beam span supporting a uaudl of 54 kN/m for a 400 mm square column.

Column moment is approximately 150 kNm.

From Table 3.37, assuming columns above and below.

Increase in column moment = 8%.

Therefore column moment = 1.08 x 150 = 162 kNm.

From Figure 3.40c, for 1050 kN and 162 kNm.

Reinforcement required = 1.6%.

From Figure 3.45 try 400 mm square with 4 no. H32s (2.08% : 228 kg/m^3).

Suggested column sizes: 400 mm square.

Commentary:

The perimeter columns are critical to this scheme. If this scheme is selected, these columns should be checked by design. Nonetheless, compared with the design assumptions made for the column charts, the design criteria for these particular columns do not appear to be harsh. It is probable that all columns could therefore be rationalised to, say, 375 mm square, without the need for undue amounts of reinforcement.

2.11.5 Flat slab scheme

Estimate the sizes of columns and slabs in a seven-storey building, five bays by five bays, 3.3 m floor to floor. The panels are 7.5 m x 7.5 m. Characteristic imposed load is 5.0 kN/m^2, and superimposed dead load is 1.5 kN/m^2. Curtain wall glazing is envisaged at 0.6 kN/m^2 on elevation. Approximately how much reinforcement would there be in such a superstructure?

The Concrete Centre	Project details: Examples of using ECFE: Flat slab scheme	Calculated by: chg	Job no.: CCIP – 025
		Checked by: rmw	Sheet no.: 1
		Client: TCC	Date: Oct 08

a) Slab

Interpolating from the solid flat slab chart and data (Figure 3.7 and Table 3.7), at 5.0 kN/m^2 and 7.5 m, the slab should be (246 + 284)/2 = 265 mm thick.
Say, 275 mm thick with approximately (91 + 92)/2 = 92 kg/m^3 reinforcement.
Assume roof is similar.
Similarly for plant room, but for 7.5 kN/m^2, thickness = 323 mm.
Say 325 mm thick at 80 kg/m^3.

b) Columns

The minimum square sizes of columns should be 375 mm (from Table 3.7, at 5.0 kN/m^2, average of 350 mm at 7 m and 400 mm at 8 m, to avoid punching shear problems).

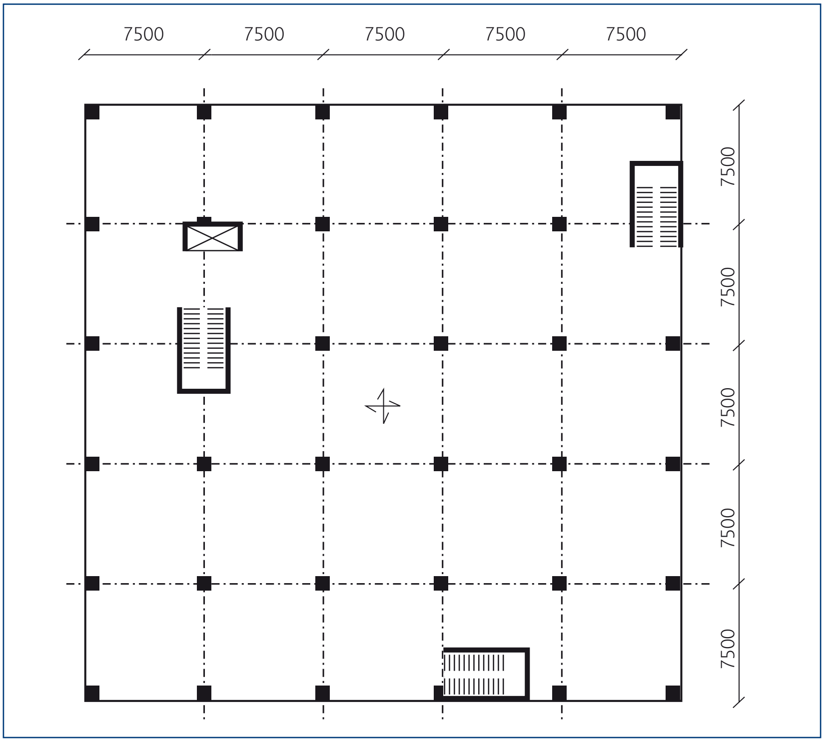


Figure 2.6
Flat slab scheme

Internal

From the flat slab data Table 3.7, and allowing an elastic reaction factor of 1.1 (see Section 8.4.5).

Ultimate load to internal column for IL of 5.0 kN/m^2 is (836 + 1167)/2 x 1.1

= 1001.5 say 1025 kN per floor.

Allow 25 kN per floor for ultimate self-weight of column.

Total axial load, (assuming roof loads = floor loads) N_{Ed} = (1025 + 25) x 7 = 7350 kN.

From internal column chart, Figure 3.35, at 7350 kN, the internal columns could, assuming the use of Grade C30/37 concrete, be 525 mm square, that is, greater than that required to avoid punching shear problems.

They would require approximately 3.4% reinforcement at the lowest level.

From Figure 3.45, provide say 8 no. H40s (3.65%), about 435 kg/m^3, including links.

This amount of reinforcement could be reduced by using a higher concrete grade for the columns. Reinforcement densities will also reduce going up the building. Therefore, use 525 mm square columns.

Allow, say, 66% of 435 kg/m^3 ≈ 300 kg/m^3 for estimating purposes.

Edge

From the flat slab data Table 3.7.

Ultimate load to edge columns is (418 + 584)/2

= 501 kN per floor.

Cladding: allow 7.5 x 3.3 x 0.6 x 1.25 = 18.5, say 19 kN.

Allow 25 kN per floor for ultimate self-weight of column.

Total axial load, N_{Ed}= (501 + 19 + 25) x 7 = 3815 kN.

From Figure 3.41c, (the moment derivation chart for a 400 mm square edge column in flat slab construction,) interpolating for an imposed load of 5.0 kN/m^2 and a 7.5 m span, for f_{ck} = 30 MPa and columns above and below, the 1st order design moment, M, is approximately 120 + 4% (allowance of 4% extra for a 3.3 m storey height, see Table 3.38) = 125 kNm.

From Figure 3.42c a 400 mm column with N_{Ed} = 3815 kN and M_{OEd} = 125 kNm would require approximately 4.7% reinforcement.

Assuming the use of a 500 mm square column, N_{Ed} = 3815. From Figure 3.41d, for an imposed load of 5.0 kN/m^2 and a 7.5 m span, M = 125 + 2% = say 128 kNm allowing 2% extra for a 3.3 m storey height from Table 3.38, and from Figure 3.42d about 1.0% reinforcement would be required.

Neither 400 mm nor 500 mm square columns provide an ideal solution, so presume the use of a 450 mm square column with approximately 2.85% reinforcement.

Punching shear: as 450 mm > 375 mm minimum, OK.

Use 450 mm square columns.

From Figure 3.45 provide maximum of 8H32 (356 kg/m^3) and allow average of 240 kg/m^3.

Corner
Load per floor will be approximately (418 x 584)/4 = 250 kN per floor.

Self-weight of column, say, = 25 kN per floor.

Cladding = 19 kN per floor as before.

Total = 250 + 19 + 25 = 294 kN per floor.

N_{Ed} = 294 x 7 floors = 2058 kN.

From corner column charts (Figures 3.43c and 3.44c) moment for a 400 mm square column, $M \approx 90$ kNm leading to a requirement of approximately 4.0% reinforcement. No adjustment for storey height is required.

For a 500 mm square column, $M \approx 105$ kNm and 1.1% reinforcement would be required.

Again the use of 450 mm square columns would appear to be the better option. Assume require max 2.55%. Punching shear OK.

Use 450 mm square columns.

Assume reinforcement for corner columns is same as for edge columns.

Edge and corner
To simplify quantities, take all perimeter columns as 450 mm square; average reinforcement density at 2.85% maximum 356 kg/m^3, but use average of say 240 kg/m^3.

c) Walls

From Table 6.2 assuming 200 mm thick walls, reinforcement density is approximately 35 kg/m^3. Allow 41 m of wall on each floor.

d) Stairs

From Table 6.3, say 5 m span and 4.0 kN/m^2 imposed load, reinforcement density is approximately 14 kg/m^2 (assume landings included with floor slab estimate). Assume 30 flights 1.5 m wide.

e) Reinforcement quantities

Slabs	= $(7.5 \times 5 + 0.5)^2 \times 7 \times 0.275 \times 92/1000$	=	256
Internal columns	= $0.525^2 \times 3.3 \times 16 \times 7 \times 300/1000$	=	31
Perimeter columns	= $0.45^2 \times 3.3 \times 20 \times 7 \times 240/1000$	=	23
Walls, say	= $41 \times 3.3 \times 0.2 \times 7 \times 35/1000$	=	7
Stairs, say	= 30 flights $\times 5 \times 1.5 \times 14 \times 30/1000$	=	3
Plant room, say	= $7.5 \times 7.5 \times 3 \times 1 \times 0.325 \times 80/1000$	=	4
Plant room columns, say	= $0.525^2 \times 3.3 \times 8 \times 200/1000$	=	2
Total, approximately		=	326 tonnes

Scheme summary

Use 275 mm flat slabs with 525 mm square internal columns and 450 mm square perimeter columns. Reinforcement required for the superstructure would be about 330 tonnes (but see Section 2.2.4). This excludes reinforcement to ground floor slabs and foundations.

3 In-situ concrete construction

Figure 3.A
Indescon Court, Phase 1, London E14. These residential blocks consist of flat slab construction above retail and commercial units and basement car parking.
Photo courtesy of Grant Smith

3.1 In-situ slabs

3.1.1 Using in-situ slabs

In-situ slabs offer economy, versatility and inherent robustness. They can easily accommodate large and small service holes, fixings for suspended services and ceilings, and cladding support details. Also, they can be quick and easy to construct. Each type has implications on overall costs, speed, self-weight, storey heights and flexibility in use: the relative importance of these factors must be assessed in each particular case.

3.1.2 The charts and data

The charts and data give overall depths against spans for a range of characteristic imposed loads (IL). An allowance of 1.5 kN/m^2 has been made for superimposed dead loads (finishes, services, etc.).

Where appropriate, the charts and data are presented for both single simply supported spans and the end span of three continuous spans. Continuity allows the use of thinner, more economic slabs. However, depths can often be determined by the need to allow for single spans in parts of the floor plate.

In general, charts and data assume that one-way slabs have line supports (i.e. beams or walls). The size of beams required can be estimated by noting the load to supporting beams and referring to the appropriate beam charts. See Section 2.6.

Two-way slab systems (i.e. flat slabs, troughed slabs and waffle slabs) do not, generally, need separate consideration of beams. In these cases, the ultimate load to supporting columns is given. Otherwise these charts and data make an allowance of 10 kN/m characteristic load from the slab around perimeters to allow for the self-weight of cladding (approximately the weight of a traditional brick-and-block cavity wall with 25% glazing and 3.5 m floor-to-floor height; see Section 8.3.3.).

Flat slabs are susceptible to punching shear around columns: the sizes of columns supporting flat slabs should therefore be checked. The charts and data include the minimum sizes of column for which the slab thickness is valid. The charts and data assume one 150 mm hole adjoining each column. Larger holes adjacent to columns may invalidate the flat slab charts and data unless column sizes are increased appropriately.

3.1.3 Design assumptions

Design

The charts and data are based on moments and shears from continuous slab analysis to Eurocode 2[2, 3], assuming end spans are critical and knife-edge supports. See Section 7.1.

Load factors to the least favourable of BS EN 1990[9], Expressions (6.10a) and (6.10b) have been employed throughout. If the more basic Expression (6.10) is used in design, greater slab thicknesses may be required. Values for ψ_2, the permanent portion of imposed loading, are given in Section 7.1.3.

In order to satisfy deflection criteria, the steel service stress, σ_s, has in very many cases been reduced by increasing the area of steel provided, $A_{s,\,prov}$ to a maximum of 150% as required, such that $310/\sigma_s \leq 1.5$.

Fire and durability

Fire resistance 1 hour; exposure class XC1; cover to all max[15; ϕ] + Δc_{dev} (where Δc_{dev} = 10 mm).

Concrete

C30/37; 25 kN/m^3; 20 mm aggregate.

Reinforcement

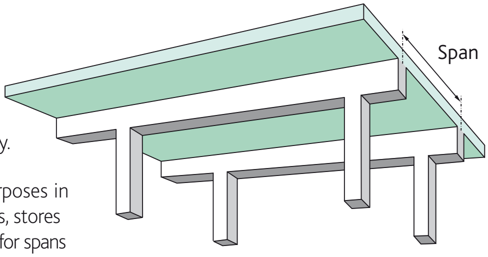
Main reinforcement and links, f_{yk} = 500 MPa. Quantities of reinforcement relate to the slabs only and not supporting beams etc. See also Section 2.2.4.

Variations

Variations from the above assumptions and assumptions for the individual types of slab are described in the relevant data. Other assumptions made are described and discussed in Section 7, *Derivation of charts and data*.

3.1.4 One-way solid slabs

One-way in-situ solid slabs are the most basic form of slab. Deflection usually governs the design, and steel content is usually increased to reduce service stress and increase span capacity.



Generally employed for utilitarian purposes in offices, retail developments, warehouses, stores and similar buildings. Can be economical for spans from 4 to 6 m.

Advantages/disadvantages

One-way in-situ solid slabs are simple to construct and the provision of holes causes few structural problems. However, the associated downstand beams may deter fast formwork cycles and can result in greater storey height.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes. End supports min. 300 mm wide.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included.
ψ_2 factors – For 2.5 kN/m^2 , ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6 and for 10.0 kN/m^2, ψ_2 = 0.8.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. Main bar diameters and distribution steel as required. To comply with deflection criteria, service stress, σ_s, may have been reduced. Top steel provided in mid-span.

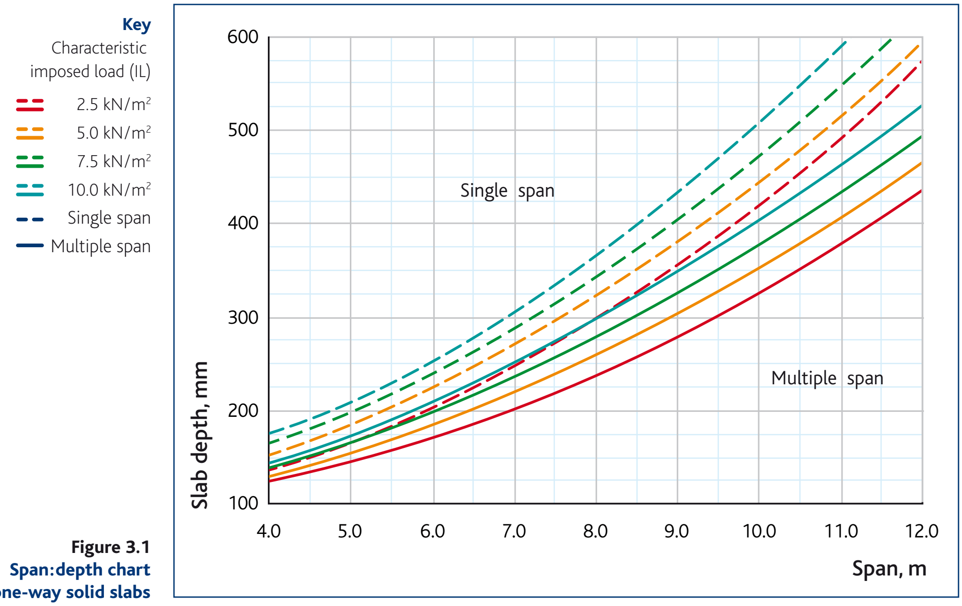


Figure 3.1
Span:depth chart for one-way solid slabs

Table 3.1a

Data for one-way solid slabs: single span

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m^2	138	171	204	242	291	345	430	489	561
IL = 5.0 kN/m^2	152	188	227	264	317	381	443	510	
IL = 7.5 kN/m^2	164	200	241	279	342	404	470	545	
IL = 10.0 kN/m^2	173	213	252	297	361	429	508		
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	n/a (20)	n/a (27)	n/a (36)	n/a (46)	n/a (59)	n/a (74)	n/a (96)	n/a (116)	n/a (142)
IL = 5.0 kN/m^2	n/a (28)	n/a (38)	n/a (49)	n/a (62)	n/a (77)	n/a (96)	n/a (116)	n/a (139)	
IL = 7.5 kN/m^2	n/a (36)	n/a (48)	n/a (62)	n/a (76)	n/a (95)	n/a (101)	n/a (139)	n/a (166)	
IL = 10.0 kN/m^2	n/a (46)	n/a (61)	n/a (77)	n/a (95)	n/a (117)	n/a (125)	n/a (171)		
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m^2	6 (43)	8 (49)	11 (53)	15 (60)	19 (64)	19 (55)	20 (47)	30 (62)	30 (54)
IL = 5.0 kN/m^2	7 (49)	10 (55)	12 (55)	18 (68)	19 (59)	23 (61)	30 (68)	30 (60)	
IL = 7.5 kN/m^2	8 (50)	11 (54)	15 (61)	18 (65)	19 (55)	23 (58)	30 (64)	31 (56)	
IL = 10.0 kN/m^2	10 (60)	14 (68)	18 (71)	22 (75)	28 (79)	30 (70)	31 (60)		
Variations: overall slab depth, mm, for IL = 5.0 kN / m^2									
2 hours fire	163	198	233	271	324	381	443	510	
4 hours fire	191	225	262	299	353	411	474	542	
Exp. XD1 + C40/50	169	204	242	280	333	393	456	523	

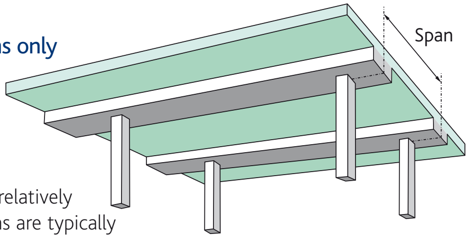
Table 3.1b

Data for one-way solid slabs: multiple span

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m^2	125	141	167	195	236	277	321	369	440
IL = 5.0 kN/m^2	128	156	184	216	257	301	349	407	461
IL = 7.5 kN/m^2	136	166	198	227	273	321	378	432	489
IL = 10.0 kN/m^2	144	176	206	237	293	347	402	460	530
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	38 (19)	50 (25)	65 (33)	82 (41)	104 (52)	128 (64)	156 (78)	189 (94)	234 (117)
IL = 5.0 kN/m^2	53 (27)	71 (36)	91 (45)	113 (56)	139 (70)	169 (84)	203 (101)	243 (121)	285 (143)
IL = 7.5 kN/m^2	69 (35)	92 (46)	116 (58)	141 (71)	173 (87)	208 (104)	249 (125)	293 (146)	341 (170)
IL = 10.0 kN/m^2	88 (44)	115 (57)	144 (72)	175 (88)	215 (108)	259 (129)	306 (153)	358 (179)	419 (209)
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m^2	6 (48)	7 (53)	9 (55)	12 (63)	13 (54)	15 (55)	16 (49)	19 (52)	24 (54)
IL = 5.0 kN/m^2	8 (60)	10 (64)	12 (67)	14 (67)	17 (67)	20 (68)	22 (62)	26 (65)	27 (59)
IL = 7.5 kN/m^2	9 (69)	13 (76)	15 (74)	17 (77)	20 (75)	22 (68)	26 (70)	27 (64)	34 (69)
IL = 10.0 kN/m^2	12 (85)	14 (82)	18 (87)	22 (91)	25 (86)	26 (76)	33 (81)	34 (74)	35 (66)
Variations: overall slab depth, mm, for IL = 5.0 kN/m^2									
2 hours fire	139	166	194	222	264	308	356	407	
4 hours fire	166	193	221	250	293	338	386	437	492
Exp. XD1 + C40/50	144	172	200	231	273	318	366	419	474

3.1.5 One-way slabs

for use with 2400 mm wide band beams only



Used in car parks, schools, shopping centres, offices and similar buildings where spans in one direction predominate and live loads are relatively light.

Slabs effectively span between edges of the relatively wide and shallow band beams. Overall depths are typically governed by deflection and the need to suit formwork, and so the beam downstands are ideally restricted to 150 mm. Perimeter beams may take the form of upstands. Economic for slab spans up to 10 m (centreline support to centreline support) and band beam spans up to 15 m.

Advantages/disadvantages

Providing medium-range spans, these slabs are fast and simple to construct and can accommodate large and small holes. They also facilitate the distribution of horizontal services, but the associated downstand beams may result in greater storey height, and can deter fast formwork cycles.

Design assumptions

Supported by – Internally, 2400 mm wide beams (1200 mm wide at edges, assuming 250 mm square edge columns). Refer to beam charts to estimate sizes.
Dimensions – Square panels, minimum of two (for end spans) or three slab spans x three beam spans.
Spans – Multiple spans assumed. Spans quoted in charts and data are centreline of support to centreline of support (e.g. grid to grid). However, the designs of these slabs are based on internal spans of (span – 2.4 m + h) and end spans of (span – 1.2 m + h/2) where h is overall depth of the slab.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included.
ψ_2 factors – For 2.5 kN/m^2 ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6 and for 10.0 kN/m^2, ψ_2 = 0.8.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. Main bar diameters and distribution steel as required. To comply with deflection criteria, service stress, σ_s, may have been reduced. Top steel provided in mid-span.

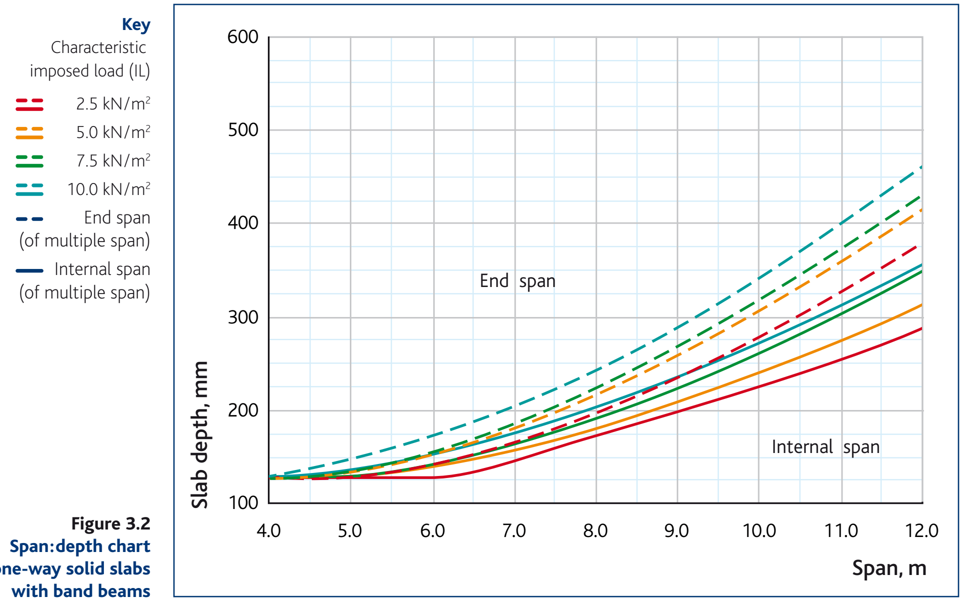


Figure 3.2
Span:depth chart for one-way solid slabs with band beams

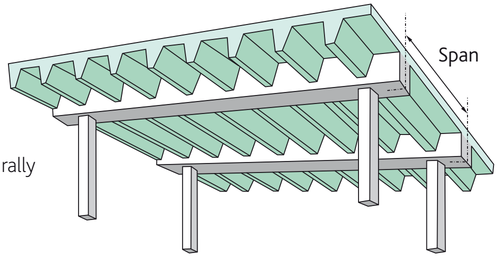
Table 3.2a
Data for one-way solid slabs with band beams: end span

END span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m²	125	125	137	163	192	232	274	326	374
IL = 5.0 kN/m²	125	127	152	177	208	258	305	353	404
IL = 7.5 kN/m²	125	131	155	186	217	262	310	373	429
IL = 10.0 kN/m²	125	145	171	204	239	287	341	403	
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m²	38 (19)	48 (24)	59 (30)	75 (37)	93 (46)	116 (58)	142 (71)	174 (87)	208 (104)
IL = 5.0 kN/m²	53 (27)	67 (33)	85 (42)	104 (52)	127 (63)	157 (78)	189 (94)	224 (112)	264 (132)
IL = 7.5 kN/m²	68 (34)	86 (43)	108 (54)	133 (66)	159 (80)	192 (96)	228 (114)	272 (136)	318 (159)
IL = 10.0 kN/m²	85 (42)	110 (55)	137 (68)	167 (84)	201 (100)	240 (120)	285 (143)	337 (168)	
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m²	6 (47)	8 (62)	12 (86)	12 (74)	14 (74)	17 (75)	18 (66)	22 (66)	27 (73)
IL = 5.0 kN/m²	7 (53)	12 (97)	16 (103)	19 (107)	20 (95)	24 (93)	25 (81)	31 (87)	32 (78)
IL = 7.5 kN/m²	8 (63)	16 (123)	19 (126)	24 (130)	24 (113)	30 (114)	32 (103)	33 (88)	41 (95)
IL = 10.0 kN/m²	11 (89)	17 (119)	21 (120)	25 (124)	31 (130)	31 (107)	32 (94)	39 (97)	
Variations: overall slab depth, mm, for IL = 5.0 kN/m²									
2 hours fire	125	135	159	184	215	259	305	OK	OK
4 hours fire	136	162	186	212	244	288	336	385	436
Exp. XD1 + C40/50	125	131	155	178	210	250	292	334	379

Table 3.2b
Data for one-way solid slabs with band beams: internal span

INTERNAL span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m²	125	125	125	155	176	197	220	239	274
IL = 5.0 kN/m²	125	125	135	160	180	197	242	272	309
IL = 7.5 kN/m²	125	125	141	161	188	218	257	300	346
IL = 10.0 kN/m²	125	131	151	174	204	233	264	307	359
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m²	19 (n/a)	24 (n/a)	29 (n/a)	37 (n/a)	44 (n/a)	53 (n/a)	62 (n/a)	72 (n/a)	85 (n/a)
IL = 5.0 kN/m²	27 (n/a)	33 (n/a)	41 (n/a)	50 (n/a)	60 (n/a)	70 (n/a)	85 (n/a)	98 (n/a)	114 (n/a)
IL = 7.5 kN/m²	34 (n/a)	43 (n/a)	53 (n/a)	64 (n/a)	76 (n/a)	90 (n/a)	106 (n/a)	124 (n/a)	144 (n/a)
IL = 10.0 kN/m²	42 (n/a)	54 (n/a)	66 (n/a)	80 (n/a)	96 (n/a)	112 (n/a)	130 (n/a)	151 (n/a)	175 (n/a)
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m²	6 (45)	7 (52)	11 (86)	14 (89)	16 (93)	18 (91)	19 (88)	23 (97)	25 (91)
IL = 5.0 kN/m²	6 (51)	8 (66)	14 (102)	16 (99)	16 (91)	24 (122)	24 (97)	23 (84)	31 (99)
IL = 7.5 kN/m²	7 (57)	12 (97)	17 (120)	20 (123)	26 (136)	29 (133)	30 (116)	35 (115)	36 (104)
IL = 10.0 kN/m²	8 (64)	16(124)	20 (135)	25 (144)	30 (149)	33 (140)	39 (148)	40 (131)	40 (112)
Variations: overall slab depth, mm, for IL = 5.0 kN/m²									
2 hours fire	125	125	144	164	185	207	246	280	318
4 hours fire	125	145	167	191	211	240	261	290	322
Exp. XD1 + C40/50	125	125	131	155	177	209	221	262	297

3.1.6 Ribbed slabs



Introducing voids to the soffit of a slab reduces dead-weight and increases the efficiency of the concrete section. The profile may be expressed architecturally and/or used for passive cooling. Can be economic in the range 8 to 12 m.

Ribs should be at least 150 mm wide to suit reinforcement detailing.

Advantages/disadvantages

These lightweight slabs provide medium to long spans. Compared with solid slabs, a slightly deeper section is required, but the stiffer floors facilitate longer spans and the provision of holes. The saving in materials tends to be offset by some complication in formwork (commonly expanded polystyrene moulds on flat formwork/falsework) and reinforcement operations, which make voided slabs slower to construct.

Design assumptions

Supported by – Line supports i.e. beams or walls. For beams refer to beam charts and data.
Dimensions – Square panels, minimum of three slab spans. Ribs 150 mm wide @ 750 mm centres. Topping 100 mm. Moulds of bespoke depth. Rib/solid intersection at 300 mm from centrelines of supports.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Self-weight used accounts for 10° slope to ribs and solid ends as described above. Additional self-weight from solid areas assumed spread throughout spans.
ψ_2 factors – For 2.5 kN/m^2 ψ_2 = 0.3; for 5.0 MPa, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. H8 links. Main bar diameters as required. To comply with deflection criteria, service stress, σ_s, may have been reduced. Top steel provided in mid-span.

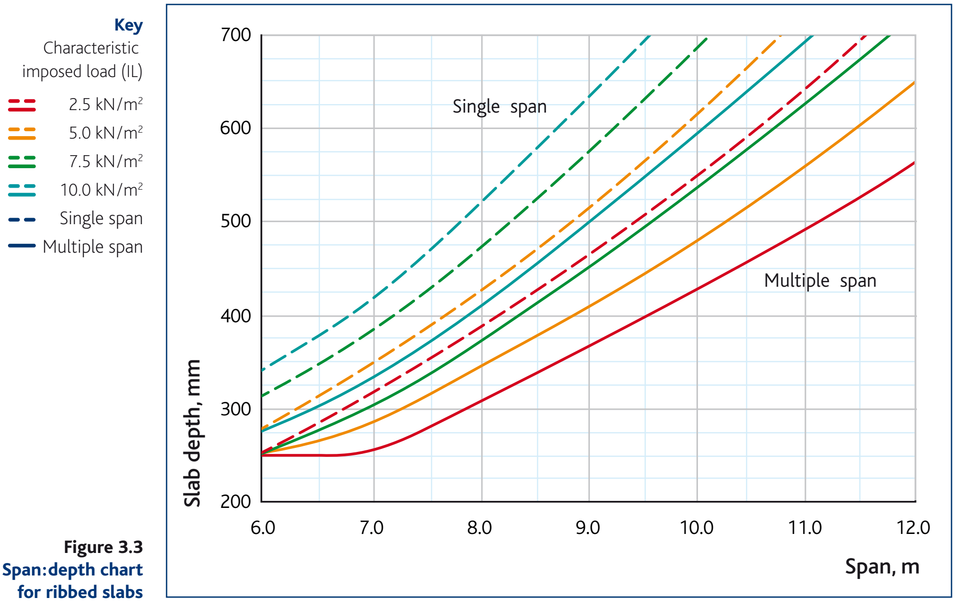


Figure 3.3
Span:depth chart for ribbed slabs

Table 3.3a

Data for ribbed slabs: single span

SINGLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm							
IL = 2.5 kN/m²	251	312	379	451	539	639	749
IL = 5.0 kN/m²	278	347	425	516	621	732	852
IL = 7.5 kN/m²	311	381	474	576	690	809	938
IL = 10.0 kN/m²	340	416	522	634	759	896	
Ultimate load to supporting beams, internal (end), kN/m							
IL = 2.5 kN/m²	n/a (30)	n/a (37)	n/a (44)	n/a (53)	n/a (64)	n/a (77)	n/a (91)
IL = 5.0 kN/m²	n/a (42)	n/a (51)	n/a (61)	n/a (73)	n/a (88)	n/a (104)	n/a (122)
IL = 7.5 kN/m²	n/a (54)	n/a (65)	n/a (78)	n/a (93)	n/a (111)	n/a (130)	n/a (152)
IL = 10.0 kN/m²	137 (68)	167 (82)	201 (98)	240 (116)	285 (138)	337 (163)	
Reinforcement, kg/m² (kg/m³)							
IL = 2.5 kN/m²	13 (84)	13 (78)	14 (73)	14 (68)	15 (60)	16 (56)	17 (51)
IL = 5.0 kN/m²	13 (83)	14 (75)	14 (69)	15 (64)	16 (57)	17 (52)	19 (50)
IL = 7.5 kN/m²	13 (77)	14 (72)	15 (65)	15 (59)	17 (53)	18 (52)	19 (47)
IL = 10.0 kN/m²	13 (73)	14 (68)	15 (62)	16 (58)	18 (53)	19 (48)	
Variations: overall slab depth, mm, for IL = 5.0 kN/m²							
2 hours fire	285	352	433	524	622	728	844
4 hours fire	300	362	428	497	571	658	760
Exp. XD1 + C40/50	260	324	400	483	579	679	788

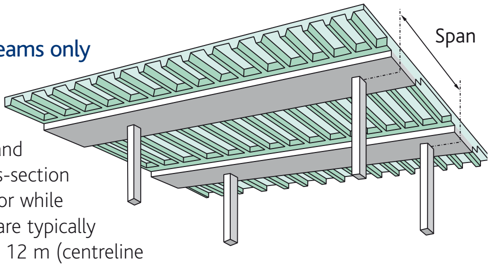
Table 3.3b

Data for ribbed slabs: multiple span

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm									
IL = 2.5 kN/m²	250	255	308	366	430	495	567	653	748
IL = 5.0 kN/m²	250	284	344	408	483	566	655	751	854
IL = 7.5 kN/m²	250	301	374	453	541	633	729	833	944
IL = 10.0 kN/m²	273	330	410	500	597	698	803	936	
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m²	60 (30)	70 (35)	84 (42)	99 (49)	117 (58)	135 (68)	157 (78)	182 (91)	212 (106)
IL = 5.0 kN/m²	82 (41)	98 (49)	116 (58)	136 (68)	160 (80)	185 (93)	214 (107)	246 (123)	283 (142)
IL = 7.5 kN/m²	105 (52)	125 (63)	148 (74)	174 (87)	204 (102)	235 (118)	270 (135)	309 (154)	353 (177)
IL = 10.0 kN/m²	132 (66)	157 (79)	186 (93)	218 (109)	255 (128)	295 (147)	338 (169)	391 (195)	
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m²	6 (40)	11 (74)	12 (71)	12 (65)	12 (59)	13 (57)	13 (51)	14 (48)	15 (46)
IL = 5.0 kN/m²	9 (57)	13 (78)	13 (70)	13 (65)	14 (60)	14 (55)	15 (52)	16 (47)	17 (46)
IL = 7.5 kN/m²	13 (86)	13 (80)	13 (71)	14 (66)	14 (57)	15 (53)	16 (48)	18 (48)	18 (44)
IL = 10.0 kN/m²	14 (86)	14 (78)	14 (70)	15 (65)	15 (57)	16 (52)	18 (49)	18 (44)	
Variations: overall slab depth, mm, for IL = 5.0 kN/m²									
2 hours fire	250	278	335	399	463	531	602	677	755
4 hours fire	250	290	343	399	459	521	589	654	724
Exp. XD1 + C40/50	250	264	319	381	453	529	610	697	790

3.1.7 Ribbed slabs

for use with 2400 mm wide band beams only



Used in car parks and offices where spans in one direction predominate and imposed loads are relatively light. The band beam has a relatively wide, shallow cross-section that reduces the overall depth of the floor while permitting longer spans. Overall depths are typically governed by deflection. Slab spans up to 12 m (centreline support to centreline support) with beam spans up to 15 m are economic.

Advantages/disadvantages

These lightweight floors provide medium to long spans that can accommodate large holes (provided the beams are avoided). The need for more complex formwork makes them slower to construct, and the floor depth is greater than the solid slab band beam option.

Design assumptions

Supported by – 2400 mm wide beams internally and 1200 mm wide beams at edges. Downstands 100 to 180 mm.
Dimensions – Square panels, minimum of three slab spans x three beam spans. Bespoke moulds, ribs 150 mm wide @ 750 mm centres. Topping 100 mm. Rib/solid intersection at 50 mm from edge of supporting beams.
Spans – Spans quoted in charts and data are centreline of support to centreline of support (e.g. grid to grid). However, the designs of these slabs are based on internal spans of (span – 2.4 m + h), and end spans of (span – 1.2 m + h/2).
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Self-weight used accounts for 10° slope to ribs and solid ends as described above.
ψ_2 factors – For 2.5 kN/m^2, $\psi_2 = 0.3$; for 5.0 kN/m^2, $\psi_2 = 0.6$; for 7.5 kN/m^2, $\psi_2 = 0.6$; and for 10.0 kN/m^2, $\psi_2 = 0.8$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. H8 links. Main bar diameters as required. To comply with deflection criteria, service stress, σ_s, may have been reduced. Top steel provided in mid-span.

Key
Characteristic imposed load (IL)
2.5 kN/m^2
5.0 kN/m^2
7.5 kN/m^2
10.0 kN/m^2

Multiple span

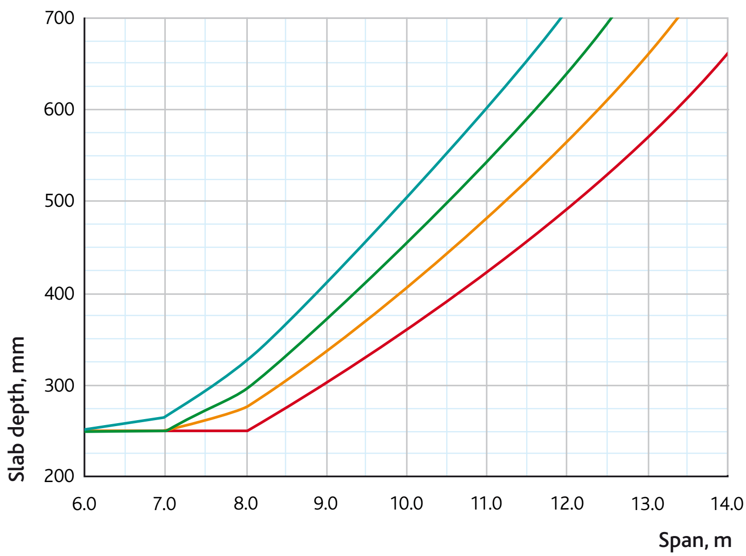
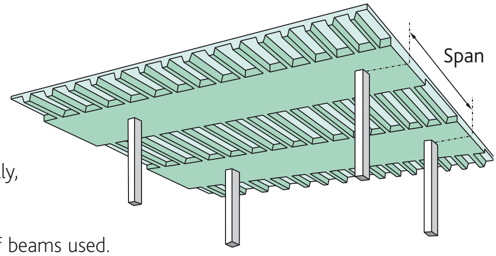


Figure 3.4
Span:depth chart for ribbed slabs with wide band beams: multiple span

Table 3.4
Data for ribbed slabs with wide band beams: multiple span

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm									
IL = 2.5 kN/m^2	250	250	250	301	359	424	491	567	660
IL = 5.0 kN/m^2	250	250	277	337	403	481	568	662	766
IL = 7.5 kN/m^2	250	250	296	369	451	542	639	743	856
IL = 10.0 kN/m^2	250	267	325	407	500	603	710	824	
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	67 (34)	77 (39)	87 (43)	103 (52)	121 (61)	142 (71)	166 (83)	191 (96)	223 (111)
IL = 5.0 kN/m^2	90 (45)	103 (52)	120 (60)	142 (71)	165 (82)	192 (96)	224 (112)	258 (129)	298 (149)
IL = 7.5 kN/m^2	112 (56)	130 (65)	152 (76)	179 (90)	209 (105)	243 (121)	282 (141)	323 (162)	370 (185)
IL = 10.0 kN/m^2	139 (69)	162 (81)	191 (96)	225 (112)	262 (131)	304 (152)	353 (176)	404 (202)	
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m^2	5 (35)	7 (46)	8 (58)	12 (77)	12 (70)	12 (63)	13 (61)	14 (54)	14 (50)
IL = 5.0 kN/m^2	6 (43)	9 (62)	13 (89)	13 (80)	13 (71)	14 (64)	14 (57)	15 (52)	17 (51)
IL = 7.5 kN/m^2	7 (50)	14 (97)	14 (90)	14 (80)	14 (68)	15 (61)	15 (55)	16 (49)	18 (46)
IL = 10.0 kN/m^2	10 (70)	14 (96)	15 (94)	16 (84)	16 (72)	16 (60)	17 (54)	18 (49)	
Variations: overall slab depth, mm, for IL = 5.0 kN/m^2									
2 hours fire	250	250	271	329	394	460	530	603	681
4 hours fire	250	250	284	338	395	456	520	591	679
Exp. XD1 + C40/50	250	250	257	312	375	449	528	613	706
175 mm wide ribs	250	250	275	334	398	475	561	660	764

3.1.8 Troughed slabs



Troughed slabs are popular in spans up to 12 m as they combine the advantages of ribbed slabs with those of level soffits. The profile may be expressed architecturally, and/or used for passive cooling.

Economic depths depend on the widths of beams used.
Deflection is usually critical to the design of the beams, which, therefore, tend to be wide and heavily reinforced.

The chart and data assume internal beam widths of beam span/3.5, perimeter beam width of beam span/9 plus column width/2. They include an allowance for an edge loading of 10 kN/m. (See also Ribbed slabs). In rectangular panels, the ribbed slab should usually span in the longer direction.

Advantages/disadvantages

These lightweight floors provide longer spans than one-way solid or flat slabs. They create level soffits and the provision of holes causes little or no problem in the ribbed area, but formwork costs are higher and time required is longer than for plain soffits.

Design assumptions

Supported by – Columns. Refer to column charts and data to estimate sizes.
Dimensions – Square panels, minimum of three slab spans x three beam spans. Ribs 150 mm wide @ 750 mm centres. Topping 100 mm. Moulds variable depth. Edges flush with columns. Level soffits.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services etc.) and a perimeter load of 10 kN/m is included. Self-weight used accounts for 10° slope to ribs and solid ends as described above. Self-weight from solid areas assumed spread throughout spans.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. H8 links. Main bar diameters as required and to fit within ribs. To comply with deflection criteria, service stress, σ_s, may have been reduced. Top steel provided in mid-span.

Key
Characteristic imposed load (IL)
- 2.5 kN/m^2
- 5.0 kN/m^2
- 7.5 kN/m^2
- 10.0 kN/m^2

Multiple span

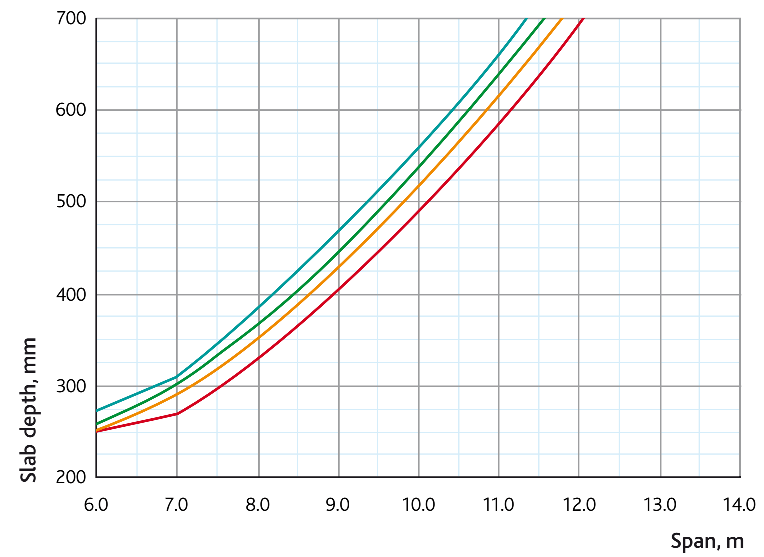


Figure 3.5
Span:depth chart for troughed slabs: multiple span

Table 3.5a
Data for troughed slabs: multiple span

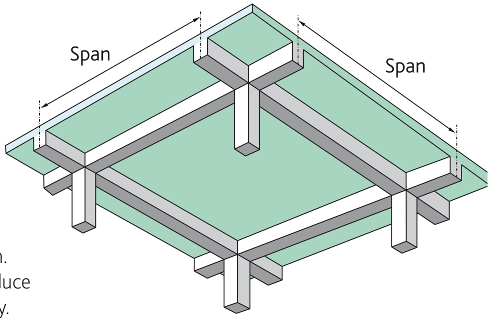
MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm									
IL = 2.5 kN/m^2	250	270	329	404	487	580	693	810	941
IL = 5.0 kN/m^2	250	290	352	426	512	616	722	840	979
IL = 7.5 kN/m^2	258	301	366	444	538	636	745	867	
IL = 10.0 kN/m^2	271	311	381	465	556	657	773	906	
Ultimate load to supporting columns, internal (edge*) per storey, kN; *excludes cladding loads									
IL = 2.5 kN/m^2	390 (280)	545 (370)	765 (500)	1050 (655)	1400 (845)	1830 (1080)	2420 (1390)	3130 (1760)	4030 (2230)
IL = 5.0 kN/m^2	525 (350)	740 (475)	1030 (635)	1380 (825)	1810 (1060)	2350 (1340)	3030 (1700)	3830 (2120)	4850 (2650)
IL = 7.5 kN/m^2	665 (425)	935 (575)	1280 (765)	1710 (995)	2220 (1270)	2840 (1600)	3620 (2010)	4540 (2480)	
IL = 10.0 kN/m^2	830 (510)	1160 (690)	1580 (920)	2100 (1200)	2710 (1520)	3450 (1910)	4390 (2400)	5510 (2980)	
Reinforcement, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	14 (80)	17 (97)	20 (94)	24 (100)	26 (94)	29 (90)	33 (85)	36 (79)	41 (75)
IL = 5.0 kN/m^2	18 (106)	22 (118)	27 (122)	29 (114)	32 (107)	34 (98)	37 (92)	41 (86)	43 (75)
IL = 7.5 kN/m^2	22 (126)	29 (150)	31 (135)	33 (123)	34 (111)	38 (105)	41 (98)	45 (90)	
IL = 10.0 kN/m^2	28 (155)	35 (177)	37 (156)	39 (142)	41 (129)	44 (117)	48 (108)	47 (87)	
Variations: overall slab depth, mm, for IL = 5.0 kN/m^2									
2 hours fire	250	290	354	429	516	621	729	849	995
4 hours fire	257	295	359	435	525	629	740	869	
Exp. XD1 + C40/50	250	283	341	410	490	584	686	792	923

Table 3.5b
Data for multiple span rectangular panels: equivalent square span, m

Ribbed slab span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Equivalent square span, m									
Beam span = 7.0 m	6.6	7.0	7.1	7.6	8.1	8.6	9.1	9.8	11.0
Beam span = 8.0 m	7.8	7.9	8.0	8.1	8.2	8.5	9.1	9.8	11.0
Beam span = 9.0 m	8.6	8.8	8.9	9.0	9.1	9.2	9.3	9.8	11.0
Beam span = 10.0 m	9.5	9.6	9.8	9.9	10.0	10.1	10.2	10.3	11.0
Beam span = 11.0 m	10.4	10.6	10.7	10.8	10.9	11.0	11.1	11.2	11.3
Beam span = 12.0 m	11.3	11.4	11.5	11.7	11.8	11.9	12.0	12.1	12.2
Beam span = 13.0 m	12.1	12.3	12.4	12.6	12.7	12.8	12.9	13.0	13.1
Beam span = 14.0 m	12.9	13.1	13.2	13.4	13.6	13.7	13.8	13.9	14.0

Note
The equivalent square span from this table should be used to derive the overall depth and load

3.1.9 Two-way solid slabs



Two-way in-situ solid slabs are utilitarian and generally used for retail developments, warehouses, stores and similar buildings. Economic for more heavily loaded spans from 4 to 12 m.

Design is usually governed by deflection. Steel content is usually increased to reduce service stress and increase span capacity.

Advantages/disadvantages

Two-way in-situ solid slabs are economical for longer spans carrying heavier loads. They provide a robust and adaptable slab with few restrictions on the position and size of holes. However, the slabs can be difficult to form when used with a grid of downstand beams. These downstand beams may result in a greater storey height, and can produce a lack of flexibility in the location of partitions and services. A regular column grid is generally required.

Design assumptions

Supported by – Beams. Refer to Section 8.3.4 then to beam charts and data to estimate sizes. End supports min.
300 mm wide.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6 and for 10.0 kN/m^2, ψ_2 = 0.8.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. Main bar diameters and distribution steel as required. To comply with deflection criteria, service stress, σ_s, may have been reduced. Top steel provided in mid-span.

Key
Characteristic imposed load (IL)
2.5 kN/m^2
5.0 kN/m^2
7.5 kN/m^2
10.0 kN/m^2
Single span
Multiple span

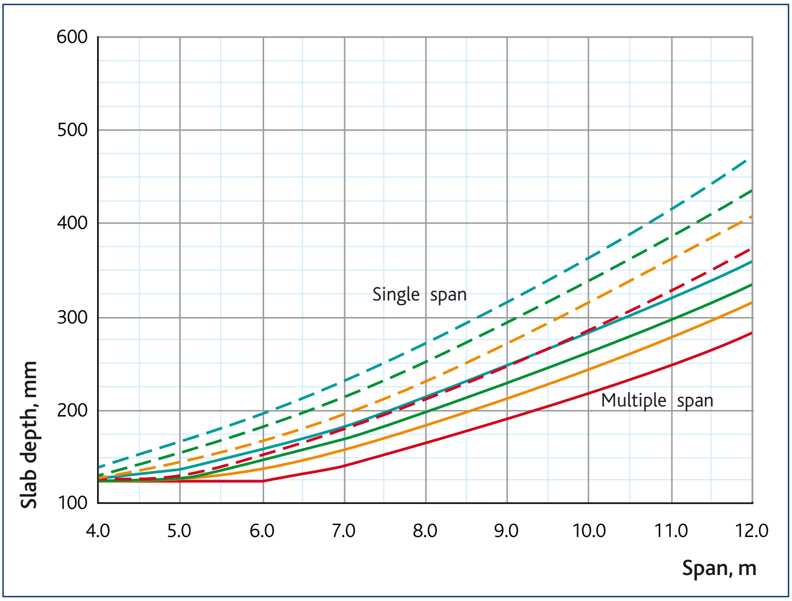


Figure 3.6
Span:depth chart for two-way solid slabs

Table 3.6a

Data for two-way solid slabs: single span

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m²	125	129	153	178	212	247	286	331	376
IL = 5.0 kN/m²	125	144	170	198	233	276	317	361	408
IL = 7.5 kN/m²	128	156	183	213	251	296	340	386	436
IL = 10.0 kN/m²	138	168	197	231	273	317	364	414	474
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m²	n/a (19)	n/a (24)	n/a (31)	n/a (39)	n/a (49)	n/a (60)	n/a (73)	n/a (88)	n/a(104)
IL = 5.0 kN/m²	n/a (27)	n/a (35)	n/a (44)	n/a (54)	n/a (67)	n/a (81)	n/a (96)	n/a(114)	n/a(133)
IL = 7.5 kN/m²	n/a (34)	n/a (45)	n/a (57)	n/a (69)	n/a (84)	n/a(101)	n/a(119)	n/a(138)	n/a(160)
IL = 10.0 kN/m²	n/a (43)	n/a (57)	n/a (71)	n/a (87)	n/a(105)	n/a(125)	n/a(147)	n/a(170)	n/a(198)
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m²	5 (37)	7 (56)	9 (60)	12 (65)	13 (62)	16 (64)	17 (60)	20 (61)	24 (63)
IL = 5.0 kN/m²	6 (47)	9 (64)	11 (66)	14 (68)	16 (69)	19 (69)	22 (69)	23 (64)	29 (71)
IL = 7.5 kN/m²	7 (58)	10 (63)	12 (64)	16 (74)	17 (69)	20 (68)	24 (69)	28 (73)	29 (67)
IL = 10.0 kN/m²	9 (67)	12 (73)	15 (79)	19 (81)	22 (80)	23 (73)	28 (76)	29 (71)	34 (73)

Table 3.6b

Data for two-way solid slabs: multiple span

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m²	125	125	125	141	166	191	220	250	282
IL = 5.0 kN/m²	125	125	137	157	183	213	243	275	313
IL = 7.5 kN/m²	125	126	148	169	199	229	262	299	335
IL = 10.0 kN/m²	125	136	158	182	213	249	284	321	360
Ultimate load to supporting beams, internal (end), kN/m Note: see Section 8.3.4									
IL = 2.5 kN/m²	38 (19)	48 (24)	57 (29)	70 (35)	86 (43)	104 (52)	125 (62)	148 (74)	173 (87)
IL = 5.0 kN/m²	53 (27)	66 (33)	82 (41)	100 (50)	121 (60)	144 (72)	170 (85)	198 (99)	230 (115)
IL = 7.5 kN/m²	68 (34)	85 (43)	106 (53)	129 (64)	155 (77)	182 (91)	213 (107)	247 (124)	283 (141)
IL = 10.0 kN/m²	85 (42)	108 (54)	134 (67)	162 (81)	194 (97)	229 (114)	266 (133)	306 (153)	350 (175)
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m²	4 (32)	5 (43)	8 (63)	9 (66)	11 (67)	13 (68)	15 (67)	16 (64)	19 (67)
IL = 5.0 kN/m²	5 (40)	7 (57)	10 (72)	12 (76)	14 (75)	15 (72)	18 (73)	20 (72)	22 (70)
IL = 7.5 kN/m²	6 (49)	9 (72)	11 (77)	14 (82)	16 (79)	18 (79)	20 (77)	22 (75)	24 (72)
IL = 10.0 kN/m²	7 (59)	11 (81)	13 (84)	16 (89)	19 (88)	21 (84)	23 (80)	26 (81)	28 (77)
Variations: overall slab depth, mm, for IL = 5.0 kN/m²									
2 hours fire	No change								
4 hours fire	125	129	148	168	195	223	254	286	320
Exp. XD1 + C40/50	125	133	154	174	200	230	261	293	330

Table 3.6c

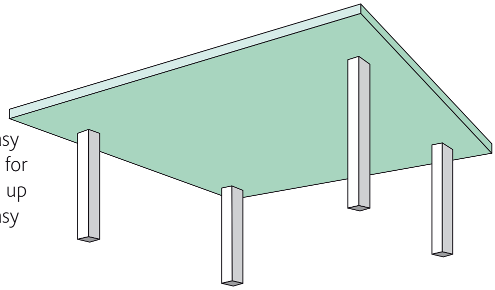
Data for multiple span rectangular panels: equivalent square span, m

LONG span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Equivalent square span, m							
Short span = 5 m	5.7	5.9	6.0				
Short span = 6 m	6.7	6.8	7.0	7.1	7.2		
Short span = 7 m	7.4	7.7	7.9	8.1	8.1	8.2	8.3
Short span = 8 m	8.0	8.4	8.7	8.9	9.0	9.2	9.3
Short span = 9 m		9.0	9.4	9.7	9.9	10.1	10.2
Short span = 10 m			10.0	10.2	10.4	10.6	10.7

Note

The equivalent square span from this table should be used to derive the overall depth and load.

3.1.10 Flat slabs



Flat slabs are very popular for office buildings, hospitals, hotels and blocks of apartments as they are quick and easy to construct. They are very economical for spans of 5 to 9 m and commonly used up to 12 m span. Their flat soffits allow easy service distribution.

Advantages/disadvantages

These slabs are easy and fast to construct, and the architectural finish can be applied directly to the underside of the slab. The absence of beams allows lower storey heights and flexibility of both partition location and horizontal service distribution. It is easy to seal partitions for airtightness, fire protection and acoustic isolation. However, the provision of large holes can prove difficult, especially near perimeter columns. Punching shear should be checked and provided for. Deflections, especially of edges supporting cladding, may cause concern. A higher concrete grade, larger columns and/or lack of holes can be advantageous – see *Variations*.

Design assumptions

Supported by – Columns. Charts and data assume slabs are supported by columns whose sizes at least equal those given in the data. Refer to column charts and data to estimate sizes.
Dimensions – Square panels, minimum of three spans x three bays. Outside edges flush with columns.
Loads – A superimposed dead load (SDL) of 150 kN/m^2 (for finises, services etc.) and a perimeter load of 10 kN/m (cladding) are included.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. Main to comply with deflection criteria, service stress σ_s, may have been reduced. Top steel provided in mid-span.
Holes – One 150 mm square hole assumed to adjoin each column. Larger holes may invalidate the data below.

Key
Characteristic imposed load (IL)
2.5 kN/m^2
5.0 kN/m^2
7.5 kN/m^2
10.0 kN/m^2

Multiple span

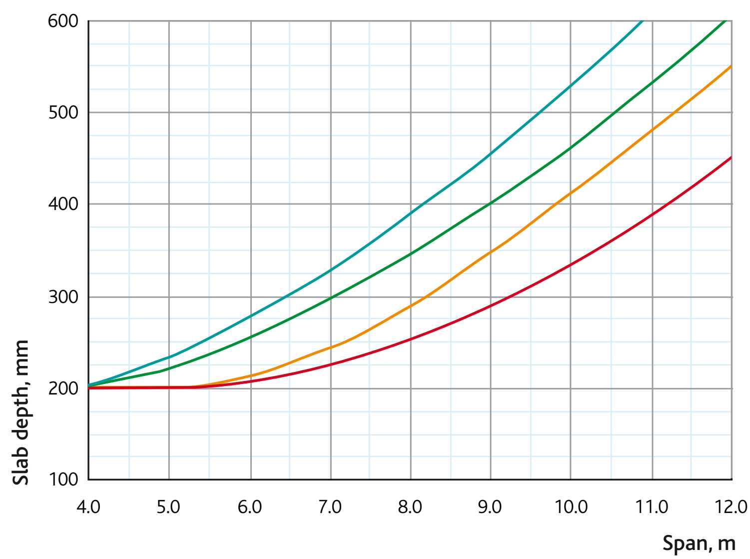


Figure 3.7
Span:depth chart for flat slabs: multiple span

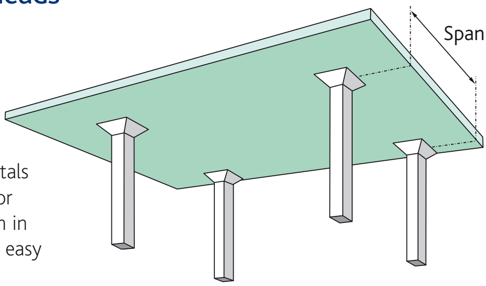
Table 3.7
Data for flat slabs:multiple span

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m²	200	200	206	227	250	286	343	386	450
IL = 5.0 kN/m²	200	200	215	246	284	347	427	479	565
IL = 7.5 kN/m²	200	220	253	305	342	404	460	549	602
IL = 10.0 kN/m²	200	236	278	327	399	452	533	615	696
Ultimate load to supporting columns, internal (edge*) per storey, kN; * excludes cladding loads									
IL = 2.5 kN/m²	190 (95)	297 (148)	434 (217)	623 (311)	859 (430)	1179 (589)	1633 (817)	2139 (1069)	2857 (1428)
IL = 5.0 kN/m²	250 (125)	390 (195)	579 (290)	836 (418)	1167 (584)	1637 (818)	2270 (1135)	2944 (1472)	3890 (1945)
IL = 7.5 kN/m²	310 (155)	500 (250)	757 (378)	1110 (555)	1523 (762)	2085 (1042)	2748 (1374)	3662 (1831)	4596 (2298)
IL = 10.0 kN/m²	380 (190)	625 (312)	951 (475)	1375 (688)	1951 (976)	2615 (1307)	3501 (1751)	4572 (2286)	5834 (2917)
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m²	10 (50)	12 (59)	14 (70)	18 (79)	22 (88)	25 (88)	28 (81)	32 (82)	36 (80)
IL = 5.0 kN/m²	11 (54)	15 (73)	19 (90)	22 (91)	26 (92)	29 (83)	33 (76)	36 (75)	40 (71)
IL = 7.5 kN/m²	13 (63)	16 (73)	20 (80)	24 (79)	28 (81)	32 (78)	36 (79)	40 (73)	46 (76)
IL = 10.0 kN/m²	15 (74)	19 (82)	23 (85)	27 (81)	30 (76)	36 (80)	40 (75)	46 (74)	51 (74)
Column sizes assumed, (sq.) mm									
IL = 2.5 kN/m²	225	225	225	350	400	450	500	550	600
IL = 5.0 kN/m²	225	225	225	350	400	450	500	550	600
IL = 7.5 kN/m²	225	225	225	350	400	450	500	550	600
IL = 10.0 kN/m²	225	225	225	350	400	450	500	550	600
Links, maximum number of perimeters (percentage by weight of reinforcement), no. (%)									
IL = 2.5 kN/m²	3 (0.7%)	5 (0.4%)	6 (0.2%)	8 (0.2%)	8 (0.3%)	7 (0.3%)	8 (0.5%)	7 (0.8%)	8 (0.8%)
IL = 5.0 kN/m²	3 (0.6%)	8 (0.3%)	8 (0.2%)	8 (0.3%)	7 (0.4%)	8 (0.4%)	8 (0.3%)	8 (0.7%)	8 (0.9%)
IL = 7.5 kN/m²	5 (0.5%)	7 (0.2%)	7 (0.3%)	8 (0.3%)	8 (0.5%)	8 (0.7%)	7 (0.6%)	8 (0.8%)	7 (1.1%)
IL = 10.0 kN/m²	8 (0.5%)	8 (0.3%)	8 (0.3%)	7 (0.6%)	8 (0.7%)	7 (0.8%)	8 (0.9%)	8 (1.3%)	8 (1.2%)
Variations: overall slab depths for IL = 5.0 kN/m²									
Columns below only	Minimal effect (moment transfer restricted to 0.17 $b_e d^2 f_{ck}$)								
Rectangular panels	Use values for longer span								
2 hours fire	No change								
4 hours fire	200	200	220	251	284	347	427	479	565
Grade C35/45	200	200	209	230	265	308	379	430	507
Column size = span/15	200	200	205	231	268	301	362	407	476
XC3/4 + C40/50	200	200	210	236	272	308	384	430	506
Edge cols ⅔ area of internal cols	200	200	272	319	373	436	479	536	624
300 sq. holes at cols	200	220	240	267	304	362	427	479	565
No holes at edges	200	200	214	240	269	321	418	450	565
20 kN/m cladding	200	233	266	314	356	402	466	515	613

3.1.11 Flat slabs with column heads

Increasing the size of column heads under the slab increases its shear-carrying capacity at columns.

These slabs are popular for office buildings, retail developments, hospitals and hotels as they are economical for heavily loaded spans from 8 to 13 m in square panels. Their flat soffits allow easy service distribution.



Advantages/disadvantages

These slabs are easy and fast to construct, although the column heads can disrupt cycle time unless they can be poured with the columns. The absence of beams allows lower storey heights and flexibility of partition location and horizontal service distribution. However, the provision of large holes can prove difficult, especially near columns, and punching shear should be checked and provided for. Deflections, especially of edges supporting cladding, may cause concern.

Design assumptions

Supported by – Columns with column heads. Charts and data assume slabs are supported by columns with heads whose sizes at least equal those given in the data.
Dimensions – Square panels, minimum of three spans x three bays. Internal column head sizes as indicated in the data. Outside edges of slabs flush with columns, column heads reduced accordingly.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services etc.) and a perimeter load of 10 kN/m (cladding) are included.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. Main to comply with deflection criteria, service stress σ_s, may have been reduced. Top steel provided in mid-span.
Holes – One 150 mm square hole assumed to adjoin each column (within column head). Larger holes may invalidate the data below.

Key
Characteristic imposed load (IL)

- 2.5 kN/m^2
- 5.0 kN/m^2
- 7.5 kN/m^2
- 10.0 kN/m^2

Multiple span

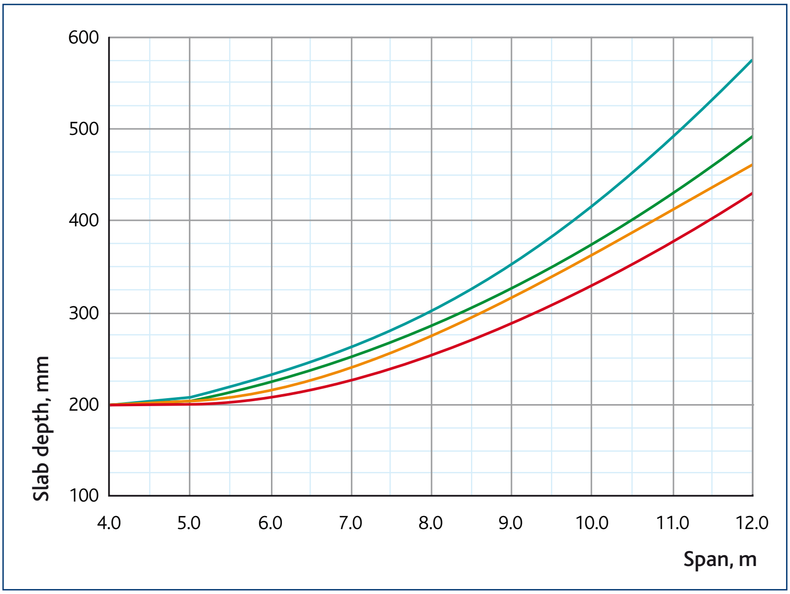
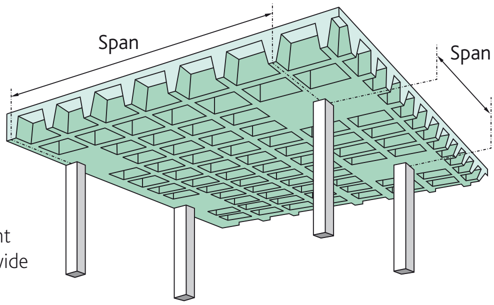


Figure 3.8
Span:depth chart for flat slabs with column heads: multiple span

Table 3.8
Data for flat slabs with column heads: multiple span

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm									
IL = 2.5 kN/m^2	200	200	205	224	251	284	330	380	429
IL = 5.0 kN/m^2	200	200	211	240	270	313	359	410	462
IL = 7.5 kN/m^2	200	202	220	250	282	322	378	437	491
IL = 10.0 kN/m^2	200	205	229	267	299	342	410	509	558
Ultimate load to supporting columns, internal (edge*) per storey, kN; *excludes cladding loads									
IL = 2.5 kN/m^2	190 (95)	297 (148)	433 (216)	618 (309)	861 (431)	1174 (587)	1593 (796)	2116 (1058)	2755 (1377)
IL = 5.0 kN/m^2	250 (125)	390 (195)	575 (287)	826 (413)	1139 (570)	1551 (775)	2058 (1029)	2683 (1341)	3427 (1713)
IL = 7.5 kN/m^2	310 (155)	484 (242)	720 (360)	1025 (513)	1403 (702)	1877 (939)	2492 (1246)	3239 (1619)	4097 (2049)
IL = 10.0 kN/m^2	380 (190)	594 (297)	891 (446)	1276 (638)	1735 (868)	2314 (1157)	3086 (1543)	4139 (2069)	5163 (2582)
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m^2	10 (48)	12 (59)	14 (70)	18 (79)	21 (82)	24 (85)	27 (82)	31 (81)	36 (83)
IL = 5.0 kN/m^2	11 (56)	14 (70)	18 (86)	21 (89)	25 (94)	31 (101)	36 (100)	39 (95)	44 (96)
IL = 7.5 kN/m^2	12 (62)	17 (84)	22 (98)	26 (104)	32 (113)	37 (114)	42 (110)	44 (101)	51 (103)
IL = 10.0 kN/m^2	14 (72)	21 (106)	28 (121)	33 (124)	39 (131)	46 (134)	50 (122)	48 (95)	57 (102)
Column head sizes assumed, (sq) mm									
Internal	900	950	1000	1050	1200	1350	1500	1650	1800
Perimeter	Heads to match those of internal columns, column flush with edge								
Links, maximum number of perimeters (percentage by weight of reinforcement), no. (%)									
IL = 2.5 kN/m^2	0	0	3 (0.8%)	4 (0.7%)	5 (0.7%)	5 (0.6%)	6 (0.6%)	6 (1.0%)	6 (0.9%)
IL = 5.0 kN/m^2	0	3 (1.8%)	4 (0.6%)	5 (0.8%)	5 (0.8%)	6 (0.8%)	6 (1.1%)	6 (1.0%)	6 (1.2%)
IL = 7.5 kN/m^2	0	4 (1.4%)	6 (0.7%)	6 (0.9%)	7 (1.0%)	7 (1.1%)	7 (1.7%)	7 (1.7%)	8 (1.8%)
IL = 10.0 kN/m^2	0	6 (1.7%)	8 (0.7%)	8 (1.0%)	8 (1.3%)	7 (1.8%)	8 (2.0%)	8 (2.5%)	8 (3.2%)
Variations: overall slab depths for IL = 5.0 kN/m^2									
Columns below only	Minimal effect (moment transfer restricted to 0.17 $b_e d^2 f_{ck}$)								
Rectangular panels	Use values for longer span.								
2 hours fire	No change								
4 hours fire	200	200	221	250	282	323	371	425	476
Grade C35/45	200	200	205	232	258	301	344	391	441
XC3/4 + C40/50	200	200	209	235	262	302	343	385	430
20 kN/m cladding	200	200	220	257	285	324	370	419	470

3.1.12 Waffle slabs



Introducing voids to the soffit of a flat slab reduces dead weight. The profile may be expressed architecturally. The depth of these slabs is governed by deflection, punching shear around columns and shear in ribs.

The charts assume a solid area adjacent to supporting columns up to span/2 wide and long.

Advantages/disadvantages

Designed as flat slabs, these waffle slabs are light and benefit from flexibility of partition location and horizontal service distribution. However, the formwork is more costly and the slightly deeper profile results in greater overall floor depth than for flat slabs. The reinforcement is difficult to prefabricate and so may be slow to fix.

Design assumptions

Supported by – Columns above and below. Refer to column charts and data to estimate sizes.
Dimensions – Square panels, minimum of three spans x three bays. If panels are not square, obtain values from longer span. Ribs 180 mm wide @ 900 mm centres. Topping 100 mm. Bespoke moulds of variable depth. Solid area ≤ span/2 in each direction.
Loads – A superimposed dead load (SDL) of 150 kN/m^2 (for finishes, services etc.) and a perimeter load of 10 kN/m (cladding) are included. Self-weight used accounts for 10:1 slope to ribs and solid areas as described above.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. Main to comply with deflection criteria, service stress σ_s, may have been reduced. Top steel provided in mid-span.

Key
Characteristic imposed load (IL)
- 2.5 kN/m^2
- 5.0 kN/m^2
- 7.5 kN/m^2
- 10.0 kN/m^2

Multiple span

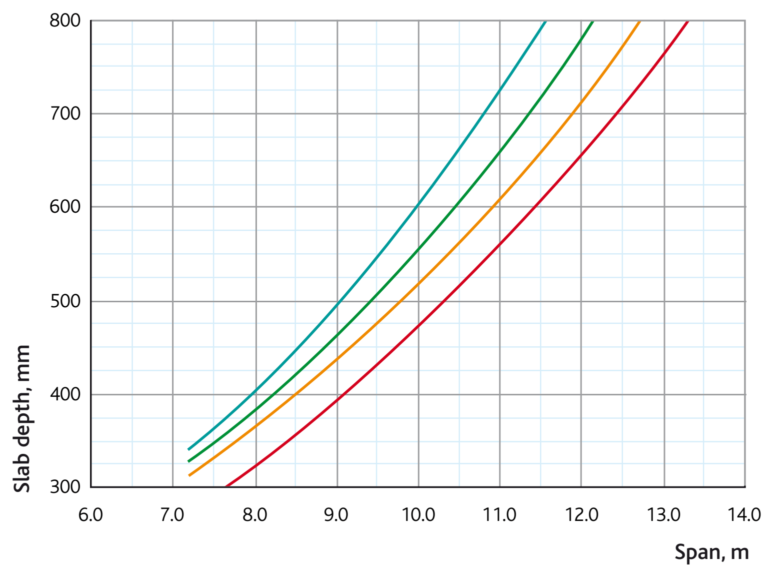


Figure 3.9
Span:depth chart for waffle slabs: multiple span

Table 3.9
Data for waffle slabs: multiple span

MULTIPLE span, m	7.2	8.1	9.0	9.9	10.8	11.7	12.6	13.5	14.4
Overall depth, mm									
IL = 2.5 kN/m^2	267	326	403	465	541	610	716	812	949
IL = 5.0 kN/m^2	310	371	440	508	588	671	786	889	
IL = 7.5 kN/m^2	325	389	461	539	644	741	862	972	
IL = 10.0 kN/m^2	339	408	497	589	704	811	942		
Ultimate load to supporting columns, internal (end*) kN; *excludes cladding loads									
IL = 2.5 kN/m^2	600 (300)	800 (400)	1100 (550)	1400 (700)	1900 (950)	2300 (1150)	3100 (1550)	3900 (1950)	5200 (2700)
IL = 5.0 kN/m^2	800 (400)	1100 (550)	1500 (750)	1900 (950)	2400 (1200)	3000 (1500)	3900 (1950)	4800 (2400)	
IL = 7.5 kN/m^2	1000 (500)	1400 (700)	1800 (900)	2300 (1150)	3000 (1500)	3700 (1850)	4800 (2400)	5800 (2900)	
IL = 10.0 kN/m^2	1300 (650)	1700 (950)	2200 (1100)	2800 (1400)	3700 (1850)	4600 (2200)	5900 (2950)		
Reinforcement, kg/m² (kg/m³)									
IL = 2.5 kN/m^2	18 (101)	20 (96)	22 (90)	30 (111)	31 (101)	32 (94)	36 (87)	37 (79)	39 (72)
IL = 5.0 kN/m^2	29 (146)	30 (131)	30 (116)	33 (112)	33 (100)	36 (94)	37 (83)	39 (75)	
IL = 7.5 kN/m^2	31 (152)	32 (136)	34 (124)	34 (111)	35 (96)	38 (92)	40 (79)	41 (73)	
IL = 10.0 kN/m^2	33 (153)	35 (141)	36 (123)	36 (108)	38 (95)	40 (86)	43 (77)		
Variations: overall slab depth, mm, for IL = 5.0 kN/m^2									
Columns below only	Minimal effect (moment transfer restricted to 0.17 $b_e d^2 f_{ck}$)								
Rectangular panels	Use values for longer span.								
C35/40, Δc_{dev} = 5 mm, b_w = 170 mm	295	352	419	484	560	641	751	849	978
2 hours fire, h_f = 120 mm, b_w = 200 mm	303	362	430	497	576	663	777	880	
Exp. XC3/4, C40/50	291	345	409	470	546	626	730	823	945
4 hours fire, h_f = 175 mm, b_w = 450 mm, 1200 mm centres	287	359	440	530	626	732	866	1029	

3.2 In-situ beams

3.2.1 Using in-situ beams

Essentially, beams provide vertical support. In building structures they generally transfer loads from slabs to columns and walls. They are designed to resist resulting ultimate bending moments and shear forces and then checked against serviceability requirements. In-situ beams offer strength, robustness and, above all, versatility, for instance in accommodating cladding support details.

In overall terms, wide flat-beams are less costly to construct than narrow deep beams: the deeper and narrower, the more costly they are to construct. The following comments also apply.

- If beams and columns are of the same width, the common planes can lead to efficient working as formwork can proceed along a continuous line. However, used internally, these relatively deep beams result in additional perimeter cladding. They also tend to disrupt progress and service runs.
- Downstand edge beams may limit the use of flying form systems on the slab. Upstand perimeter beams (designed as rectangular beams) can reduce overall cost. Parapet wall beams are less disruptive and less costly to form than deep downstand beams.
- Upstand beams and shallow downstand band beams can be easier to construct and have less impact on horizontal services distribution and floor-to-floor heights than relatively deep downstand beams (see Figures 3.B and 3.C).

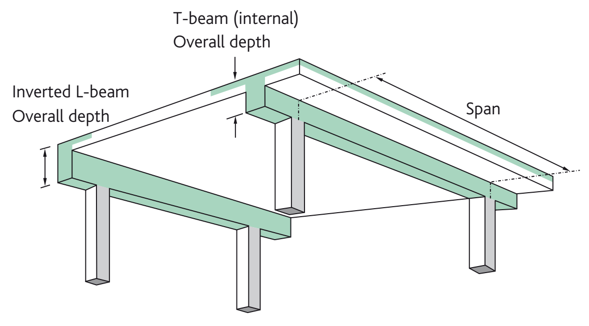


Figure 3.B
In-situ concrete beams: T- and inverted L-beams

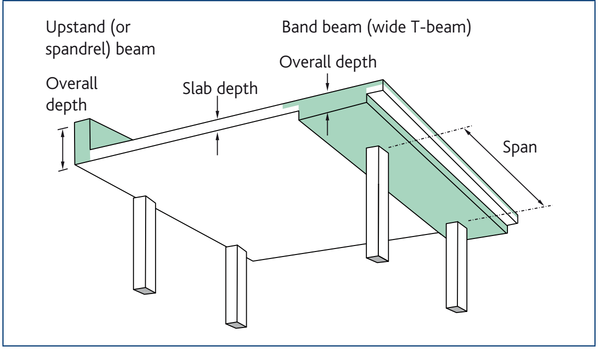


Figure 3.C
In-situ upstand beams and band beams

3.2.2 The charts and data

The intersections of beams and columns require special consideration of reinforcement details. Sufficient width is required to provide room for both beam and column reinforcement; end supports need to be long enough to allow bends in bottom reinforcement to start within the support yet maintain cover for links and/or lacers.

The charts for in-situ reinforced beams cover a range of web widths and ultimate applied uniformly distributed loads (uaudl). They are divided into:

- Rectangular beams – isolated or upstand beams, beams with no flange, beams not homogeneous with supported slabs.
- Inverted L-beams – perimeter beams with top flange one side of the web.
- T-beams – internal beams with top flange both sides of the web

Table 3.A lists web widths for which information is provided in the charts and data.

Table 3.A
Range of in-situ beams covered in charts and data

Span type	Rectangular beams	Inverted L-beams	T-beams
Single span	300 600	300 600 900 1200	300 600 1200 2400
Multiple span	300 600	225 300 450 600 900 1200	300 450 600 900 1200 1800 2400

The user must determine which form of beam is appropriate and, therefore, select which figure and table to use. From the appropriate chart(s) and data, use the maximum span and appropriate ultimate applied uniformly distributed loads (uaudl), determined in accordance with Section 8.3, to interpolate between values given in the charts and data.

The charts and data for multiple-span beams are based on a minimum of three spans, so the user is expected to make adjustments for two-span configurations. In particular, the user is expected to round up both the derived depth and loads to supports in line with modular sizing and with his or her confidence in the criteria used. A nominal depth limit of 800 mm is used in the charts and data.

Users should note that the data for slabs give ultimate load to supporting beams. These loads assume the use of elastic reaction factors of 1.0 to internal beams and 0.5 to end supports. For internal beams acting as penultimate supports, a suitable elastic reaction factor should be applied in accordance with Section 8.3.2.

3.2.3 Design assumptions

Dimensions

The default dimensions are given in Table 3.B. Flange widths are in accordance with Eurocode 2[2], Cl. 5.3.2.1.

Table 3.B
Assumed dimensions for different types of beams

Beam type	Rectangular	L-beam	T-beam
Flange width, single span	b_w	$b_w + 0.10L$	$b_w + 0.20L$
Flange width, continuous spans	b_w	$b_w + 0.07L$	$b_w + 0.14L$
Top flange thickness	100	100	100

Design

The assumptions used to derive the charts and data are detailed in Section 7.1.4. Essentially the charts and data are based on:

- Moments and shears from three-span sub-frame analysis to Eurocode 2, assuming continuity with nominal 250 mm sq. columns above and below.
- Variable actions, $Q_k \leq$ permanent actions, G_k.
- Substantially uniformly distributed loads.
- Quasi-permanent value of variable actions = $0.6Q_k$ (i.e. $\psi_2 = 0.6$, applicable to all but storage areas where an allowance for $\psi_2 = 0.8$ should be made).
- The more onerous of Expressions (6.10a) or (6.10b).
- Minimum span ≥ 0.85 x maximum span.

End spans are considered critical. Unless subjected to more than 15% redistribution of support moments, two-span slab elements will be subject to greater support moments and shears than those assumed. Nonetheless, the sizes given in the charts and data can be used cautiously for two-span conditions unless support moment or shear is considered critical. In such cases two-span beams should be justified by analysis and design.

In the charts, sizes of beams are based on a single layer of reinforcement where feasible. In any case, no more than two layers of reinforcement have been considered or used.

Load factors to BS EN 1990[9], Expressions (6.10a) or (6.10b) have been employed throughout. If the more basic Expression (6.10) is used in design, greater beam depths may be required.

In order to satisfy deflection criteria, the steel service stress, σ_s, has in many cases been reduced by increasing $A_{s,prov}$ (area of steel provided) but keeping $A_{s,prov}/A_{s,req}$ within code limitations.

Fire and durability

Fire resistance 1 hour (R60); exposure class XC1; cover to all max[15; ϕ] + Δc_{dev} where Δc_{dev} = 10 mm.

Loads

Beam self-weight (in addition to an assumed 200 mm depth of solid slab in T- and L-beams) has been allowed for and is included in ultimate loads to supports.

Ultimate loads to supports assume reaction factors of 1.0 internally and 0.5 to ends. The user should make allowance for elastic effects, particularly at penultimate supports (see Section 8.2.2).

Concrete

This is taken as C30/37; 25 kN/m^3; 20 mm aggregate.

Reinforcement

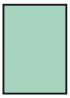
Main bars: f_{yk} = 500 MPa. Maximum H32s top and bottom, links: minimum H8. Minimum 50 mm between top bars. No additional top cover has been allowed for bars passing at right-angles.

Reinforcement quantities are for the beams only. For T- and L-beams, density of reinforcement relates to overall depth x web width. See also Section 2.2.4.

Variations

Variations from the above assumptions and assumptions for the individual types of beam are described in the relevant data. Other assumptions made are described and discussed in Section 7, *Derivation of charts and data*.

3.2.4 Rectangular beams, single span, 300 mm wide



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 **factor** – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Single span

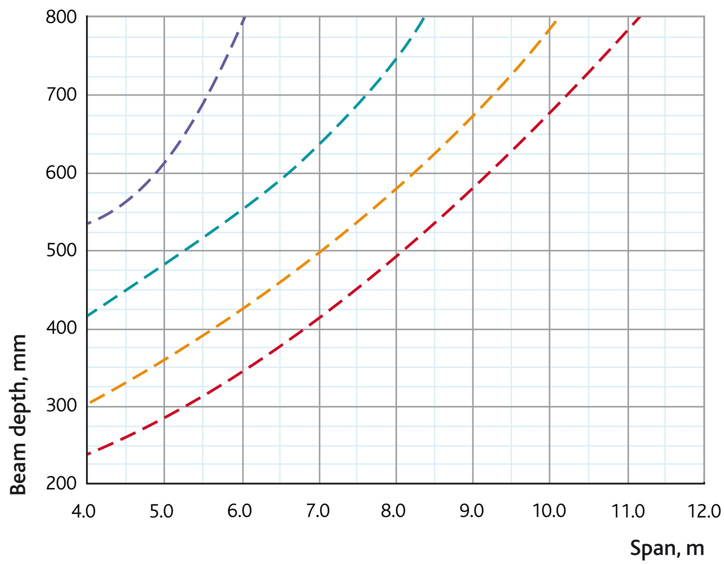
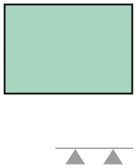


Figure 3.10
Span:depth chart for single-span rectangular beams 300 mm wide

Table 3.10
Data for single-span rectangular beams 300 mm wide

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	237	285	347	405	488	578	674	786	896
uaudl = 50 kN/m	289	379	433	491	545	670	797	933	
uaudl = 100 kN/m	410	483	558	630	735	924			
uaudl = 200 kN/m	535	609	799	1064					
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	54	69	85	101	118	137	157	178	200
uaudl = 50 kN/m	105	134	162	191	220	253	287	323	
uaudl = 100 kN/m	208	261	316	371	428	489			
uaudl = 200 kN/m	410	514	622	735					
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	18 (248)	17 (199)	16 (158)	24 (194)	23 (159)	23 (134)	24 (118)	25 (105)	31 (114)
uaudl = 50 kN/m	18 (204)	18 (159)	27 (207)	27 (180)	34 (209)	32 (159)	32 (133)	34 (123)	
uaudl = 100 kN/m	21 (171)	28 (196)	29 (172)	41 (218)	38 (171)	38 (136)			
uaudl = 200 kN/m	32 (201)	50 (272)	39 (164)	40 (126)					
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	297	380	442	502	564	671	798	935	
4 hours fire	346	416	478	565	689	843			
Exp. XD1 + C40/50	274	331	388	447	544	654	772	904	

3.2.5 Rectangular beams, single span, 600 mm wide



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)

- 25 kN/m
- 50 kN/m
- 100 kN/m
- 200 kN/m

Single span

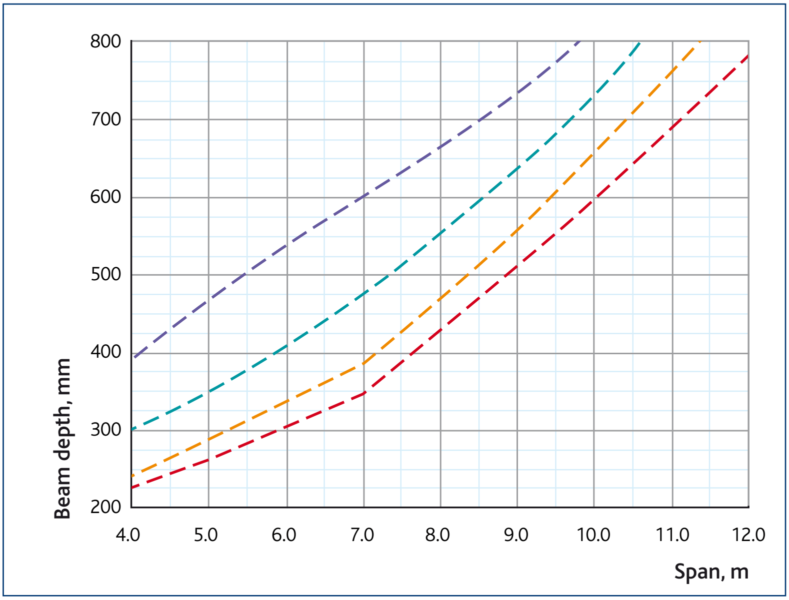
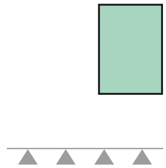


Figure 3.11
Span:depth chart for single-span rectangular beams, 600 mm wide

Table 3.11
Data for single-span rectangular beams, 600 mm wide

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	225	258	301	345	429	508	593	684	781
uaudl = 50 kN/m	237	285	333	383	466	556	652	754	865
uaudl = 100 kN/m	289	351	420	482	521	612	738	857	
uaudl = 200 kN/m	392	466	525	594	664	751	819		
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	58	75	92	110	132	155	181	208	238
uaudl = 50 kN/m	109	138	169	200	235	272	311	353	397
uaudl = 100 kN/m	211	266	324	382	439	502	569	638	
uaudl = 200 kN/m	415	522	630	739	850	963	1077		
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	19 (142)	24 (153)	26 (142)	28 (134)	30 (115)	33 (107)	34 (95)	37 (91)	45 (96)
uaudl = 50 kN/m	31 (218)	33 (193)	37 (187)	44 (191)	48 (173)	49 (147)	59 (150)	59 (131)	60 (115)
uaudl = 100 kN/m	34 (197)	41 (197)	50 (197)	61 (211)	74 (237)	80 (217)	80 (182)	80 (156)	
uaudl = 200 kN/m	45 (191)	63 (224)	77 (243)	94 (264)	105 (264)	111 (247)	141 (287)		
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	239	287	335	OK	OK	OK	OK	OK	OK
4 hours fire	264	312	361	401	484	573	669	772	886
Exp. XD1 + C40/50	235	284	327	375	453	536	625	719	827

3.2.6 Rectangular beams, multiple span, 300 mm wide



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 **factor** – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

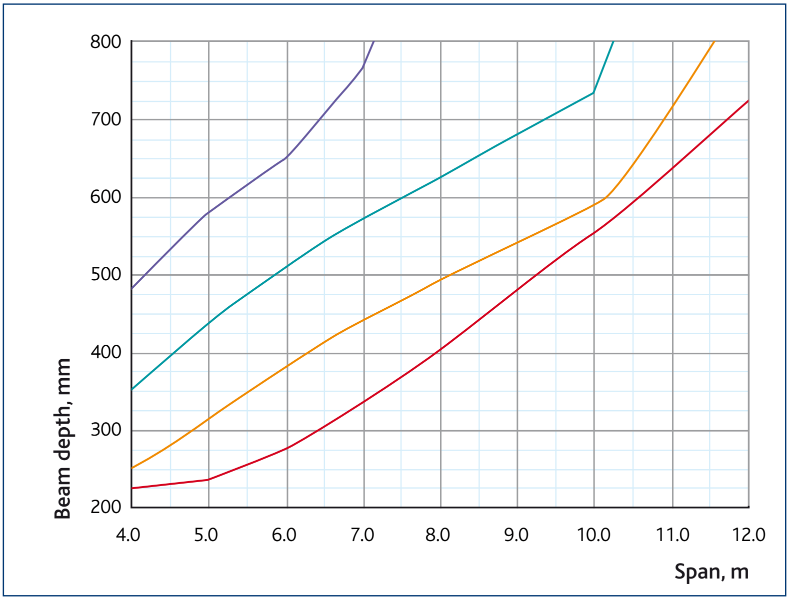
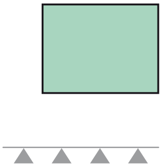


Figure 3.12
Span:depth chart for multiple-span rectangular beams, 300 mm wide

Table 3.12
Data for multiple-span rectangular beams, 300 mm wide

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	225	237	275	335	402	481	553	635	721
uaudl = 50 kN/m	254	306	388	441	495	548	594	724	837
uaudl = 100 kN/m	352	439	510	574	624	679	733		
uaudl = 200 kN/m	482	582	649	763	1026				
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	108 (54)	136 (68)	165 (83)	197 (98)	230 (115)	266 (133)	302 (151)	340 (170)	381 (191)
uaudl = 50 kN/m	210 (105)	264 (132)	322 (161)	379 (189)	437 (219)	496 (248)	556 (278)	625 (312)	694 (347)
uaudl = 100 kN/m	413 (207)	521 (260)	629 (314)	738 (369)	847 (423)	957 (479)	1069 (534)		
uaudl = 200 kN/m	818 (409)	1027 (514)	1237 (618)	1450 (725)	1677 (838)				
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	11 (161)	17 (242)	18 (213)	19 (184)	17 (144)	22 (155)	23 (137)	23 (121)	23 (108)
uaudl = 50 kN/m	18 (231)	20 (219)	22 (185)	22 (169)	28 (186)	34 (206)	38 (211)	35 (162)	35 (140)
uaudl = 100 kN/m	23 (215)	23 (175)	33 (217)	37 (213)	44 (236)	46 (225)	55 (248)		
uaudl = 200 kN/m	35 (239)	42 (242)	49 (253)	58 (252)	49 (161)				
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	283	331	406	459	512	577	640	744	857
Exp. XD1 + C40/50	250	296	347	393	465	516	606	705	812

3.2.7 Rectangular beams, multiple span, 600 mm wide



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 **factor** – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

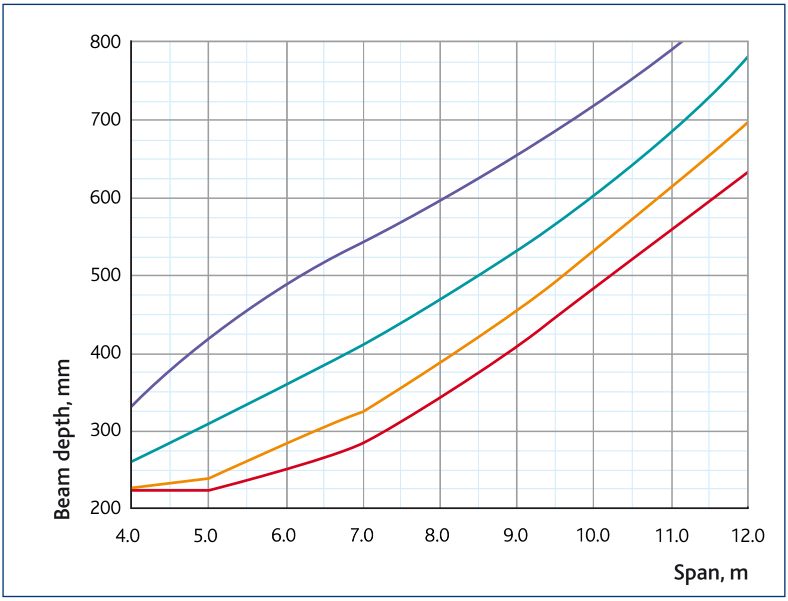
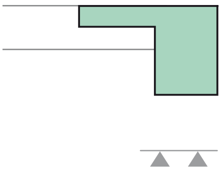


Figure 3.13
Span:depth chart for multiple-span rectangular beams, 600 mm wide

Table 3.13
Data for multiple-span rectangular beams, 600 mm wide

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	225	225	250	283	343	406	485	557	634
uaudl = 50 kN/m	225	237	283	324	387	454	531	612	699
uaudl = 100 kN/m	255	307	366	419	473	517	583	689	793
uaudl = 200 kN/m	337	417	490	543	602	662	723	782	836
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	117 (58)	146 (73)	178 (89)	212 (106)	251 (126)	294 (147)	341 (170)	390 (195)	443 (221)
uaudl = 50 kN/m	217 (108)	272 (136)	332 (166)	393 (196)	458 (229)	527 (263)	600 (300)	676 (338)	757 (379)
uaudl = 100 kN/m	419 (210)	529 (264)	641 (321)	755 (377)	871 (435)	987 (494)	1109 (555)	1242 (621)	1378 (689)
uaudl = 200 kN/m	825 (413)	1039 (520)	1255 (628)	1471 (736)	1690 (845)	1912 (956)	2136 (1068)	2361 (1181)	2588 (1294)
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	14 (107)	19 (141)	22 (150)	24 (142)	25 (123)	25 (104)	26 (90)	29 (86)	30 (78)
uaudl = 50 kN/m	20 (150)	30 (212)	31 (183)	38 (204)	38 (163)	41 (150)	46 (143)	47 (128)	49 (117)
uaudl = 100 kN/m	32 (206)	38 (205)	42 (192)	47 (186)	57 (200)	63 (204)	72 (205)	74 (180)	77 (162)
uaudl = 200 kN/m	43 (211)	52 (206)	62 (212)	71 (218)	79 (219)	93 (233)	106 (243)	120 (256)	142 (283)
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	225	249	288	326	392	461	533	615	701
Exp. XD1 + C40/50	225	238	275	311	374	440	512	588	667

3.2.8 Inverted L-beams, single span, 300 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Single span

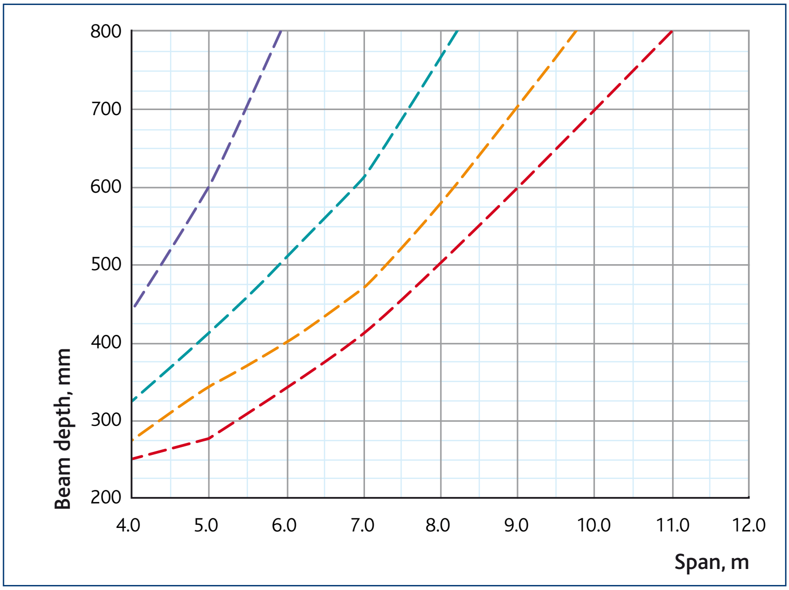
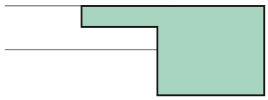


Figure 3.14
Span:depth chart for single-span inverted L-beams, 300 mm web

Table 3.14
Data for single-span inverted L-beams 300 mm wide

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	250	274	339	410	499	595	693	797	903
uaudl = 50 kN/m	275	341	399	470	576	699	831		
uaudl = 100 kN/m	322	409	506	612	765	932			
uaudl = 200 kN/m	439	596	809						
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	53	67	82	98	115	133	153	173	195
uaudl = 50 kN/m	103	131	158	187	218	250	284		
uaudl = 100 kN/m	204	257	311	367	425	485			
uaudl = 200 kN/m	406	512	620						
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	13 (174)	17 (201)	18 (178)	23 (190)	23 (155)	23 (131)	24 (114)	29 (120)	30 (112)
uaudl = 50 kN/m	19 (228)	24 (239)	32 (266)	32 (224)	31 (181)	31 (150)	32 (127)		
uaudl = 100 kN/m	33 (345)	34 (278)	36 (237)	37 (200)	36 (158)	37 (133)			
uaudl = 200 kN/m	41 (314)	39 (221)	39 (162)						
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	357	424	516	621	792	1010			
Exp. XD1 + C40/50	277	330	391	459	564	682	807		
$\psi_2 = 0.8$	287	343	412	485	600	729	867		

3.2.9 Inverted L-beams, single span, 600 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
- 25 kN/m
- 50 kN/m
- 100 kN/m
- 200 kN/m

Single span

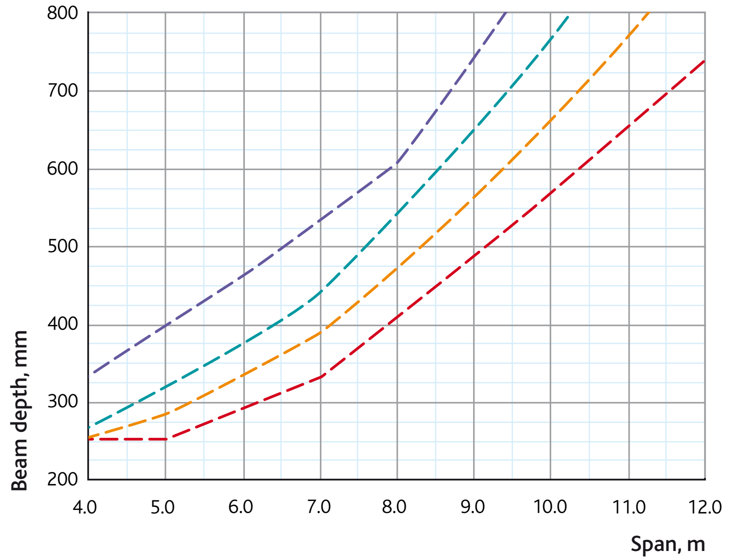
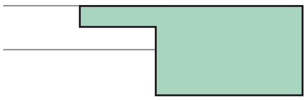


Figure 3.15
Span:depth chart for single-span inverted L-beams, 600 mm web

Table 3.15
Data for single-span inverted L-beams, 600 mm web

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	250	250	286	329	405	489	567	650	738
uaudl = 50 kN/m	250	284	332	385	469	561	662	771	888
uaudl = 100 kN/m	265	318	373	437	536	648	765	895	
uaudl = 200 kN/m	332	401	460	535	606	739	884		
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	n/a (56)	n/a (70)	n/a (85)	n/a (103)	n/a (123)	n/a (145)	n/a (169)	n/a (194)	n/a (222)
uaudl = 50 kN/m	n/a (106)	n/a (134)	n/a (163)	n/a (194)	n/a (228)	n/a (264)	n/a (303)	n/a (344)	n/a (389)
uaudl = 100 kN/m	n/a (206)	n/a (260)	n/a (315)	n/a (372)	n/a (433)	n/a (496)	n/a (562)	n/a (632)	
uaudl = 200 kN/m	n/a (409)	n/a (514)	n/a (620)	n/a (729)	n/a (838)	n/a (954)	n/a(1074)		
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	19 (127)	26 (176)	29 (167)	31 (158)	30 (124)	30 (102)	36 (105)	41 (106)	54 (122)
uaudl = 50 kN/m	24 (159)	33 (193)	37 (187)	41 (179)	41 (146)	48 (143)	50 (125)	59 (127)	66 (123)
uaudl = 100 kN/m	45 (282)	50 (263)	57 (256)	64 (245)	66 (204)	71 (183)	74 (162)	79 (148)	
uaudl = 200 kN/m	53 (268)	63 (263)	88 (318)	92 (288)	117(322)	113 (254)	113 (213)		
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	252	299	348	397	482	574	674	784	901
Exp. XD1 + C40/50	250	276	322	368	446	531	624	724	830
$\psi_2 = 0.8$	250	289	338	388	473	566	668	779	900

3.2.10 Inverted L-beams, single span, 900 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 **factor** – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Single span

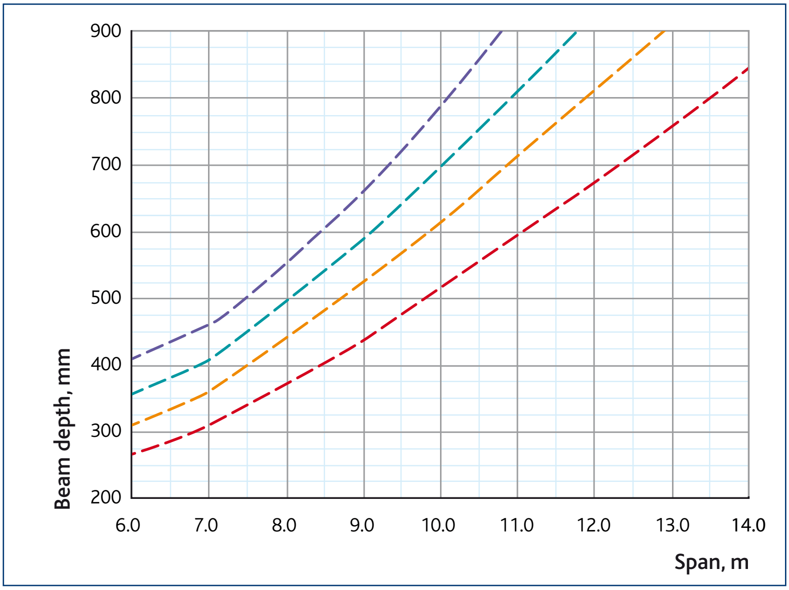
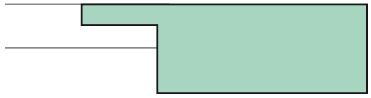


Figure 3.16
Span:depth chart for single-span inverted L-beams, 900 mm web

Table 3.16
Data for single-span inverted L-beams, 900 mm web

SINGLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 25 kN/m	261	306	367	432	514	589	668	752	842
uaudl = 50 kN/m	304	356	436	519	609	706	803		
uaudl = 100 kN/m	352	403	491	587	691	803			
uaudl = 200 kN/m	405	462	553	665	789	921			
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	n/a (89)	n/a (108)	n/a (130)	n/a (155)	n/a (183)	n/a (213)	n/a (246)	n/a (282)	n/a (321)
uaudl = 50 kN/m	n/a (167)	n/a (200)	n/a (238)	n/a (278)	n/a (322)	n/a (369)	n/a (419)	n/a (472)	n/a (529)
uaudl = 100 kN/m	n/a (321)	n/a (380)	n/a (444)	n/a (512)	n/a (583)	n/a (659)	n/a (739)	n/a (824)	
uaudl = 200 kN/m	n/a (626)	n/a (736)	n/a (851)	n/a (972)	n/a (1097)	n/a (1227)	n/a (1362)		
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	32 (138)	32 (116)	36 (108)	40 (104)	45 (97)	49 (93)	56 (93)	70 (103)	78 (103)
uaudl = 50 kN/m	46 (169)	48 (148)	47 (119)	51 (108)	61 (112)	70 (110)	72 (100)		
uaudl = 100 kN/m	65 (205)	74 (203)	85 (193)	83 (157)	109 (176)	94 (130)			
uaudl = 200 kN/m	113 (311)	111 (267)	129 (259)	130 (216)	138 (194)	143 (173)			
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	306	OK	OK	OK	OK	OK	OK	OK	OK
4 hours fire	327	372	448	532	622	719	816		
Exp. XD1 + C40/50	298	340	410	487	570	656	744	835	
ψ_2 = 0.8	317	363	440	524	615	715	815		

3.2.11 Inverted L-beams, single span, 1200 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Single span

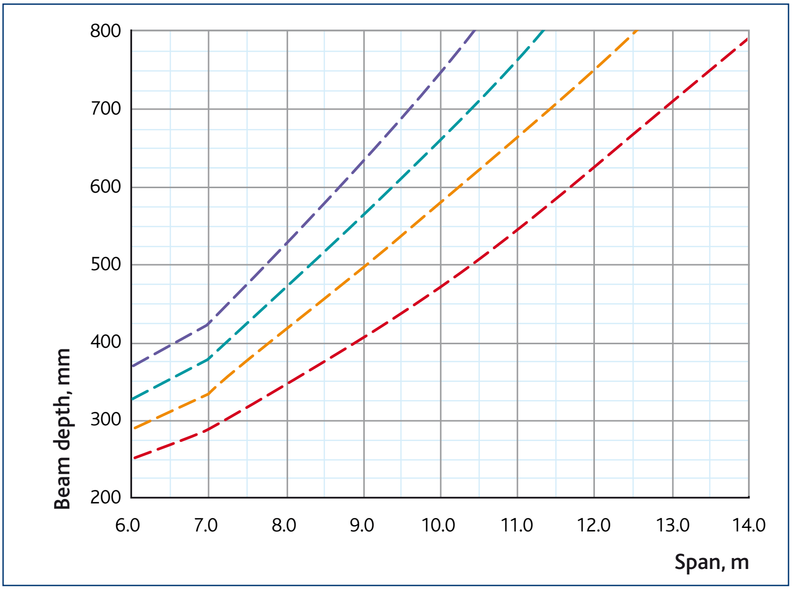
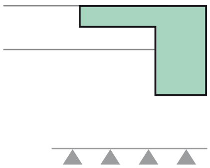


Figure 3.17
Span:depth chart for single-span inverted L-beams, 1200 mm web

Table 3.17
Data for single-span inverted L-beams, 1200 mm web

SINGLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 25 kN/m	250	285	342	402	467	541	624	704	790
uaudl = 50 kN/m	285	332	414	492	575	659	748	841	
uaudl = 100 kN/m	326	375	468	558	655	758	869		
uaudl = 200 kN/m	367	419	520	627	742	863			
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	92	112	136	163	194	228	268	310	356
uaudl = 50 kN/m	171	205	247	291	339	390	446	506	
uaudl = 100 kN/m	325	386	455	527	604	686	773		
uaudl = 200 kN/m	630	742	863	989	1120	1257			
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	36 (121)	40 (116)	41 (100)	47 (97)	60 (106)	68 (104)	66 (88)	74 (87)	85 (90)
uaudl = 50 kN/m	53 (156)	51 (129)	52 (105)	60 (101)	65 (95)	87 (110)	93 (104)	113 (112)	
uaudl = 100 kN/m	79 (203)	93 (207)	84 (150)	108 (161)	115 (146)	126 (139)	117 (112)		
uaudl = 200 kN/m	138 (312)	133 (265)	140 (225)	144 (191)	150 (168)	158 (153)			
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire									
4 hours fire	313	356	427	505	588	673	762	855	
Exp. XD1 + C40/50	281	325	390	462	537	613	692	776	866
$\psi_2 = 0.8$	292	335	418	497	582	670	761	857	

3.2.12 Inverted L-beams, multiple span, 225 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

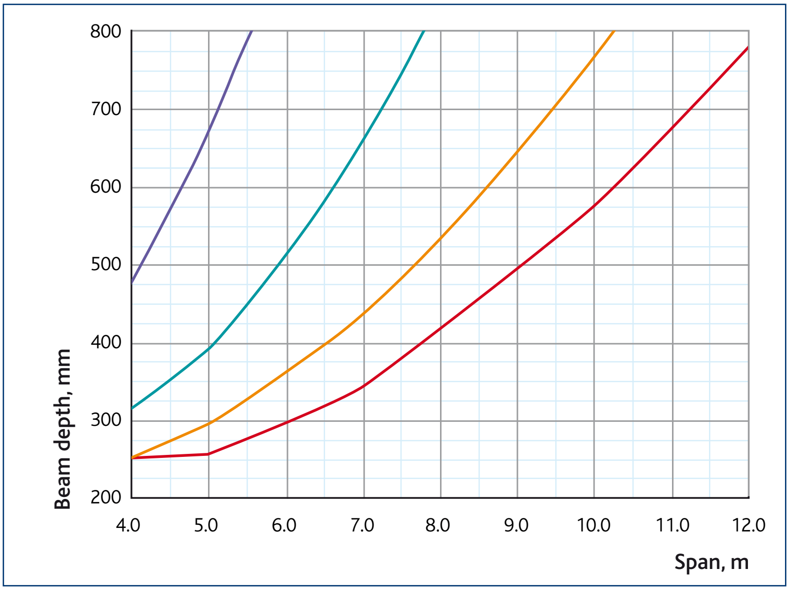
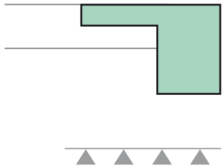


Figure 3.18
Span:depth chart for multiple-span inverted L-beams, 225 mm web

Table 3.18
Data for multiple-span inverted L-beams, 225 mm web

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	250	255	296	340	417	493	573	669	776
uaudl = 50 kN/m	250	293	360	431	530	641	763	896	
uaudl = 100 kN/m	313	390	509	655	839				
uaudl = 200 kN/m	475	668	917						
Ultimate load to supports/columns, each end, kN ult									
uaudl = 25 kN/m	104 (52)	130 (65)	158 (79)	187 (93)	218 (109)	250 (125)	283 (142)	319 (160)	357 (179)
uaudl = 50 kN/m	204 (102)	257 (128)	311 (155)	366 (183)	424 (212)	484 (242)	547 (273)	612 (306)	
uaudl = 100 kN/m	406 (203)	510 (255)	617 (309)	727 (364)	842 (421)				
uaudl = 200 kN/m	811 (405)	1020 (510)	1234 (617)						
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	9 (165)	13 (222)	16 (243)	19 (242)	19 (207)	20 (177)	20 (152)	20 (135)	20 (117)
uaudl = 50 kN/m	18 (313)	24 (366)	24 (300)	24 (244)	23 (194)	23 (161)	26 (153)	27 (132)	
uaudl = 100 kN/m	29 (414)	32 (367)	31 (267)	30 (204)	30 (158)				
uaudl = 200 kN/m	36 (336)	34 (226)	35 (167)						
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	Increase b_w to 280 mm min.								
Exp. XD1 + C40/50	250	295	361	431	526	632	748	875	
ψ_2 = 0.8	250	304	376	452	556	673	800		

3.2.13 Inverted L-beams, multiple span, 300 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

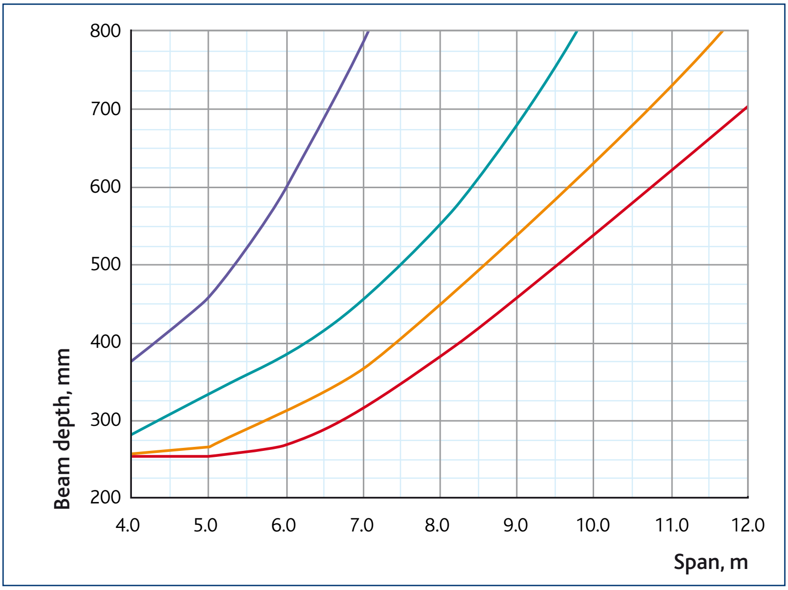
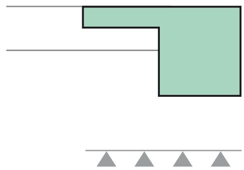


Figure 3.19
Span:depth chart for multiple-span inverted L-beams, 300 mm web

Table 3.19
Data for multiple-span inverted L-beams, 300 mm web

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	250	250	266	307	377	455	540	620	700
uaudl = 50 kN/m	250	262	309	363	446	535	630	727	830
uaudl = 100 kN/m	276	331	378	447	544	674	831		
uaudl = 200 kN/m	371	454	592	777					
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 25 kN/m	106 (53)	132 (66)	159 (80)	189 (94)	221 (110)	255 (127)	291 (146)	329 (164)	368 (184)
uaudl = 50 kN/m	206 (103)	258 (129)	312 (156)	367 (184)	426 (213)	487 (243)	550 (275)	615 (307)	682 (341)
uaudl = 100 kN/m	407 (203)	511 (255)	616 (308)	723 (361)	833 (417)	948 (474)	1069 (534)		
uaudl = 200 kN/m	810 (405)	1017 (508)	1228 (614)	1444 (722)					
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	10 (129)	14 (180)	17 (217)	21 (231)	20 (181)	20 (148)	21 (129)	23 (124)	24 (115)
uaudl = 50 kN/m	15 (195)	26 (332)	28 (303)	31 (285)	32 (243)	33 (208)	36 (188)	36 (163)	36 (143)
uaudl = 100 kN/m	28 (334)	37 (375)	43 (383)	49 (365)	48 (294)	46 (230)	44 (177)		
uaudl = 200 kN/m	45 (401)	54 (393)	52 (296)	50 (214)					
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	281	317	352	399	483	578	669	777	895
Exp. XD1 + C40/50	250	265	314	368	451	537	632	736	848
ψ_2 = 0.8	250	265	318	365	454	543	635	737	841

3.2.14 Inverted L-beams, multiple span, 450 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

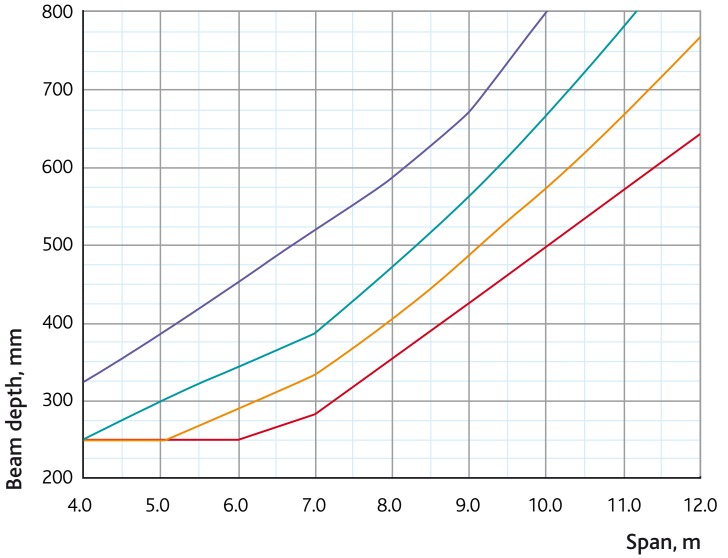
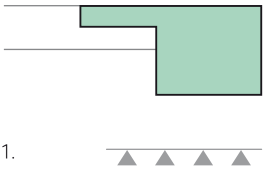


Figure 3.20
Span:depth chart for multiple-span inverted L-beams 450 mm web

Table 3.20
Data for multiple-span inverted L-beams, 450 mm web

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 25 kN/m	250	250	250	284	356	424	497	570	646
uaudl = 50 kN/m	250	250	290	332	404	484	571	666	769
uaudl = 100 kN/m	253	302	343	384	469	562	664	780	898
uaudl = 200 kN/m	324	385	454	520	584	669	800		
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 25 kN/m	108 (54)	136 (68)	163 (81)	193 (97)	229 (114)	266 (133)	306 (153)	348 (174)	392 (196)
uaudl = 50 kN/m	208 (104)	261 (130)	316 (158)	373 (186)	434 (217)	499 (249)	566 (283)	638 (319)	713 (356)
uaudl = 100 kN/m	409 (204)	514 (257)	621 (310)	728 (364)	842 (421)	958 (479)	1079 (540)	1205 (603)	1335 (667)
uaudl = 200 kN/m	813 (406)	1020 (510)	1230 (615)	1441 (721)	1654 (827)	1872 (936)	2098 (1049)		
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	10 (91)	13 (119)	19 (173)	22 (173)	21 (134)	23 (123)	24 (108)	26 (101)	30 (103)
uaudl = 50 kN/m	15 (134)	26 (232)	30 (232)	36 (243)	35 (190)	37 (169)	39 (153)	40 (134)	41 (120)
uaudl = 100 kN/m	32 (278)	41 (301)	53 (340)	60 (346)	62 (293)	62 (246)	62 (207)	63 (178)	66 (164)
uaudl = 200 kN/m	54 (373)	63 (362)	74 (362)	81 (345)	93 (352)	101 (335)	97 (271)		
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	250	282	311	351	430	515	602	697	800
Exp. XD1 + C40/50	250	250	278	317	384	458	538	634	731
ψ_2 = 0.8	250	251	292	334	407	488	580	683	787

3.2.15 Inverted L-beams, multiple span, 600 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

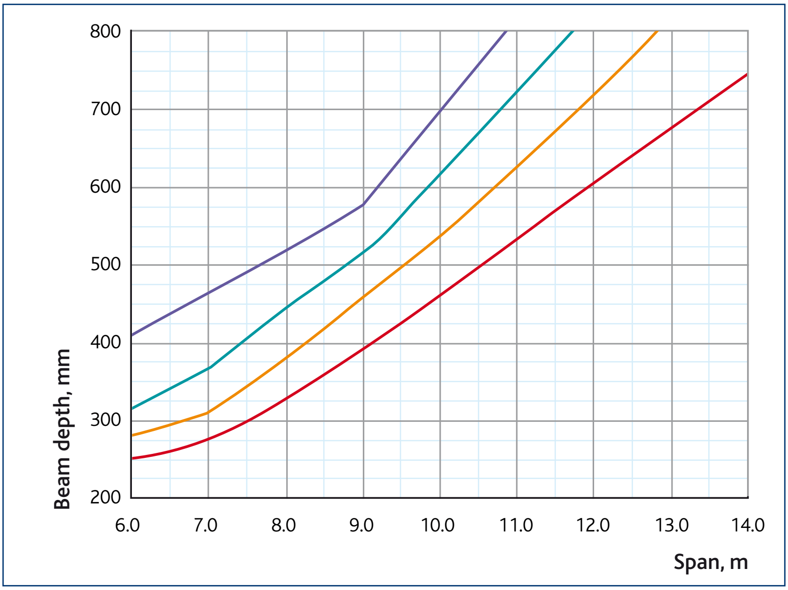
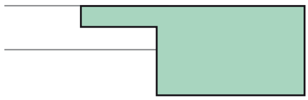


Figure 3.21
Span:depth chart for multiple-span inverted L-beams, 600 mm web

Table 3.21
Data for multiple-span inverted L-beams, 600 mm web

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 25 kN/m	250	270	327	388	463	529	598	671	748
uaudl = 50 kN/m	278	306	378	457	536	622	715	814	
uaudl = 100 kN/m	312	364	436	522	615	719	828		
uaudl = 200 kN/m	406	468	524	576	698	815			
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 25 kN/m	167 (83)	197 (99)	234 (117)	274 (137)	318 (159)	363 (182)	412 (206)	464 (232)	520 (260)
uaudl = 50 kN/m	320 (160)	377 (189)	442 (221)	510 (255)	582 (291)	658 (329)	738 (369)	824 (412)	
uaudl = 100 kN/m	624 (312)	735 (367)	850 (425)	971 (486)	1097 (548)	1228 (614)	1364 (682)		
uaudl = 200 kN/m	1234 (617)	1448 (724)	1664 (832)	1880 (940)	2112(1056)	2347(1174)			
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	20 (133)	25 (157)	24 (122)	25 (109)	26 (92)	28 (89)	31 (86)	33 (83)	39 (87)
uaudl = 50 kN/m	33 (200)	37 (202)	41 (181)	40 (145)	41 (128)	42 (112)	46 (106)	50 (103)	
uaudl = 100 kN/m	55 (292)	60 (273)	64 (246)	66 (212)	69 (186)	69 (160)	71 (143)		
uaudl = 200 kN/m	76 (311)	86 (306)	102 (325)	124 (359)	117 (279)	118 (240)			
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	280	319	389	459	538	624	717	816	
Exp. XD1 + C40/50	266	302	364	432	505	584	669	758	846
ψ_2 = 0.8	279	319	386	460	541	628	722	823	

3.2.16 Inverted L-beams, multiple span, 900 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

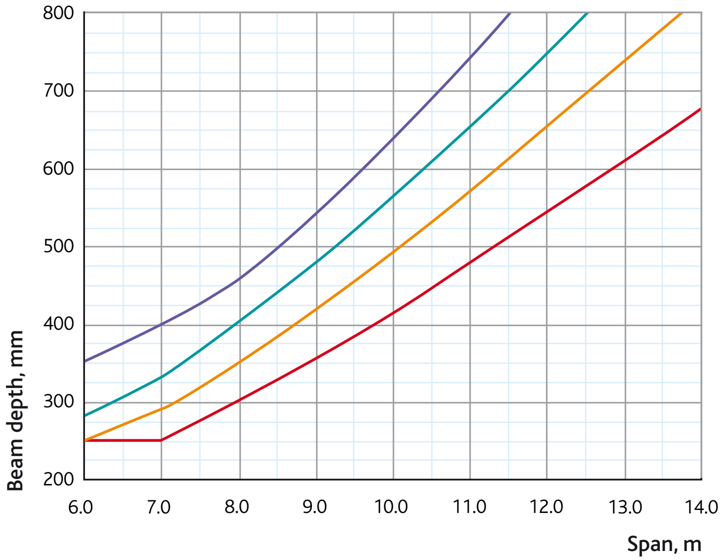
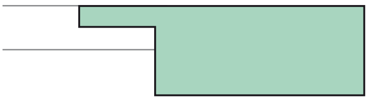


Figure 3.22
Span:depth chart for multiple-span inverted L-beams 900 mm web

Table 3.22
Data for multiple-span inverted L-beams, 900 mm web

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 25 kN/m	250	250	298	353	408	479	541	606	676
uaudl = 50 kN/m	251	287	347	416	490	568	653	734	818
uaudl = 100 kN/m	280	331	401	478	561	650	744	845	
uaudl = 200 kN/m	350	398	455	538	635	741	852		
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 25 kN/m	175 (88)	205 (102)	245 (122)	289 (145)	337 (168)	392 (196)	449 (224)	510 (255)	577 (288)
uaudl = 50 kN/m	325 (163)	387 (193)	456 (228)	530 (265)	610 (305)	695 (347)	787 (393)	882 (441)	983 (491)
uaudl = 100 kN/m	630 (315)	745 (373)	868 (434)	996 (498)	1130 (565)	1270 (635)	1417 (709)	1572 (786)	
uaudl = 200 kN/m	1242 (621)	1459 (729)	1680 (840)	1911 (955)	2150 (1075)	2398(1199)	2654(1327)		
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	22 (99)	30 (132)	31 (115)	31 (96)	33 (90)	35 (81)	38 (78)	43 (79)	47 (76)
uaudl = 50 kN/m	38 (169)	43 (168)	43 (137)	46 (123)	45 (102)	49 (96)	54 (91)	59 (89)	70 (95)
uaudl = 100 kN/m	61 (243)	67 (225)	69 (192)	74 (172)	80 (158)	79 (134)	92 (138)	86 (113)	
uaudl = 200 kN/m	94 (298)	101 (283)	121 (296)	125 (259)	128 (224)	129 (194)	138 (180)		
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	253	289	349	No change					
Exp. XD1 + C40/50	250	280	335	396	461	532	606	681	757
ψ_2 = 0.8	254	299	361	427	499	577	661	745	832

3.2.17 Inverted L-beams, multiple span, 1200 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m

Multiple span

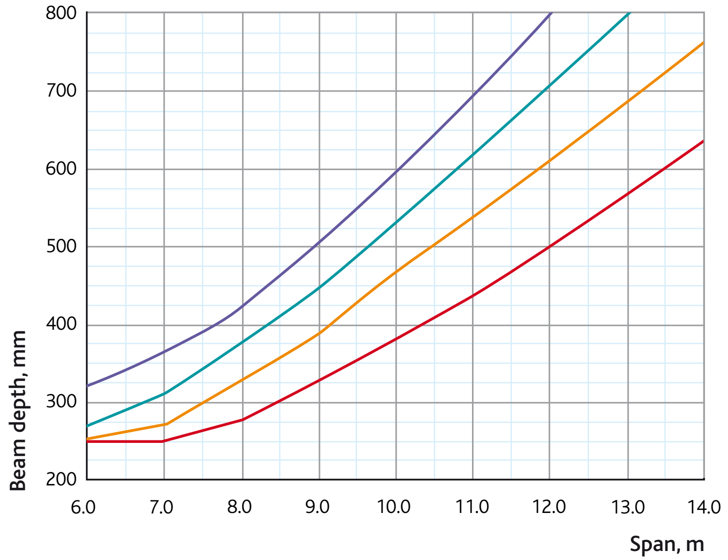
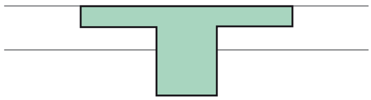


Figure 3.23
Span:depth chart for multiple-span inverted L-beams, 1200 mm web

Table 3.23
Data for multiple-span inverted L-beams, 1200 mm web

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 25 kN/m	250	250	275	329	380	434	496	566	632
uaudl = 50 kN/m	250	269	325	387	464	538	609	683	760
uaudl = 100 kN/m	269	309	373	444	532	615	703	796	895
uaudl = 200 kN/m	319	361	419	503	593	691	795	906	
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 25 kN/m	184 (92)	214 (107)	253 (126)	302 (151)	355 (178)	413 (206)	478 (239)	552 (276)	629 (315)
uaudl = 50 kN/m	334 (167)	394 (197)	468 (234)	547 (273)	637 (318)	731 (365)	829 (415)	934 (467)	1047 (523)
uaudl = 100 kN/m	638 (319)	755 (377)	882 (441)	1016 (508)	1162 (581)	1312 (656)	1471 (736)	1639 (820)	1817 (909)
uaudl = 200 kN/m	1249 (625)	1469 (734)	1696 (848)	1936 (968)	2185(1092)	2444(1222)	2713(1356)	2993(1496)	
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	26 (87)	32 (106)	38 (115)	37 (92)	38 (83)	43 (83)	48 (81)	52 (76)	55 (73)
uaudl = 50 kN/m	40 (133)	52 (160)	52 (132)	48 (104)	51 (91)	54 (83)	61 (84)	69 (84)	80 (88)
uaudl = 100 kN/m	66 (203)	76 (205)	81 (180)	87 (163)	88 (137)	86 (117)	98 (116)	94 (99)	120 (111)
uaudl = 200 kN/m	104 (272)	128 (295)	134 (267)	155 (257)	165 (231)	164 (197)	172 (180)	157 (144)	
Variations: implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	250	276	332	394	No change				
Exp. XD1 + C40/50	250	267	316	375	436	499	566	633	702
ψ_2 = 0.8	250	275	344	406	473	546	619	695	774

3.2.18 T-beams, single span, 300 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Single span

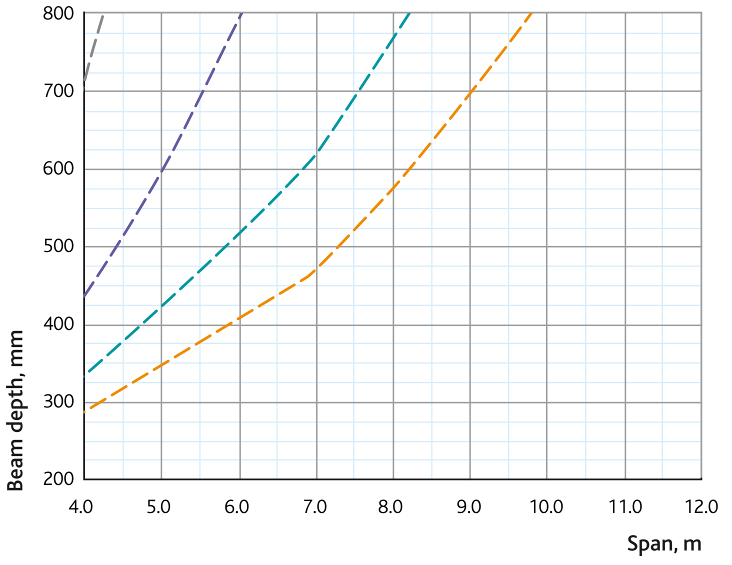


Figure 3.24
Span:depth chart for single-span T-beams, 300 mm web

Table 3.24
Data for single-span T-beams, 300 mm web

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 50 kN/m	287	342	404	467	574	693	825		
uaudl = 100 kN/m	329	418	513	616	763	925			
uaudl = 200 kN/m	427	586	789						
uaudl = 400 kN/m	717	1056							
Ultimate load to supports/columns, each end, kN ult									
uaudl = 50 kN/m	105	134	163	193	227	263	301		
uaudl = 100 kN/m	206	261	317	375	437	502			
uaudl = 200 kN/m	409	517	629	747					
uaudl = 400 kN/m	817								
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	25 (290)	24 (238)	31 (259)	32 (225)	31 (181)	31 (151)	32 (128)		
uaudl = 100 kN/m	34 (349)	34 (272)	35 (228)	37 (199)	36 (159)	37 (135)			
uaudl = 200 kN/m	40 (311)	39 (225)	39 (166)						
uaudl = 400 kN/m	45 (208)								
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	406	517	700	927					
Exp. XD1 + C40/50	322	414	506	606	746	912			
ψ_2 = 0.8	341	435	535	643	797	967			

3.2.19 T-beams, single span, 600 mm web

Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Single span

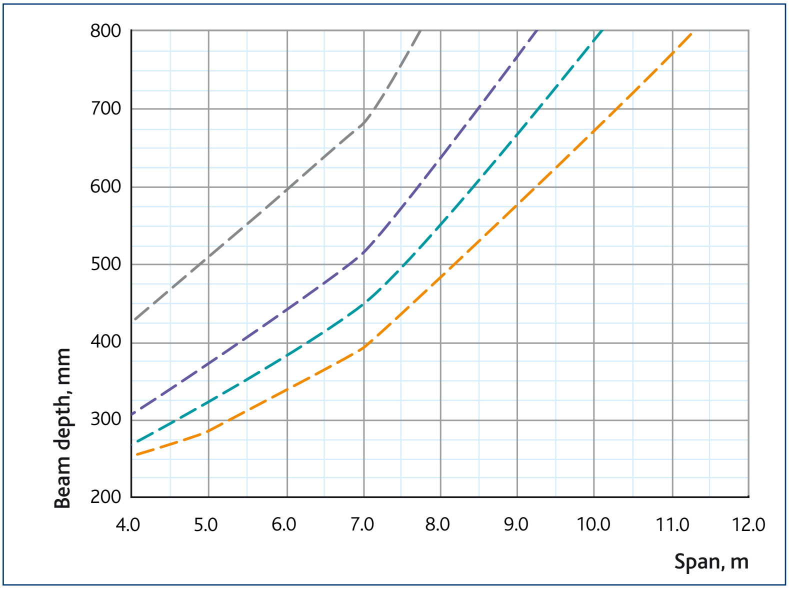


Figure 3.25
Span:depth chart for single-span T-beams, 600 mm web

Table 3.25
Data for single-span T-beams, 600 mm web

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 50 kN/m	250	282	335	388	477	573	671	768	877
uaudl = 100 kN/m	264	320	378	446	552	669	784	912	
uaudl = 200 kN/m	303	368	437	511	636	767	901		
uaudl = 400 kN/m	422	508	590	677	840				
Ultimate load to supports/columns, each end, kN ult									
uaudl = 50 kN/m	106	134	163	194	228	265	304	344	387
uaudl = 100 kN/m	206	260	316	373	434	498	564	634	
uaudl = 200 kN/m	408	513	619	727	840	956	1075		
uaudl = 400 kN/m	812	1019	1228	1438	1656				
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	34 (227)	40 (236)	44 (219)	48 (204)	52 (181)	52 (151)	55 (136)	58 (126)	59 (111)
uaudl = 100 kN/m	45 (284)	50 (258)	57 (252)	64 (239)	65 (195)	71 (177)	74 (158)	79 (145)	81 (129)
uaudl = 200 kN/m	68 (376)	84 (381)	99 (377)	114 (373)	113 (296)	113 (245)	117 (217)	117 (187)	
uaudl = 400 kN/m	86 (338)	102 (334)	120 (338)	139 (341)	135 (268)	133 (216)			
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	281	337	396	463	571	686	801	929	1059
Exp. XD1 + C40/50	254	306	369	430	529	633	744	857	981
$\psi_2 = 0.8$	266	322	388	455	561	678	795	923	1056

3.2.20 T-beams, single span, 1200 mm web

Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Single span

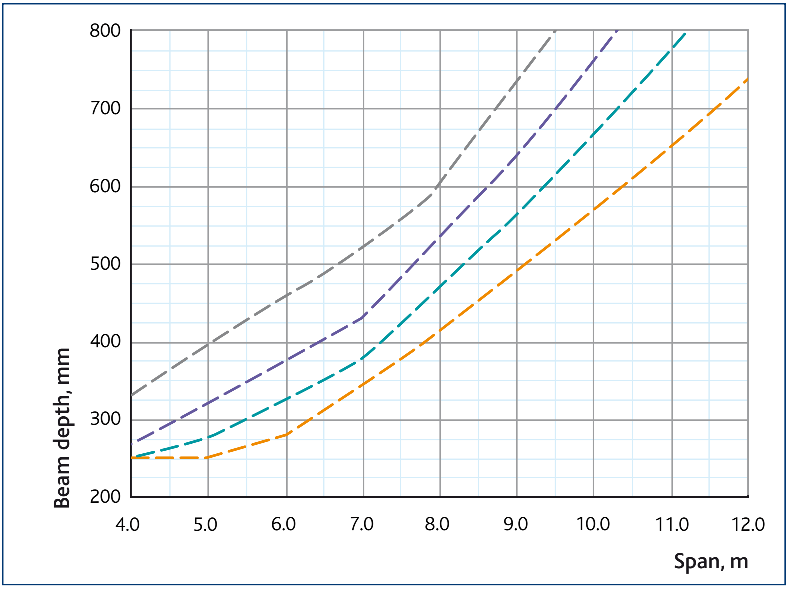


Figure 3.26
Span:depth chart for single-span T-beams, 1200 mm web

Table 3.26
Data for single-span T-beams,1200 mm web

SINGLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 50 kN/m	250	250	281	340	412	489	567	650	738
uaudl = 100 kN/m	250	276	325	376	470	561	662	771	888
uaudl = 200 kN/m	265	318	373	430	530	641	758	886	
uaudl = 400 kN/m	332	398	458	513	575	723	860		
Ultimate load to supports/columns, each end, kN ult									
uaudl = 50 kN/m	111	139	170	207	247	291	338	388	444
uaudl = 100 kN/m	211	267	325	386	456	528	605	688	777
uaudl = 200 kN/m	412	520	631	743	865	991	1123	1262	
uaudl = 400 kN/m	817	1028	1240	1454	1671	1905	2143		
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	29 (98)	43 (144)	55 (163)	53 (130)	55 (111)	62 (106)	78 (115)	91 (117)	104 (117)
uaudl = 100 kN/m	45 (151)	65 (197)	83 (213)	97 (215)	95 (168)	94 (139)	121 (152)	126 (136)	116 (109)
uaudl = 200 kN/m	82 (258)	106 (278)	133 (298)	162 (314)	137 (216)	143 (186)	150 (165)	157 (148)	
uaudl = 400 kN/m	119 (299)	149 (312)	186 (338)	224 (365)	284 (411)	238 (274)	240 (232)		
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	252	299	348	401	482	574	674	784	901
Exp. XD1 + C40/50	250	275	320	370	448	533	626	726	832
ψ_2 = 0.8	250	289	339	388	473	566	668	779	898

3.2.21 T-beams, single span, 2400 mm web

Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Single span

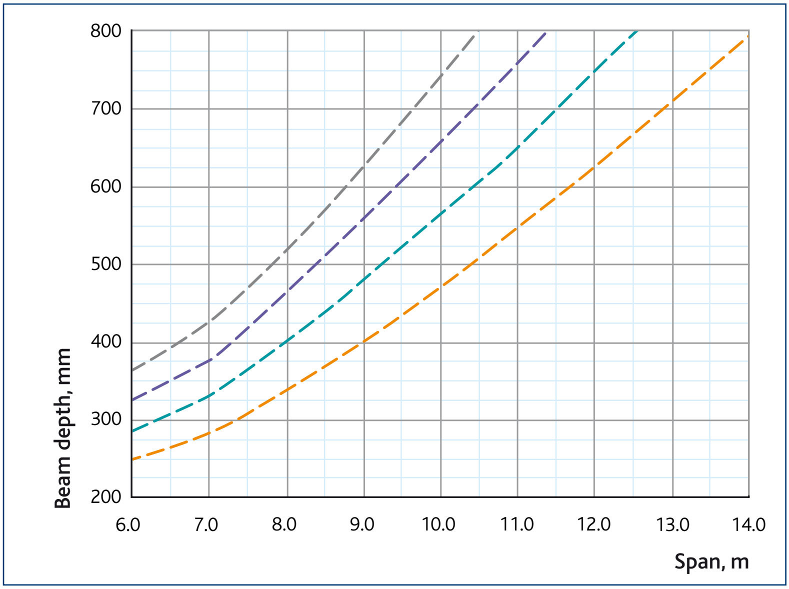
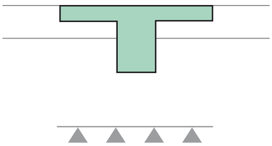


Figure 3.27
Span:depth chart for single-span T-beams, 2400 mm web

Table 3.27
Data for single-span T-beams, 2400 mm web

SINGLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 50 kN/m	250	282	337	400	467	550	624	704	790
uaudl = 100 kN/m	287	330	401	481	563	648	748	841	938
uaudl = 200 kN/m	326	378	460	559	656	758	871	988	
uaudl = 400 kN/m	363	426	518	629	739	860			
Ultimate load to supports/columns, each end, kN ult									
uaudl = 50 kN/m	184	223	271	326	388	461	536	619	712
uaudl = 100 kN/m	342	410	490	579	674	776	892	1011	1140
uaudl = 200 kN/m	651	773	908	1055	1209	1371	1547	1733	
uaudl = 400 kN/m	1259	1486	1725	1979	2240	2514			
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	94 (157)	105 (155)	115 (142)	121 (126)	121 (108)	119 (90)	132 (88)	149 (88)	178 (94)
uaudl = 100 kN/m	121 (176)	137 (173)	145 (151)	144 (125)	165 (122)	188 (121)	207 (115)	239 (118)	253 (113)
uaudl = 200 kN/m	174 (222)	193 (213)	211 (191)	211 (157)	234 (149)	262 (144)	281 (134)	299 (126)	
uaudl = 400 kN/m	364 (418)	325 (317)	354 (285)	362 (240)	300 (169)	318 (154)			
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	313	356	427	505	592	677	762	855	
Exp. XD1 + C40/50	281	321	389	460	535	611	691	778	866
$\psi_2 = 0.8$	289	346	418	497	582	670	761	857	

3.2.22 T-beams, multiple span, 300 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Multiple span

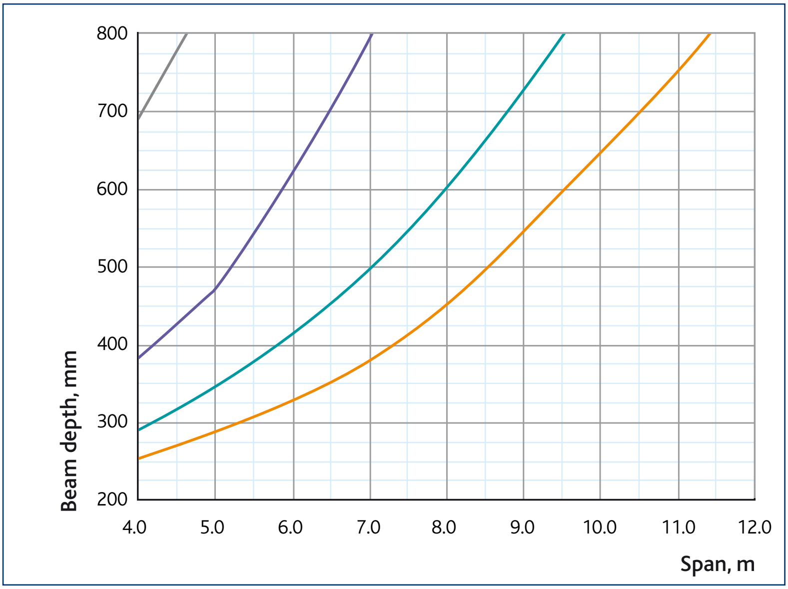
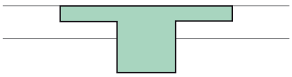


Figure 3.28
Span:depth chart for multiple-span T-beams, 300 mm web

Table 3.28
Data for multiple-span T-beams, 300 mm web

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 50 kN/m	250	285	325	377	455	545	644	752	869
uaudl = 100 kN/m	286	342	411	489	599	723	859		
uaudl = 200 kN/m	380	466	611	793					
uaudl = 400 kN/m	690	859							
Ultimate load to supports/columns, each end, kN ult									
uaudl = 50 kN/m	206 (103)	259 (129)	313 (156)	368 (184)	427 (213)	488 (244)	551 (276)	617 (309)	687 (343)
uaudl = 100 kN/m	407 (203)	511 (256)	617 (309)	726 (363)	837 (419)	953 (476)	1071 (536)		
uaudl = 200 kN/m	811 (405)	1017 (509)	1229 (614)	1445 (723)					
uaudl = 400 kN/m	1622 (811)	2036 (1018)							
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	17 (229)	26 (308)	30 (312)	32 (281)	34 (248)	35 (214)	35 (183)	35 (156)	37 (142)
uaudl = 100 kN/m	29 (338)	41 (402)	39 (317)	42 (288)	41 (231)	43 (200)	43 (168)		
uaudl = 200 kN/m	44 (389)	51 (367)	51 (279)	49 (207)					
uaudl = 400 kN/m	49 (234)	60 (235)							
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	332	381	430	507	618	741	877		
Exp. XD1 + C40/50	275	335	406	481	588	707	837		
$\psi_2 = 0.8$	295	354	430	512	628	758	901		

3.2.23 T-beams, multiple span, 450 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Multiple span

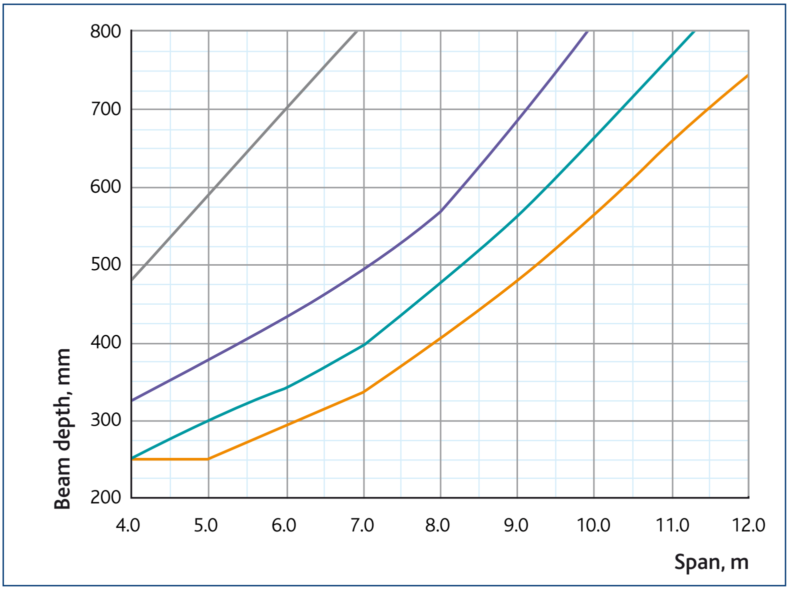
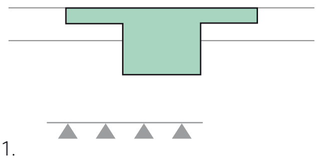


Figure 3.29
Span:depth chart for multiple-span T-beams, 450 mm web

Table 3.29
Data for multiple-span T-beams, 450 mm web

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 50 kN/m	250	250	292	334	403	478	561	655	745
uaudl = 100 kN/m	253	302	343	394	475	562	663	764	872
uaudl = 200 kN/m	324	377	430	491	567	683	811		
uaudl = 400 kN/m	475	587	699	812					
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 50 kN/m	208 (104)	261 (130)	316 (158)	373 (187)	434 (217)	498 (249)	565 (282)	636 (318)	709 (354)
uaudl = 100 kN/m	409 (204)	514 (257)	621 (310)	729 (364)	842 (421)	958 (479)	1079 (540)	1203 (601)	1330 (665)
uaudl = 200 kN/m	813 (406)	1019 (510)	1228 (614)	1438 (719)	1653 (826)	1874 (937)	2100 (1050)		
uaudl = 400 kN/m	1621 (811)	2034(1017)	2451(1225)	2870(1435)					
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	15 (134)	26 (232)	33 (250)	36 (240)	36 (199)	40 (185)	40 (159)	41 (138)	45 (134)
uaudl = 100 kN/m	31 (276)	41 (300)	56 (363)	57 (324)	59 (278)	62 (246)	65 (219)	67 (194)	68 (174)
uaudl = 200 kN/m	53 (366)	64 (377)	81 (417)	90 (409)	102 (398)	100 (324)	96 (263)		
uaudl = 400 kN/m	75 (353)	86 (325)	96 (306)	110 (300)					
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	297	339	382	420	501	589	686	790	907
Exp. XD1 + C40/50	250	281	324	378	453	542	630	729	828
ψ_2 = 0.8	255	305	346	396	483	572	668	775	884

3.2.24 T-beams, multiple span, 600 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Multiple span

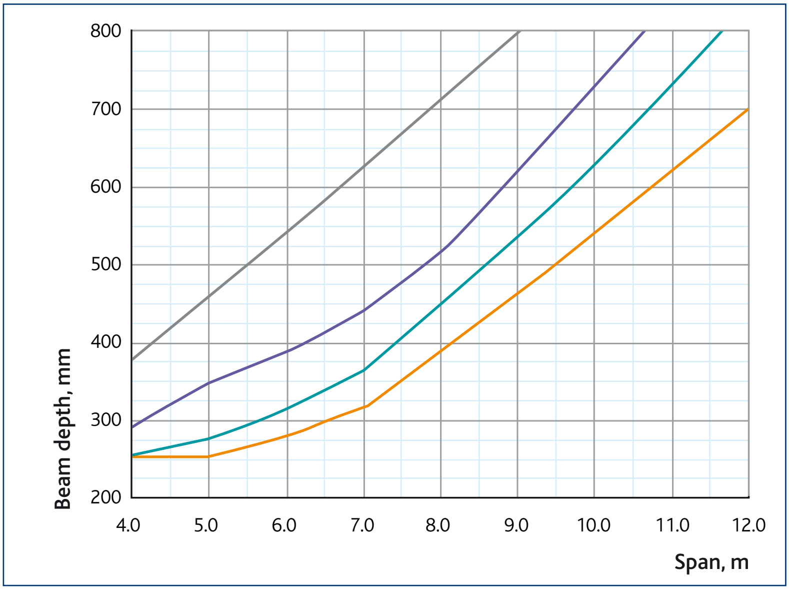
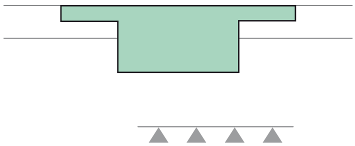


Figure 3.30
Span:depth chart for multiple-span T-beams, 600 mm web

Table 3.30
Data for multiple-span T-beams, 600 mm web

MULTIPLE span, m	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Depth, mm									
uaudl = 50 kN/m	250	250	276	314	388	463	541	621	701
uaudl = 100 kN/m	250	271	312	363	448	537	627	730	833
uaudl = 200 kN/m	288	345	388	436	515	619	727	839	
uaudl = 400 kN/m	378	456	540	626	711	798	951		
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 50 kN/m	211 (106)	264 (132)	320 (160)	378 (189)	443 (222)	511 (256)	583 (291)	657 (329)	735 (368)
uaudl = 100 kN/m	411 (206)	516 (258)	624 (312)	735 (367)	852 (426)	974 (487)	1099 (549)	1230 (615)	1365 (682)
uaudl = 200 kN/m	814 (407)	1023 (511)	1232 (616)	1444 (722)	1662 (831)	1888 (944)	2118 (1059)	2352 (1176)	
uaudl = 400 kN/m	1621 (810)	2033(1017)	2450(1225)	2869(1435)	3292 (1646)	3718 (1859)	4160 (2080)		
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	17 (115)	25 (167)	34 (207)	41 (216)	40 (172)	40 (143)	40 (124)	46 (123)	48 (115)
uaudl = 100 kN/m	28 (188)	47 (286)	54 (290)	61 (278)	62 (231)	67 (208)	67 (179)	68 (156)	74 (147)
uaudl = 200 kN/m	53 (306)	64 (308)	85 (364)	106 (404)	112 (361)	107 (289)	115 (265)	114 (227)	
uaudl = 400 kN/m	88 (387)	109 (400)	127 (392)	139 (371)	154 (362)	174 (364)	166 (291)		
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	250	282	317	365	450	539	629	732	835
Exp. XD1 + C40/50	250	258	297	350	427	513	599	689	788
ψ_2 = 0.8	250	273	316	367	453	547	638	740	846

3.2.25 T-beams, multiple span, 900 mm web



Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)

- 50 kN/m
- 100 kN/m
- 200 kN/m
- 400 kN/m

Multiple span

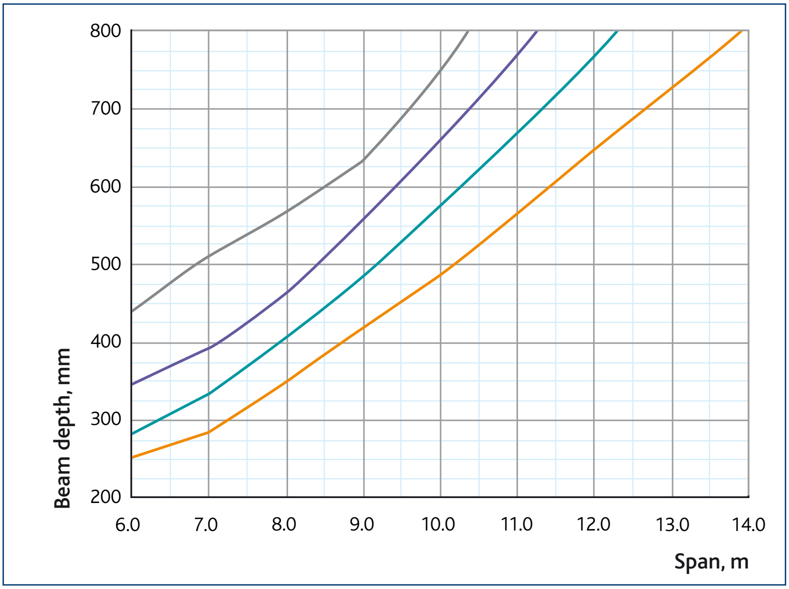


Figure 3.31
Span:depth chart for multiple-span T-beams, 900 mm web

Table 3.31
Data for multiple-span T-beams, 900 mm web

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 50 kN/m	250	284	345	417	485	561	646	726	805
uaudl = 100 kN/m	281	332	404	484	571	666	768	877	
uaudl = 200 kN/m	343	390	459	553	658	768	890		
uaudl = 400 kN/m	440	508	565	630	748	882			
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 50 kN/m	325 (163)	386 (193)	455 (228)	530 (265)	608 (304)	693 (346)	784 (392)	879 (439)	978 (489)
uaudl = 100 kN/m	631 (315)	746 (373)	868 (434)	997 (499)	1132 (566)	1275 (638)	1425 (713)	1584 (792)	
uaudl = 200 kN/m	1241 (621)	1457 (729)	1681 (840)	1915 (957)	2157 (1078)	2407 (1203)	2667 (1333)		
uaudl = 400 kN/m	2457(1229)	2880(1440)	3305(1652)	3734(1867)	4182 (2091)	4642 (2321)			
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	38 (168)	43 (169)	47 (150)	48 (127)	53 (120)	53 (105)	64 (110)	64 (98)	76 (105)
uaudl = 100 kN/m	62 (245)	68 (228)	71 (195)	78 (179)	75 (145)	88 (147)	93 (135)	83 (106)	
uaudl = 200 kN/m	99 (320)	111 (317)	119 (287)	122 (244)	125 (211)	130 (188)	130 (162)		
uaudl = 400 kN/m	149 (377)	167 (365)	194 (382)	227 (400)	222 (330)	212 (268)			
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	285	No change							
Exp. XD1 + C40/50	282	321	388	462	542	630	724	823	918
ψ_2 = 0.8	292	334	407	488	576	672	776	886	996

3.2.26 T-beams, multiple span, 1200 mm web

Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Multiple span

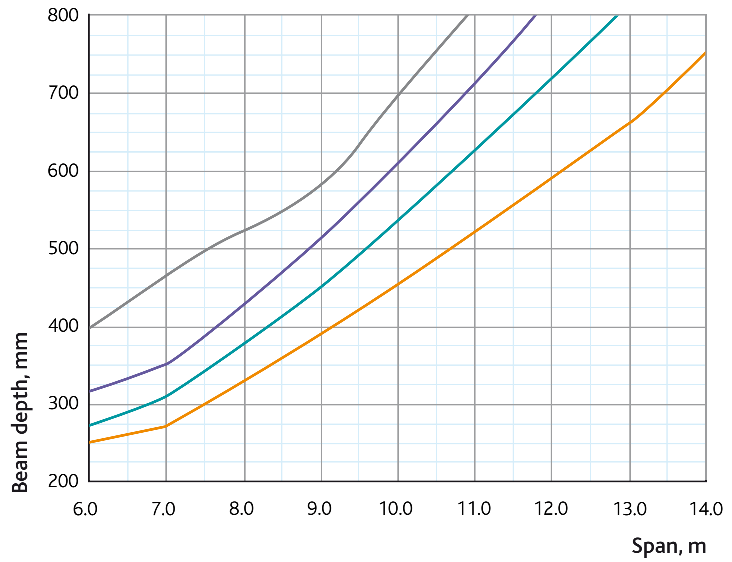


Figure 3.32
Span:depth chart for multiple-span T-beams, 1200 mm web

Table 3.32
Data for multiple-span T-beams, 1200 mm web

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 50 kN/m	250	269	326	391	454	519	589	662	748
uaudl = 100 kN/m	269	308	375	449	536	622	715	814	
uaudl = 200 kN/m	314	350	428	514	609	710	821		
uaudl = 400 kN/m	398	457	522	576	693	811			
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 50 kN/m	334 (167)	394 (197)	468 (234)	548 (274)	633 (316)	723 (361)	820 (410)	924 (462)	1040 (520)
uaudl = 100 kN/m	638 (319)	755 (377)	883 (441)	1018 (509)	1164 (582)	1315 (658)	1477 (738)	1648 (824)	1826 (913)
uaudl = 200 kN/m	1248 (624)	1466 (733)	1698 (849)	1940 (970)	2191 (1095)	2452 (1226)	2724 (1362)	3007 (1504)	
uaudl = 400 kN/m	2467(1234)	2894(1447)	3327(1663)	3761(1880)	4222 (2111)	4693 (2347)	5177 (2588)		
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	60 (200)	71 (219)	67 (172)	65 (139)	65 (119)	64 (103)	73 (104)	85 (107)	87 (96)
uaudl = 100 kN/m	86 (267)	96 (261)	94 (208)	95 (177)	94 (145)	96 (128)	101 (118)	111 (114)	
uaudl = 200 kN/m	125 (333)	149 (355)	151 (295)	154 (250)	159 (218)	165 (193)	144 (147)		
uaudl = 400 kN/m	177 (370)	184 (335)	209 (333)	243 (351)	239 (288)	240 (247)			
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	No change								
Exp. XD1 + C40/50	266	299	364	432	505	584	669	758	848
ψ_2 = 0.8	279	319	386	460	541	628	722	823	925

3.2.27 T-beams, multiple span, 1800 mm web

Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Multiple span

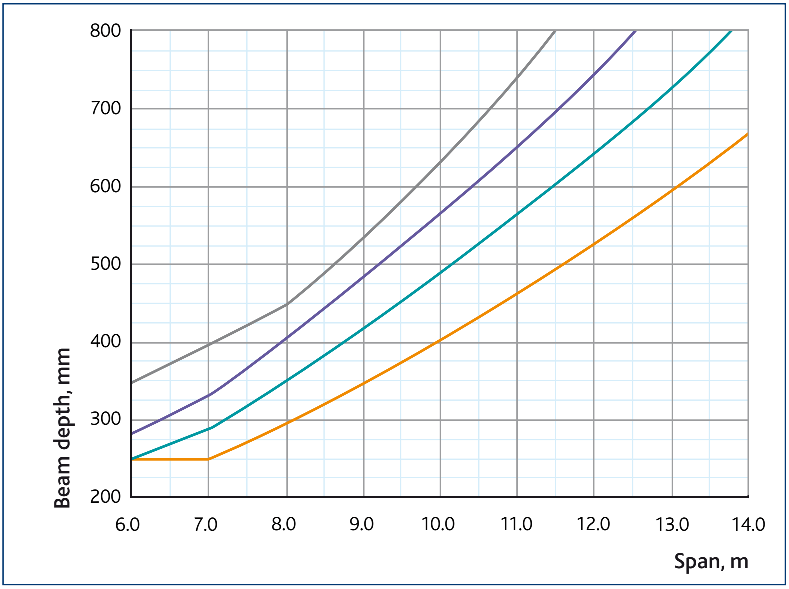


Figure 3.33
Span:depth chart for multiple-span T-beams, 1800 mm web

Table 3.33
Data for multiple-span T-beams, 1800 mm web

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 50 kN/m	250	250	297	348	403	461	528	594	663
uaudl = 100 kN/m	250	287	347	413	484	560	641	722	818
uaudl = 200 kN/m	283	331	401	478	561	650	744	845	
uaudl = 400 kN/m	346	398	450	533	630	736	848		
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 50 kN/m	351 (175)	409 (205)	489 (244)	576 (288)	670 (335)	773 (387)	889 (444)	1011 (506)	1143 (572)
uaudl = 100 kN/m	651 (325)	774 (387)	911 (456)	1058 (529)	1216 (608)	1385 (692)	1565 (783)	1755 (877)	1965 (983)
uaudl = 200 kN/m	1262 (631)	1491 (745)	1735 (868)	1991 (996)	2259 (1130)	2540 (1270)	2835 (1417)	3145 (1572)	
uaudl = 400 kN/m	2483(1242)	2917(1459)	3358(1679)	3819(1910)	4298 (2149)	4794 (2397)	5305 (2652)		
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	46 (103)	62 (139)	63 (119)	67 (107)	74 (101)	87 (105)	93 (98)	99 (93)	114 (96)
uaudl = 100 kN/m	77 (172)	90 (174)	90 (144)	94 (126)	105 (121)	117 (116)	126 (109)	135 (104)	144 (98)
uaudl = 200 kN/m	132 (258)	133 (224)	148 (205)	158 (183)	167 (166)	185 (158)	194 (145)	205 (135)	
uaudl = 400 kN/m	225 (362)	226 (316)	279 (345)	253 (264)	262 (231)	269 (203)	280 (184)		
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	252	289	349	415	486	562	643	724	818
Exp. XD1 + C40/50	250	280	335	396	461	532	604	681	757
ψ_2 = 0.8	250	289	350	427	499	577	661	745	832

3.2.28 T-beams, multiple span, 2400 mm web

Design assumptions

Design and dimensions – See Section 3.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m

Multiple span

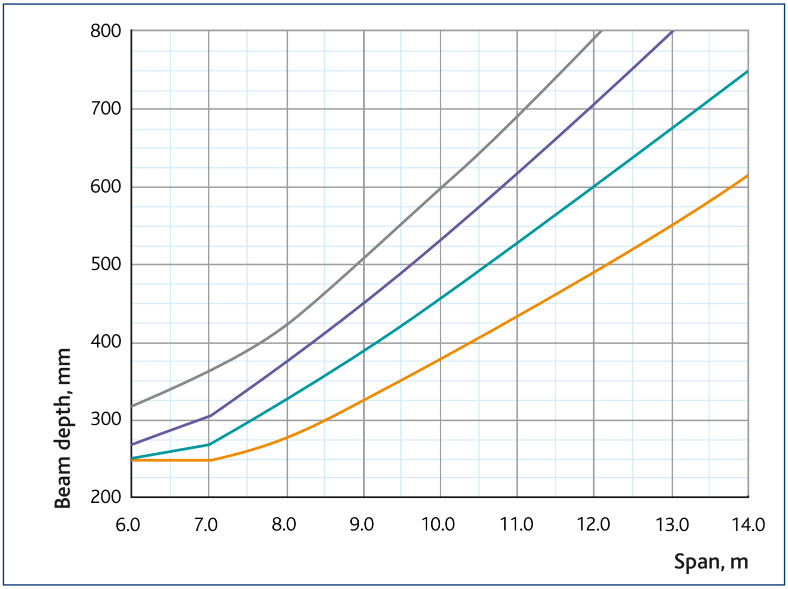


Figure 3.34
Span:depth chart for multiple-span T-beams, 2400 mm web

Table 3.34
Data for multiple-span T-beams, 2400 mm web

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Depth, mm									
uaudl = 50 kN/m	250	250	275	324	375	429	486	548	613
uaudl = 100 kN/m	250	269	325	386	453	527	597	671	748
uaudl = 200 kN/m	269	307	375	446	532	615	703	796	895
uaudl = 400 kN/m	317	360	419	503	593	689	793	903	
Ultimate load to supports/columns, internal (end), kN ult									
uaudl = 50 kN/m	368 (184)	429 (214)	505 (253)	601 (301)	706 (353)	821 (411)	947 (474)	1087 (543)	1239 (619)
uaudl = 100 kN/m	668 (334)	789 (394)	935 (468)	1093 (547)	1265 (632)	1452 (726)	1647 (824)	1857 (928)	2080(1040)
uaudl = 200 kN/m	1276 (638)	1509 (754)	1765 (883)	2034(1017)	2324 (1162)	2625 (1312)	2943 (1471)	3279 (1639)	3635(1817)
uaudl = 400 kN/m	2498(1249)	2937(1468)	3391(1696)	3872(1936)	4370 (2185)	4886 (2443)	5424 (2712)	5983 (2991)	
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	53 (89)	68 (113)	77 (116)	82 (106)	85 (95)	96 (93)	109 (93)	121 (92)	135 (92)
uaudl = 100 kN/m	82 (137)	102 (158)	105 (134)	109 (118)	121 (111)	132 (104)	146 (102)	164 (102)	177 (98)
uaudl = 200 kN/m	136 (210)	157 (212)	160 (178)	178 (166)	181 (142)	197 (134)	211 (125)	225 (118)	242 (113)
uaudl = 400 kN/m	214 (281)	251 (290)	300 (298)	310 (256)	319 (224)	340 (205)	347 (183)	361 (167)	
Variations: implications on beam depths for 100 kN/m uaudl, mm									
2 hours fire	No change								
4 hours fire	250	276	332	394	461	530	600	674	751
Exp. XD1 + C40/50	250	264	316	374	436	498	562	629	700
ψ_2 = 0.8	250	271	328	390	457	534	607	683	762

3.3 In-situ columns

3.3.1 Using in-situ columns

Columns support vertical loads and are often the most obvious and intrusive part of a structure. In-situ columns offer strength, economy, versatility, mouldability, fire resistance and robustness. Judgement is required to reconcile their position, size and shape with spans of horizontal elements and economy.

Generally the best economy comes from using regular square grids and constantly-sized columns. Ideally, the same column size should be used at all levels at all locations. If this is not possible, then the number of profiles should be kept to a minimum, e.g. one for internal columns and one for perimeter columns. Certainly up to about eight storeys, the same size and shape should be used throughout a column's height. The outside of edge columns should be flush with or inboard from the edges of slabs.

Chases, service penetrations and horizontal offsets should be avoided. Offsets are the cause of costly transition beams, which can be very disruptive to site progress.

Using high-strength concrete can decrease the size of columns required. Smaller columns occupy less lettable area and should be considered for individual projects. However, small quantities of high-strength concrete may be difficult to procure. Where the strength of the concrete in the column is more than 40% greater than that in the slab, special design details may be required[10].

For up to about five storeys the size of perimeter columns is dominated by moment: using concrete strengths greater than C40/50 appears to make little difference to the size of perimeter column required. Rectangular columns can be less obtrusive than square columns.

3.3.2 The charts and data

Column design depends on ultimate axial load, N_{Ed}, and ultimate design moment, M_{Ed}. For internal columns (see Figure 3.D), moments may generally be assumed to be nominal. Therefore the design chart for braced internal columns (Figure 3.35) gives square sizes against total ultimate axial load for a range of reinforcing steel contents.

However, in perimeter columns moments are generally critical. Therefore charts are provided so that 1st order moments, M, in edge and corner columns (see Figure 3.E) may be estimated according to whether they occur in beam-and-slab or flat slab construction. For an assumed column size, this moment and the ultimate axial load are used to interrogate moment:load charts – firstly to check the validity of the assumed column size and secondly to estimate amount of reinforcement required in that column size. Some iteration may be required (see Figure 3.E).

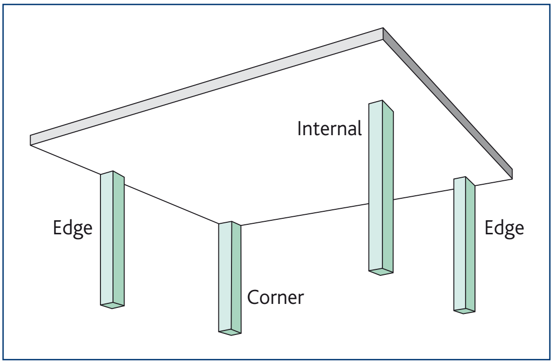


Figure 3.D
Internal, edge and corner columns

Knowing the amount of reinforcement required, Figure 3.45 and Table 3.40 allow bar arrangements to be judged and reinforcement densities estimated.

The column charts require that the total ultimate axial load, N_{Ed}, is available. The user should preferably calculate, or otherwise estimate, this load for the lowest level of column under consideration (see Section 8.4). The data for troughed slabs, flat slabs, wattle slabs and beams give 'ultimate loads to support or columns' per floor. These figures may be used subject to applying a suitable elastic reaction factor (see Section 8.3.2).

It should be noted that in the design of columns to Eurocode 2, the design moment M_{Ed} should allow not only for 1st order moments from analysis, M, but also for the effects from imperfections, $e_i N_{Ed}$, and in the case of slender columns for nominal 2nd order moments, M_2[7].

Note: The moment:load charts work on axial load N_{Ed} and, for relative simplicity, 1st order moment, M; the charts make due allowance for the effects of imperfections, slenderness and biaxial bending.

Internal columns

The load:size chart and data (Figure 3.35 and Table 3.35) for internal columns assume nominal moments only. Therefore, the size of column may be assessed from the chart by using the ultimate axial load and reading up to an appropriate reinforcement density and reading off the size on the vertical axis. The chart and data assume that the slabs and beams supported have equal spans in each orthogonal direction (i.e. $l_{y1} = l_{y2}$ and $l_{z1} = l_{z2}$). If spans differ by more than, say, 15%, consider treating internal columns as edge columns.

Where N_{Ed} and M_y have been calculated, Figure 3.36 may be used to assess the size required.

Edge and corner columns

The load:size design of perimeter columns depends on both ultimate axial load and ultimate design moment. The design of corner columns depends on ultimate moments in two directions. Design moments in columns are specific to that column and without doing full calculations can only be estimated by using a fair amount of conservatism.

Nonetheless, 1st order moments can be estimated from moment derivation charts as explained more fully in Section 7.1.5. These charts allow column moments to be estimated for a range of square column sizes according to whether the columns are within beam-and-slab construction or flat slab construction and whether they are edge or corner columns. Together with the estimated ultimate axial load, the suitability of the assumed column size is checked using moment:load charts. Reinforcement quantities may be estimated too.

For beam-and-slab construction, the moment derivation charts work by:

- Using the beam span on the horizontal axis,
- Reading up to the line representing the appropriate ultimate applied uniformly distributed load (uaudl) on the beam then
- Reading across to the vertical axis to estimate the 1st order moment, M, in an assumed size of square column.

Similarly, for flat slab construction the moment derivation charts work by:

- Using the relevant slab span
- Reading up to the line representing the appropriate imposed load (IL) on the slab and
- Reading across to the vertical axis to estimate the 1st order moment, M, in an assumed size of square column. Again, this moment is used with the estimated or calculated ultimate axial load to check the adequacy of the assumed column size and for probable reinforcement density by referring to a range of moment:load charts (see Figures 3.38 – 3.44)

This derived 1st order moment, M, is used with the estimated or calculated ultimate axial load, N_{Ed}, to check the adequacy of the assumed column size and for probable reinforcement density by referring to a parallel range of moment:load charts. This process is repeated until a suitable column size and reinforcement density is found (see Figures 3.37 – 3.44) The charts are arranged in size order so that the moment derivation and the moment:load charts for edge columns in beam-and-slab construction are adjacent; those for corner columns are also adjacent. Flat slab construction charts follow those for beam-and-slab construction.

The charts and data relate to square columns. However, these sizes can be used, with caution, to derive the sizes of rectangular columns, with equal area and aspect ratios up to 2.0, and of circular columns of at least the same cross-sectional area.

The moment derivation charts for edge and corner columns are based on a standard storey height of 3.75 m. Adjustment factors for other storey heights are given. Separate curves have been plotted for 'columns above and below' and 'columns below only'. For corner columns and biaxial bending, by assuming ratios of moments in two directions, estimates can be undertaken using moment in one direction only. Section 7.1.5 gives background to the method and charts used.

The moments and column sizes derived, particularly for perimeter columns, should be regarded as estimates only, until calculations can be made to prove their validity. Sizes derived from the charts and data should be checked for compatibility with slabs (e.g. punching shear in flat slabs) and beams (e.g. widths and end bearings). The moment in the top of a perimeter column joined to a concrete roof can prove critical in final design. Unless special measures are taken (e.g. by providing, effectively, a pin joint, designing as a beam with low axial load), it is suggested that this single storey load case should be checked at scheme design stage.

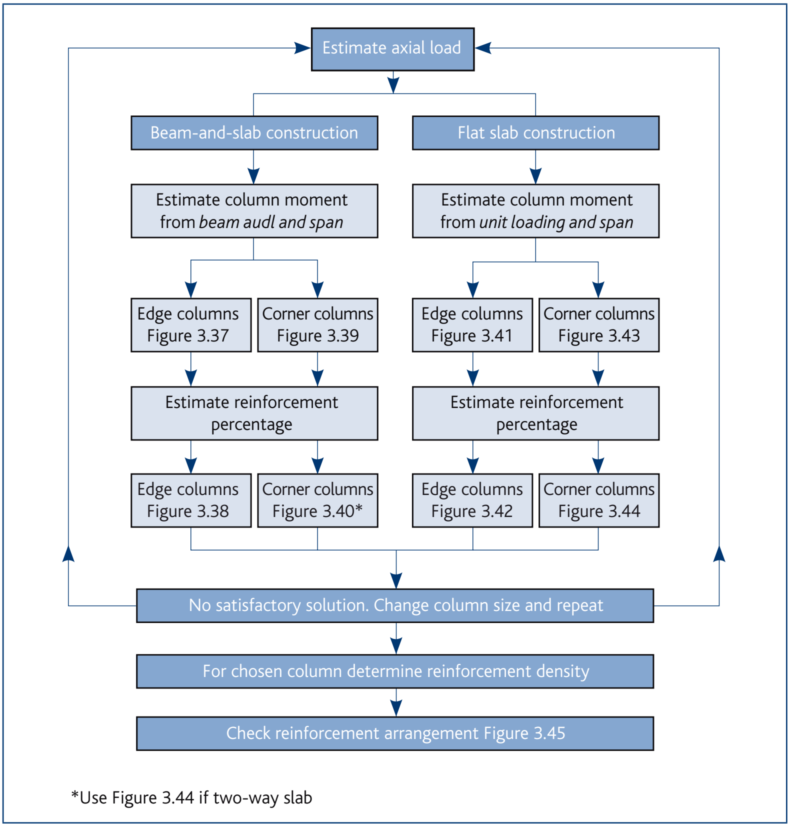


Figure 3.E
Using charts for sizing perimeter columns

3.3.3 Design assumptions

Fire and durability
Fire resistance 1 hour; exposure class XC1; cover to all max[15; ϕ] + Δc_{dev}, but not less than 30 mm.

Concrete
C30/37 (and C50/60 as noted); 25 kN/m^3; 20 mm aggregate.

Reinforcement
Main bars and links: f_{yk} = 500 MPa. Maximum main bar size H32. Minimum reinforcement 4 no. H12s. Link size H8. Reinforcement weights assume standard laps and 3.75 m storey heights with links at appropriate centres. No allowance is made for wastage. For reinforcement quantities, refer to Section 2.2.4.

Other assumptions made are described and discussed in Section 7.

3.3.4 Design notes

As described in Section 7, the charts and data are based on considering square braced columns supporting beam-and-slab construction or solid flat slabs, with a panel aspect ratio of 1.00, carrying a 10 kN/m perimeter load. The column moment derivation charts and data assume economic beam depths or slab thicknesses, which should result in a marginal over-estimatation of column moments. Generally the sizes given should prove conservative but may not be so when fully analysed and designed, or especially when less stiff structures, or very lightweight cladding, is used.

Please note that the 1st order moment, M, used should never be less than M_{min} indicated on the moment:load charts. ($M_{min} = e_i N_{Ed}$ = allowance for imperfections where e_i = max. {20 mm; h/30}).

In beam-and-column construction, beam load and span determine moment in columns. In flat slabs only, where panel aspect ratio is not equal to 1.00, the derived moment should be multiplied by the relevant aspect ratio.

In the moment derivation charts for flat slabs, a superimposed dead load of 1.5 kN/m^2 (for finishes, services, etc.) has been assumed. If the design applied loads vary from this value, the imposed load should be adjusted accordingly.

The moment:axial load curves have been adjusted to allow for biaxial bending with values M_y/M_z as noted below each chart, and for any additional buckling moments (2nd order). Greater storey heights will reduce moment capacity, but applied moment will be smaller.

Please note that for corner columns in two-way slab construction, moments should be derived from the moment derivation chart for corner columns in beam-and-slab construction (Figure 3.39), but this moment should be used on the moment:load charts for corner columns in flat slab construction in Figure 3.44 in Section 3.

Main bars
Feasible bar arrangements for various square column sizes and reinforcement percentages are given In Figure 3.45. These graphs have been prepared on the basis of maximum 300 mm centres of bars or minimum 30 mm gap at laps.

3.3.5 Internal columns

Design assumptions

Design and dimensions – See Section 3.2.3.
Curves have been adjusted to allow for biaxial bending with $M_y/M_z = 1.0$.
Fire resistance – 1 hour.
Exposure class – XC1.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Percentage reinforcement
0.2%
1.0%
2.0%
3.0%
4.0%

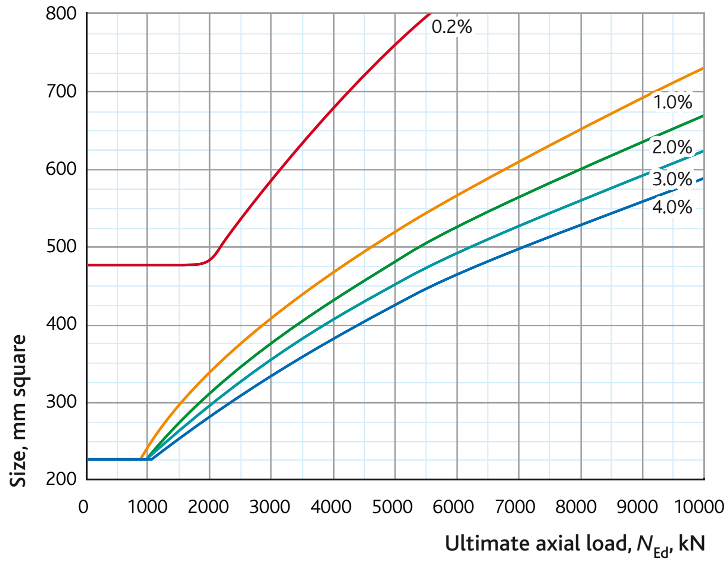


Figure 3.35
Load:size chart for internal columns

Table 3.35
Data for internal columns

Ultimate axial load, N_{Ed}, kN	1000	1500	2000	3000	4000	5000	6000	8000	10000
Size, mm square									
0.2% reinf. C30/37	476	476	480	588	679	759	831		
1.0% reinf. C30/37	250	299	339	408	467	518	565	652	729
2.0% reinf. C30/37	231	275	313	376	432	479	522	598	668
3.0% reinf. C30/37	225	259	294	354	403	448	487	559	621
4.0% reinf. C30/37	225	249	281	338	384	425	462	528	586
Variations: implications on column size of using different grades of concrete									
2.5% reinf. C35/45	225	256	291	349	400	442	482	552	614
2.5% reinf. C40/50	225	247	280	335	383	425	462	529	588
2.5% reinf. C45/55	225	238	270	323	369	409	444	509	565
2.5% reinf. C50/60	225	231	261	312	355	395	430	491	545

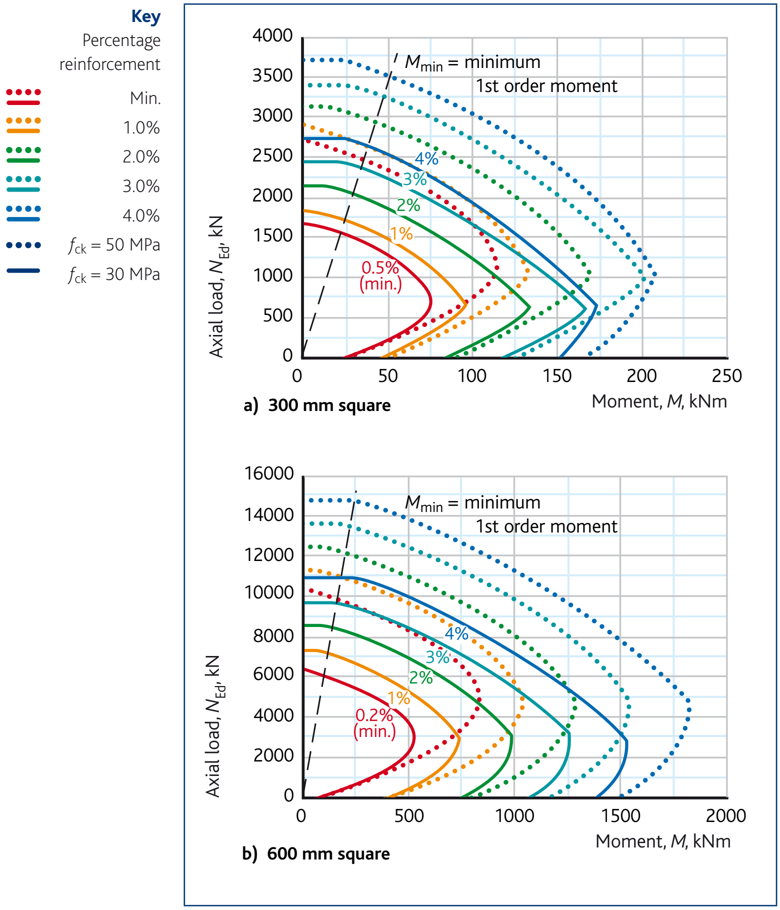


Figure 3.36
Moment:load charts for internal columns

3.3.6 Edge columns in beam-and-slab construction

Design assumptions

Design – See Section 3.2.3.
Curves have been adjusted to allow for biaxial bending with $M_y/M_z = 0.2$.

Storey height – 3.75 m.

Fire resistance – 1 hour.

Exposure class – XC1.

Concrete – C30/37; 25 kN/m^3.

Reinforcement – $f_{yk} = 500$ MPa.

Table 3.36
Adjustments to M_{0Ed} for storey height

Column size, mm	Storey height, m				
	2.5	3.0	3.5	4.0	4.5
225 sq.	31%	18%	6%	–5%	–14%
300 sq.	31%	18%	6%	–5%	–14%
400 sq.	27%	15%	5%	–4%	–11%
500 sq.	13%	8%	3%	–2%	–7%
600 sq.	8%	5%	2%	–1%	–4%

Key
Beam uaudl
50 kN/m; 100 kN/m; 200 kN/m; 400 kN/m
•••• Columns below only; —— Columns above & below

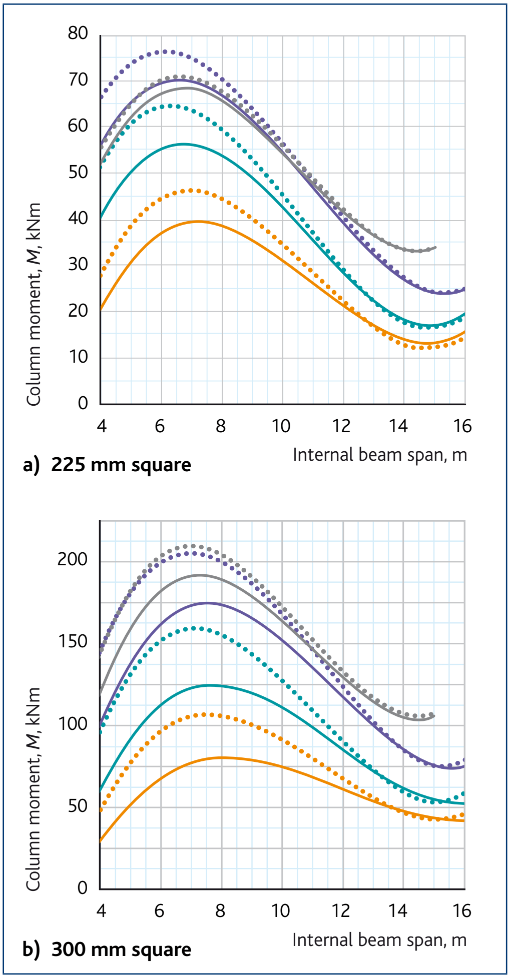


Figure 3.37
Moment derivation charts for edge columns in beam-and-slab construction

Key
Percentage reinforcement
Min.; 1.0%; 2.0%; 3.0%; 4.0%
—— $f_{ck} = 30$ MPa; •••• $f_{ck} = 50$ MPa

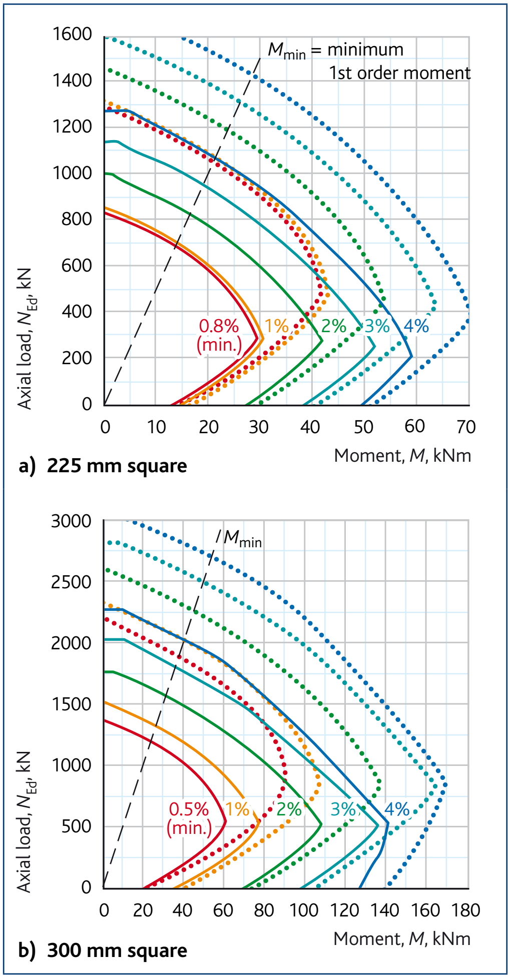


Figure 3.38
Moment:load charts for edge columns in beam-and-slab construction

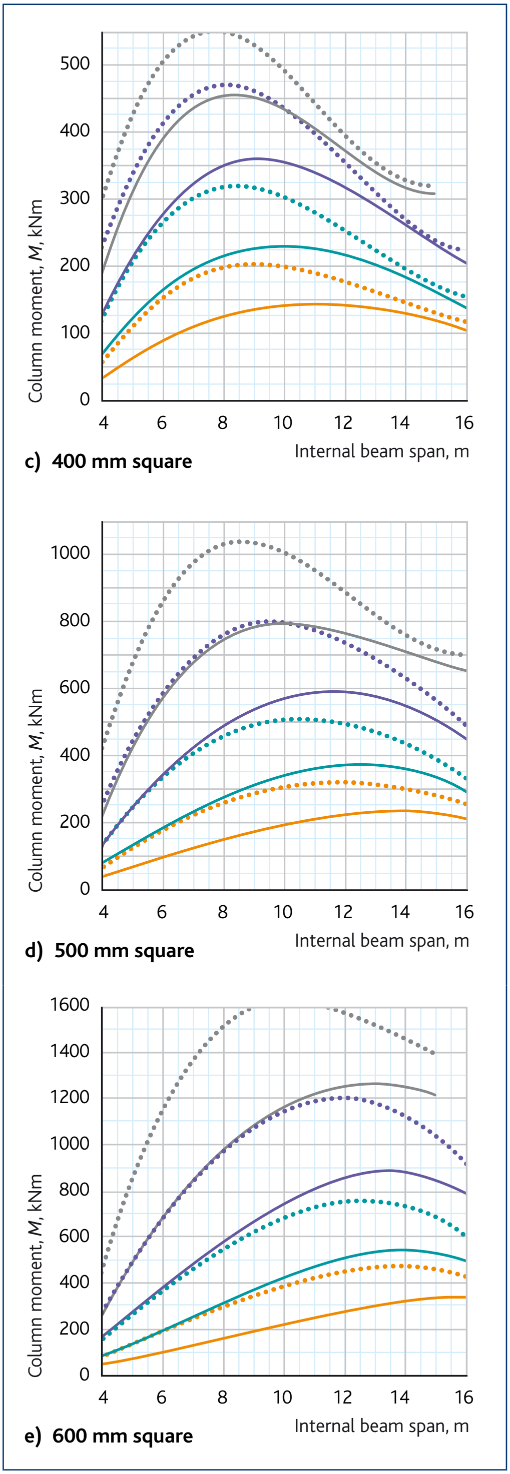


Figure 3.37
(continued)

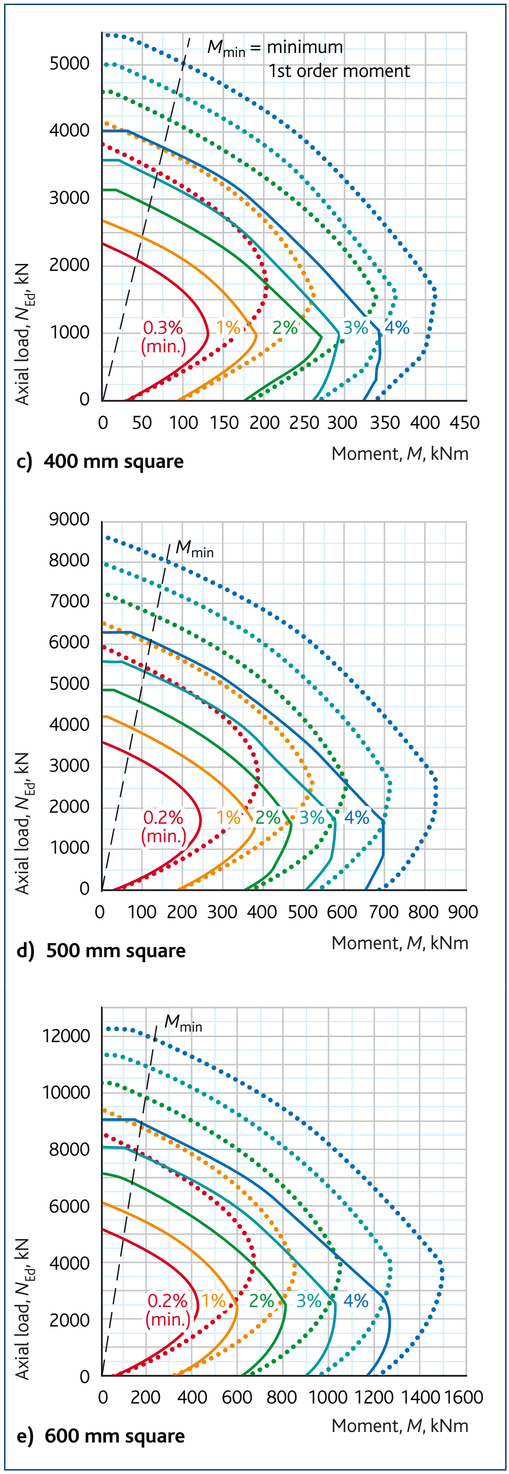


Figure 3.38
(continued)

3.3.7 Corner columns in beam-and-slab construction

Design assumptions

Design – See Section 3.2.3.
Curves have been adjusted to allow for biaxial bending with $M_y/M_z = 0.5$.
Storey height – 3.75 m.
Fire resistance – 1 hour.
Exposure class – XC1.
Concrete – C30/37; 25 kN/m^2.
Reinforcement – $f_{yk} = 500$ MPa.

Table 3.37
Adjustments to M_{0Ed} for storey height

Column size, mm	Storey height, m				
	2.5	3.0	3.5	4.0	4.5
225 sq.	23%	14%	6%	–5%	–14%
300 sq.	20%	12%	5%	–4%	–12%
400 sq.	13%	8%	3%	–3%	–8%
500 sq.	8%	5%	2%	–2%	–5%
600 sq.	4%	2%	1%	–1%	–3%

Key
Beam uaudl
25 kN/m 50 kN/m 100 kN/m 200 kN/m
Columns below only Columns above & below

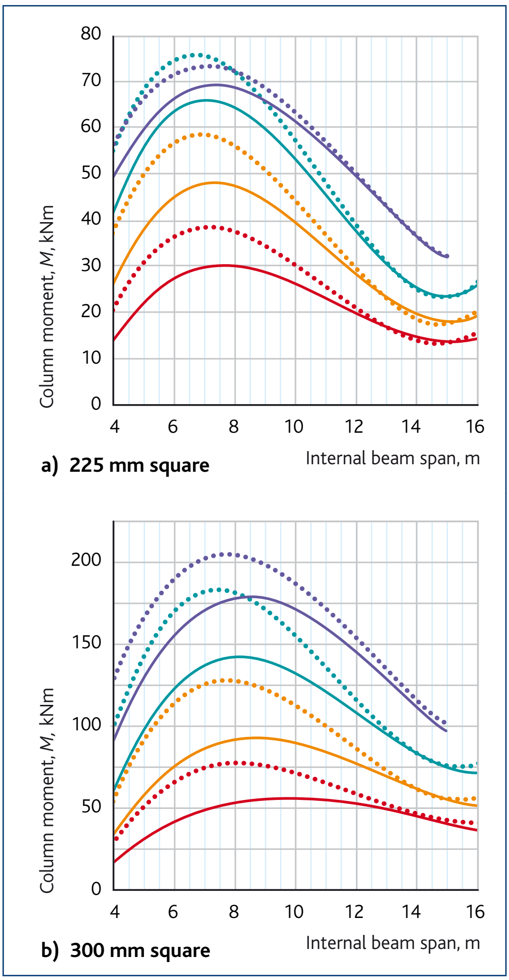


Figure 3.39
Moment derivation charts for corner columns in beam-and-slab construction

Key
Percentage reinforcement
Min. 1.0% 2.0% 3.0% 4.0%
$f_{ck} = 30$ MPa $f_{ck} = 50$ MPa

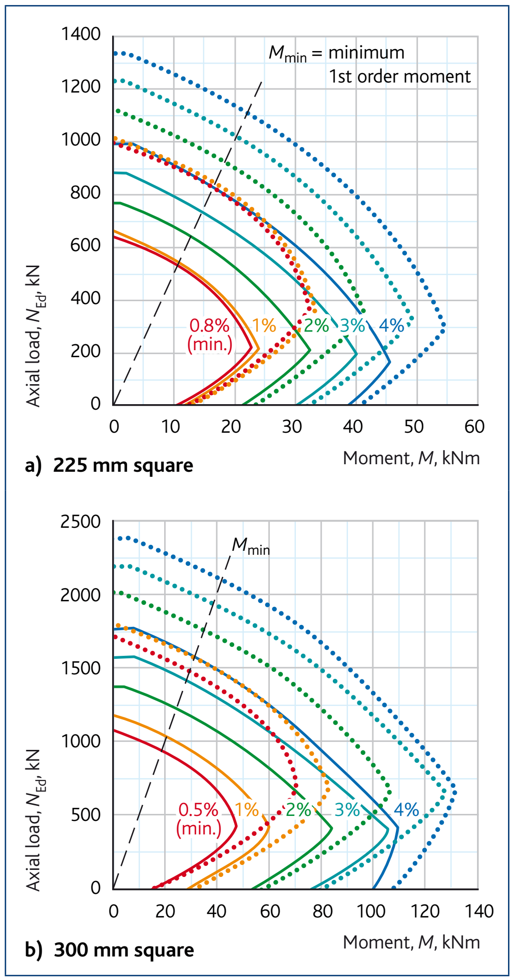
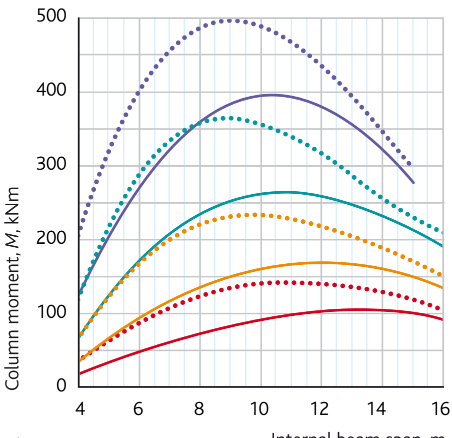
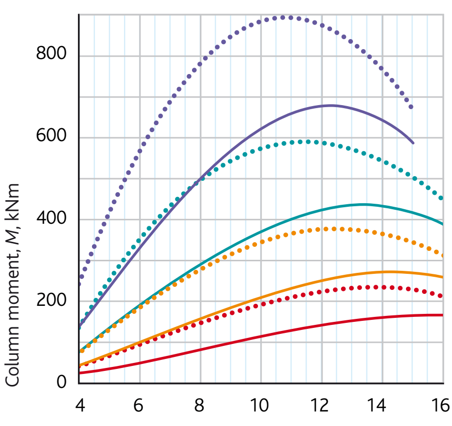


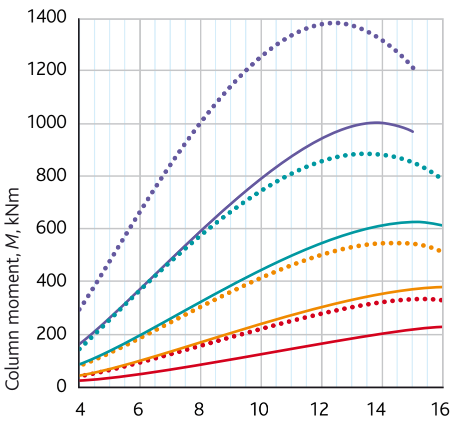
Figure 3.40
Moment:load charts for corner columns in beam-and-slab construction



c) 400 mm square

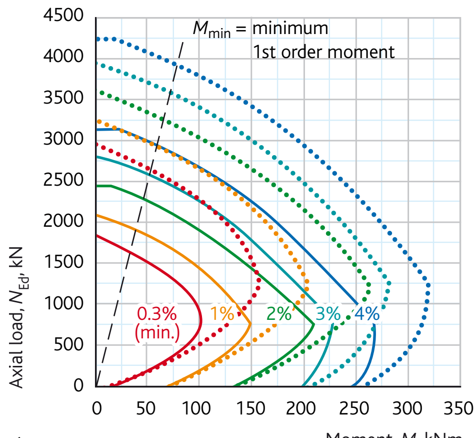


d) 500 mm square

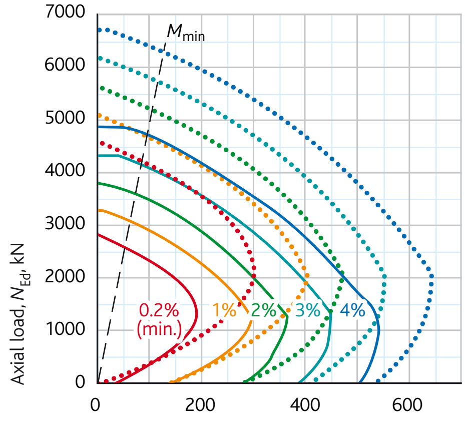


e) 600 mm square

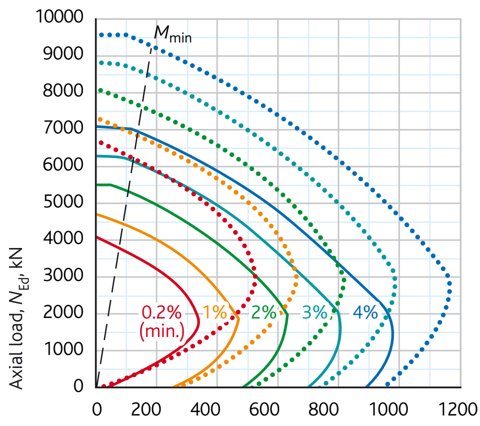
Figure 3.39
(continued)



c) 400 mm square



d) 500 mm square



e) 600 mm square

Figure 3.40
(continued)

3.3.8 Edge columns in flat slab construction

Design assumptions

Design – See Section 3.2.3.
Curves have been adjusted to allow for biaxial bending.
with $M_y/M_z = 0.2$.

Storey height – 3.75 m.

Fire resistance – 1 hour.

Exposure class – XC1.

Concrete – C30/37; 25 kN/m^2.

Reinforcement – $f_{yk} = 500$ MPa.

Table 3.38
Adjustments to M_{0Ed} for storey height

Column size, mm	Storey height, m				
	2.5	3.0	3.5	4.0	4.5
225 sq.	27%	16%	7%	–16%	–24%
300 sq.	25%	15%	6%	–15%	–22%
400 sq.	12%	7%	2%	–7%	–10%
500 sq.	5%	3%	1%	–3%	–5%
600 sq.	0%	0%	0%	0%	0%

Key
Beam uaudl
2.5 kN/m^2 5 kN/m^2 7.5 kN/m^2 10 kN/m^2
Columns below only Columns above & below

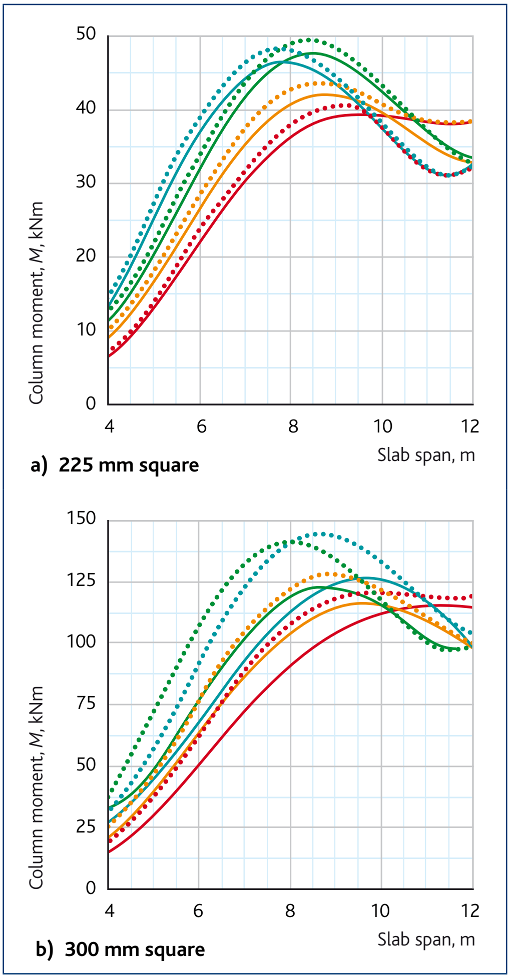


Figure 3.41
Moment derivation charts for edge columns in flat slab construction

Key
Percentage reinforcement
Min. 1.0% 2.0% 3.0% 4.0%
$f_{ck} = 30$ MPa $f_{ck} = 50$ MPa

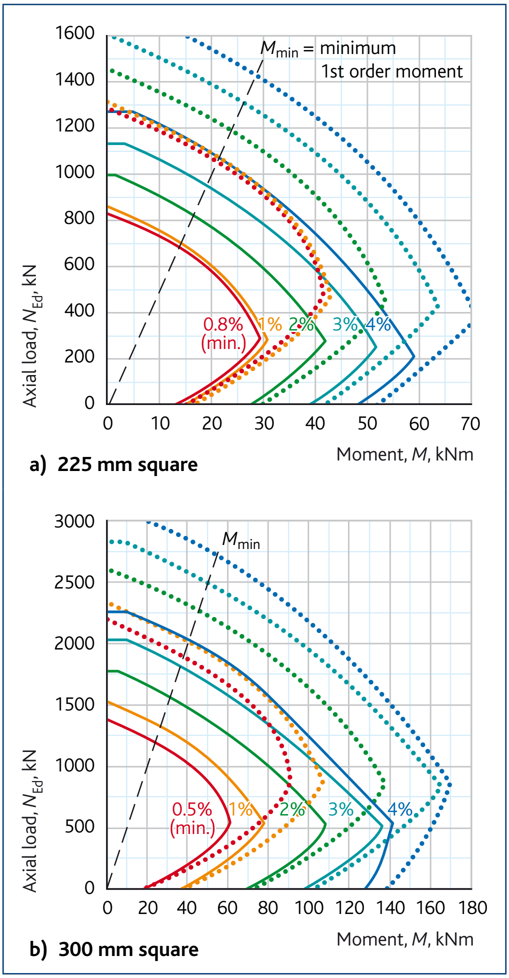
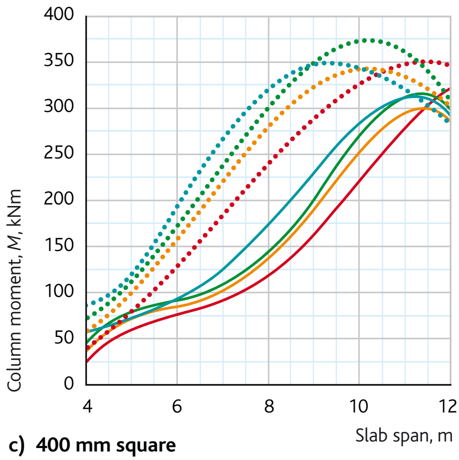
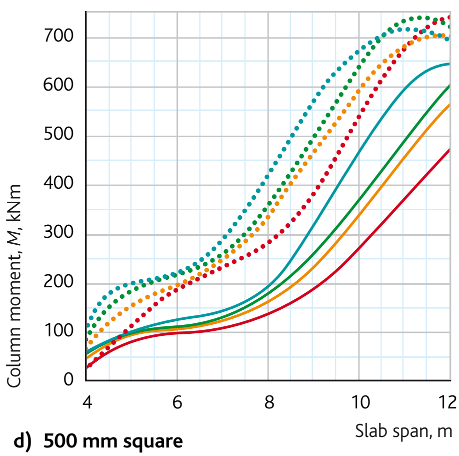


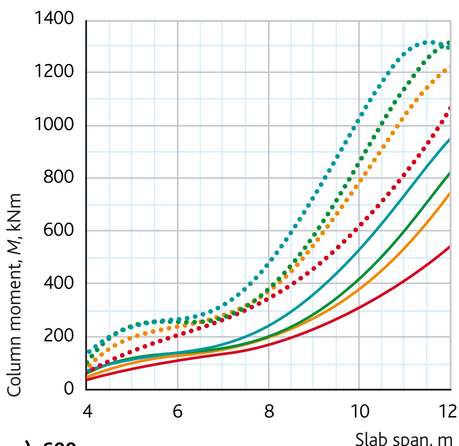
Figure 3.42
Moment:load charts for edge columns in flat slab construction



c) **400 mm square**

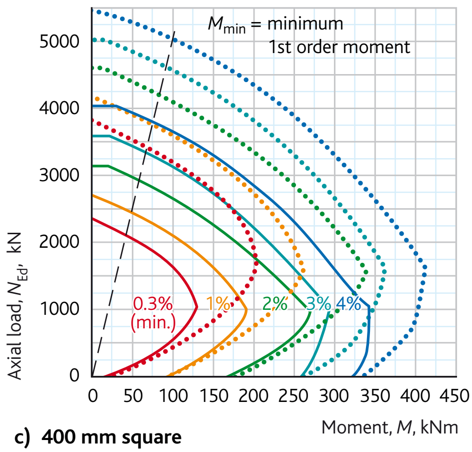


d) **500 mm square**

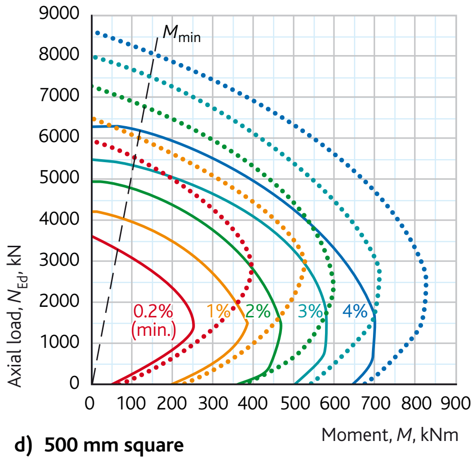


e) **600 mm square**

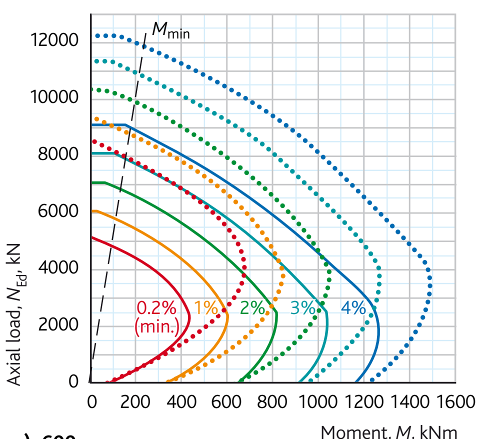
Figure 3.41
(continued)



c) **400 mm square**



d) **500 mm square**



e) **600 mm square**

Figure 3.42
(continued)

3.3.9 Corner columns in flat slab construction

Moment:load charts also used for corner column two-way slab construction

Design assumptions

Design – See Section 3.2.3.

Curves have been adjusted to allow for biaxial bending with $M_y/M_z = 1.0$.

Storey height – 3.75 m.

Fire resistance – 1 hour.

Exposure class – XC1.

Concrete – C30/37; 25 kN/m².

Reinforcement – $f_{yk} = 500$ MPa.

Table 3.39
Adjustments to M_{0Ed} for storey height

Column size, mm	Storey height, m 2.5	3.0	3.5	4.0	4.5
225 sq.	25%	15%	7%	–6%	–15%
300 sq.	10%	6%	2%	–2%	–6%
400 sq.	0%	0%	0%	0%	0%
500 sq.	0%	0%	0%	0%	0%
600 sq.	0%	0%	0%	0%	0%

Key

Beam uaudl

2.5 kN/m² 5 kN/m² 7.5 kN/m² 10 kN/m²

Columns below only Columns above & below

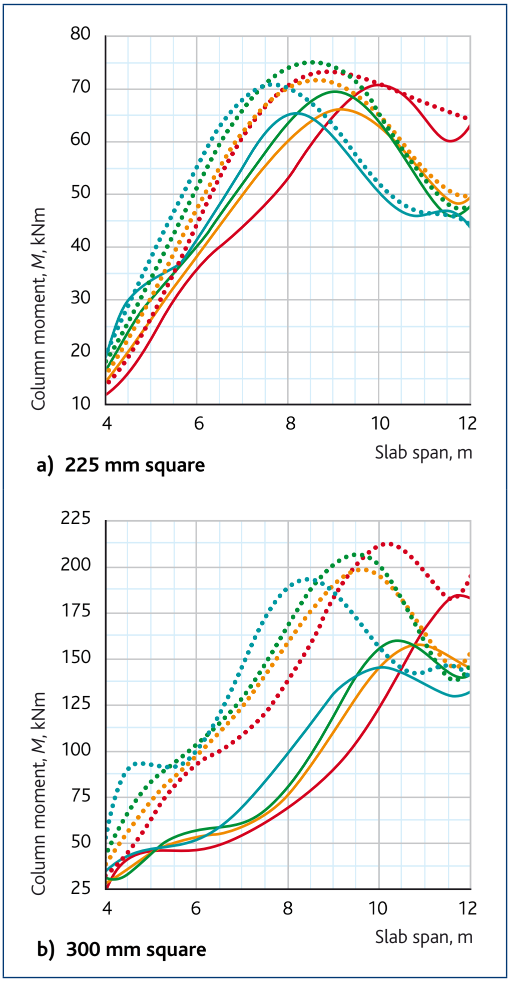


Figure 3.43
Moment derivation charts for corner columns in flat slab construction

Key

Percentage reinforcement

Min. 1.0% 2.0% 3.0% 4.0%

$f_{ck} = 30$ MPa $f_{ck} = 50$ MPa

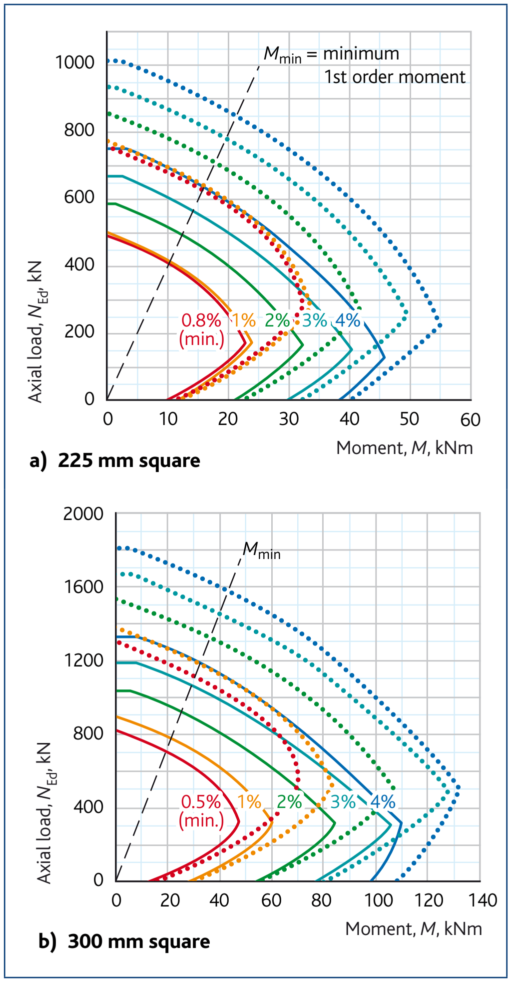


Figure 3.44
Moment:load charts for corner columns in flat slab construction

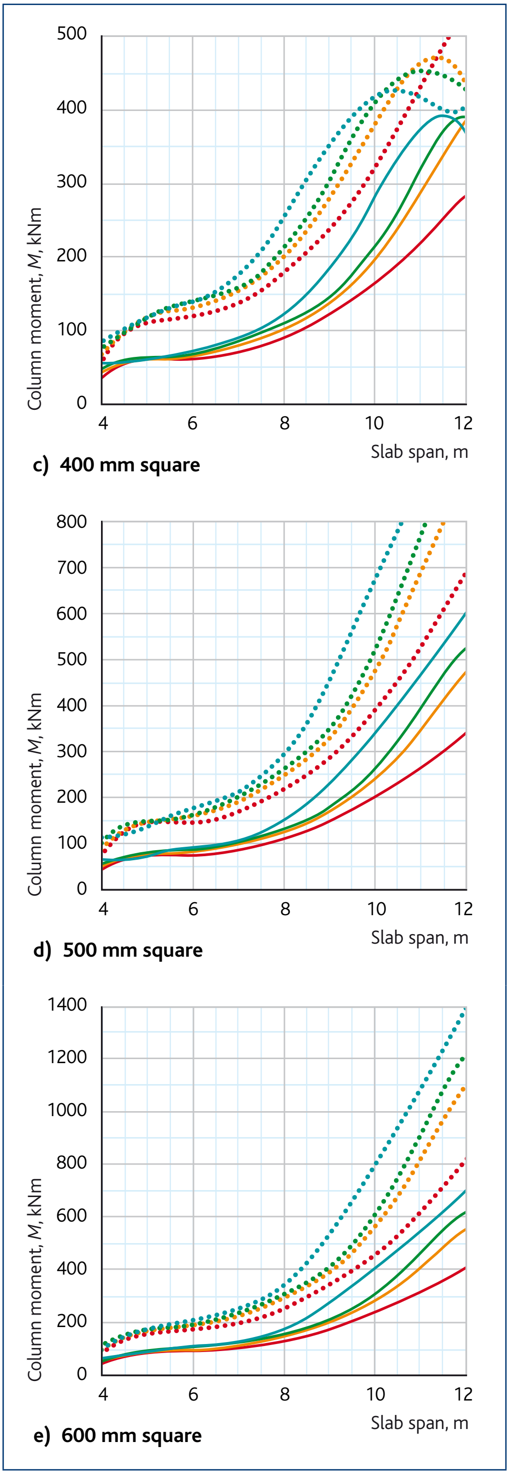


Figure 3.43
(continued)

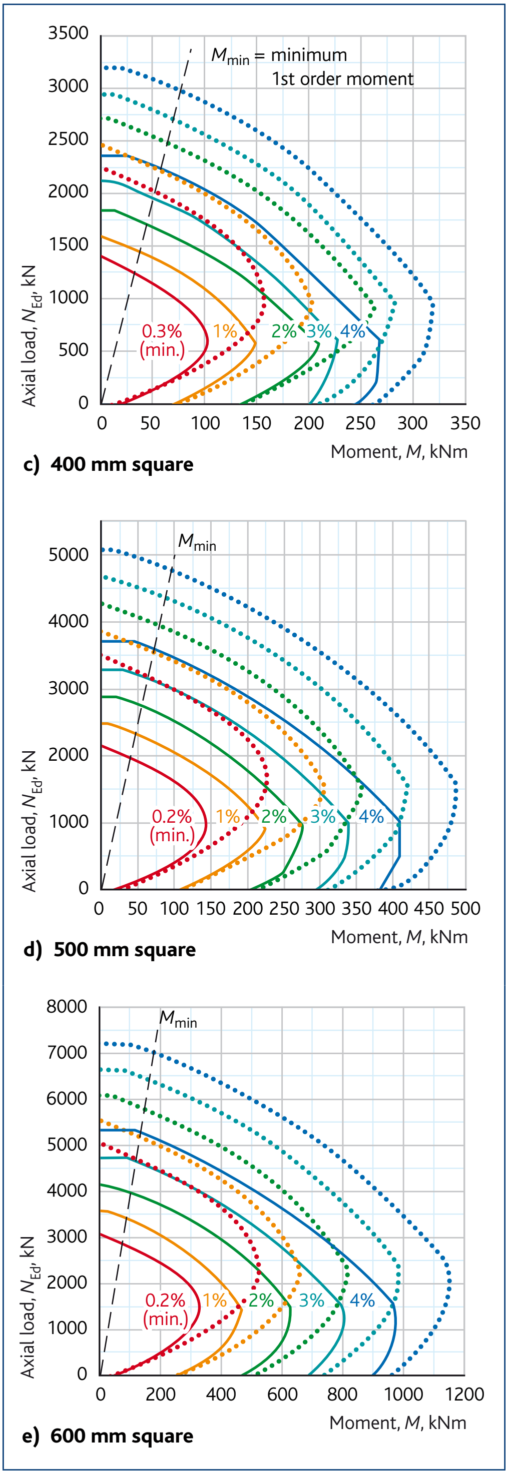


Figure 3.44
(continued)

3.3.10 Column reinforcement

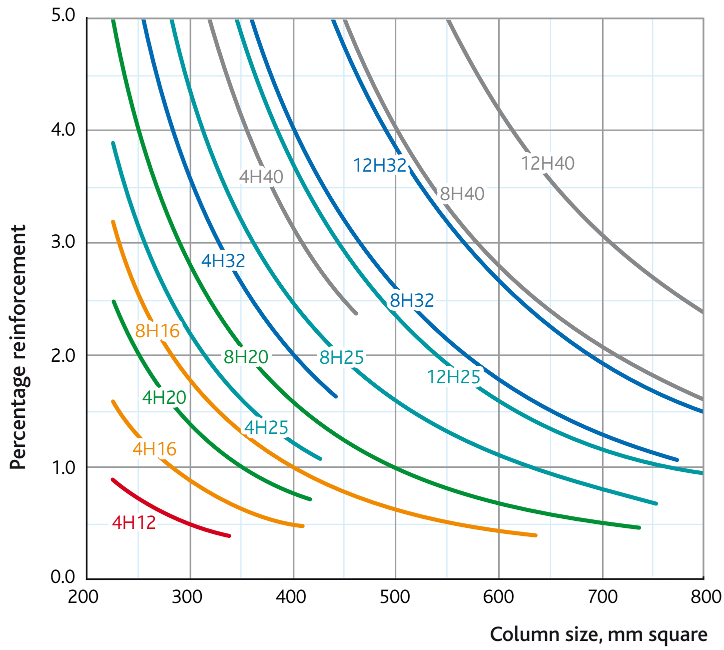


Figure 3.45
Size:percentage reinforcement chart for all columns

Table 3.40
Reinforcement quantities for square columns

Column size, mm square		250	300	350	400	450	500	600	800
Reinforcement	Area, mm^2	Quantity kg/m height (kg/m³)							
4 bars used									
4H12	**452**	7 (104)	7 (77)	8 (61)					
4H16	**804**	10 (156)	10 (108)	10 (81)	12 (74)				
4H20	**1257**	15 (229)	15 (159)	15 (117)	16 (97)				
4H25	**1963**	22 (350)	22 (243)	22 (179)	22 (137)				
4H32	**3217**	37 (582)	37 (404)	37 (297)	37 (228)				
4H40	**5027**			60 (490)	60 (375)	63 (309)			
8 bars used									
8H16	**1608**	18 (275)	18 (190)	18 (142)	20 (120)	20 (98)	21 (81)	22 (60)	
8H20	**2513**	27 (421)	27 (292)	27 (215)	28 (172)	28 (138)	29 (114)	30 (82)	
8H25	**3927**		42 (460)	42 (338)	42 (259)	44 (213)	44 (174)	45 (124)	
8H32	**6434**			71 (575)	71 (440)	72 (356)	73 (290)	74 (204)	
8H40	**10053**					119 (587)	120 (478)	121 (336)	128 (199)
12 bars used									
12H25	**5890**			61 (498)	61 (381)	63 (309)	63 (252)	64 (178)	68 (107)
12H32	**9651**					107 (524)	107 (426)	108 (299)	112 (175)
12H40	**15080**							178 (493)	184 (287)

4 Precast and composite construction

Figure 4.A
Chessington Community College, Surrey. This three-storey college facility features precast concrete frames which were chosen for clear spans to provide flexibility of use now and in the future.
Photo courtesy of Composite Ltd.

4.1 Precast and composite slabs

4.1.1 Using precast and composite slabs

Precast concrete floor units offer many advantages: small, medium and long spans, structural efficiency, economy, versatility, fire resistance, thermal capacity and sound insulation. They readily accept fixings, floor and ceiling finishes and small holes. Handling and stacking is straightforward. Precast concrete flooring provides an immediate safe working platform and can eliminate formwork and propping.

The combination of precast concrete with in-situ concrete (or hybrid concrete construction) harnesses the best of both materials. Structurally, these hybrids can act separately (non-compositely) or together (compositely). Hybrid floors combine all the advantages of speed and quality of precast concrete with the flexibility and versatility of in-situ construction. Each type has implications for overall costs, speed, self-weight, storey height and flexibility in use; some guidance is given with the charts. The relative importance of these factors should be assessed for each particular case.

All prestressed precast concrete flooring systems exhibit a degree of upward camber (in contrast to reinforced construction which exhibits downward deflection) and due allowance should be made for this. Note that specified topping thicknesses relate to topping thickness at mid-span.

4.1.2 The charts and data

The charts and data give overall depths against spans for a range of unfactored imposed loads assuming simply supported spans. Two charts are given on each pair of pages: overall depth versus span for four increments of imposed load on the left page, and characteristic imposed load versus span for various floor depths on the right page. The former is the same format as for the in-situ and post-tensioned sections. The latter follows the convention of the precast industry. An allowance of 1.5 kN/m^2 has been made for superimposed dead loads (finishes, services, etc.). The range of spans may be extended if this allowance is reduced.

The sizes, spans and weights quoted in the charts and data were derived from design spreadsheets to Eurocode 2. The span/load capacities and self-weights of units vary between manufacturers and are subject to development and change. For instance, manufacturers may propose values of transmission length L_{pt} and L_{bpc} based on tests (see *Precast Eurocode 2: Worked examples* [11]). So the user should consult manufacturers and their current literature. The thicknesses given in the tables are measured overall, whether they include the structural toppings or not.

The designer must ensure that adequate robustness is provided by, for example, the provision of effective ties. Connections are key to the integrity of precast frames and the use of precast elements. It is therefore vital to make adequate provision for joints and bearings to transfer forces. There are many types of connections and the reader should refer to specialist literature on precast concrete framed structures [11–15].

The design of bearings is subject to Cl. 10.9.5 of BS EN 1992–1–1. For dry (i.e. no) bedding, the average bearing stress should not exceed $0.3f_{cd}$. Where bedding is used, the design strength of the bedding material should not be exceeded. In determining nominal bearing lengths, allowances must be made for tolerances and ineffective lengths (e.g. to allow for spalling at edges). This is illustrated in Figure 4.B. Typical nominal bearing lengths for slabs are given in Table 4.A. Hollowcore units are usually laid direct onto precast concrete beams, double-tees are laid onto neoprene or felt pads, and solid composite and lattice girder slabs, particularly those with a width of 2.4 m, are laid onto cement-sand bedding.

4.1.3 Design assumptions

The charts and data are based on units designed to BS EN 1992–1–1[2], generally using high-strength concretes and high tensile strand or wire prestressing steel to BS EN 10138[16] or high tensile steel to BS 4449[17] or BS 4483[18]. The precast units are assumed to have attained the design strength f_{ck} at the time of installation. For composite construction, the in-situ topping is also assumed to have attained its design strength before being subject to design imposed loads. In-situ concrete in composite sections is assumed to attain an initial cylinder strength, $f_{ck,i}$, before being depropped (see Section 7.2.1). The precise properties of the units are subject to the manufacturer's design.

The self-weight of a range of precast floor units is given in Table 8.6. As explained in Section 8.1.2 the values of ψ_2 used in the precast slab charts were:

for IL = 2.5 kN/m^2, ψ_2 = 0.3; for IL = 5.0 kN/m^2, ψ_2 = 0.6;
for IL = 7.5 kN/m^2, ψ_2 = 0.6; and for IL = 10.0 kN/m^2, ψ_2 = 0.8.

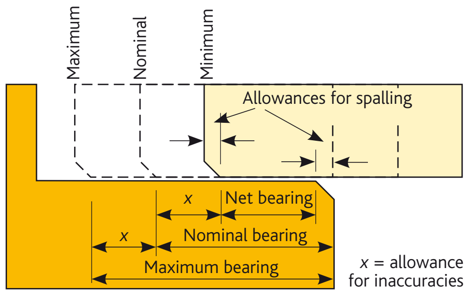


Figure 4.B
Bearing lengths

Table 4.A
Typical nominal bearings lengths for different floor types (mm)

Floor type	On concrete support	On steel support	On masonry support
Solid composite slabs	75	75	100
Lattice girder slabs	65	55	60
Hollowcore slabs	75	75	100
Double-tees	150	75	–
Spans ≥ 12.0 m[a]	150	100	140

Key
a When tied through, bearings may be reduced, subject to manufacturer's recommendation

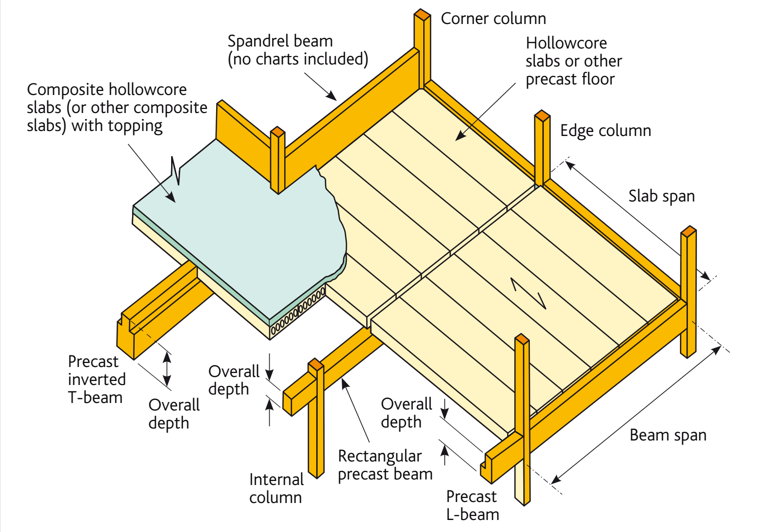


Figure 4.C
Precast concrete construction elements and definition of depths and spans

4.1.4 Chlorides, prestressed units and car parks

Table NA.4 of the UK NA to BS EN 1992–1–1[2] requires any prestressing steel within concrete of exposure classes XD1, XD2, XD3, XS1, XS2 and XS3 to be in an area of decompression under frequent load combinations. This 'decompression' requirement stipulates that all parts of the bonded tendons or duct lie at least 25 mm within concrete in compression.

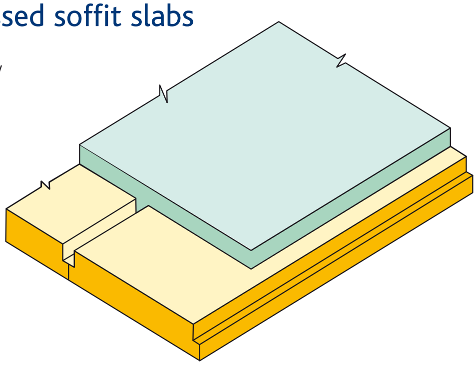
According to BS 8500-1: 2006[4], car park decks are generally taken to be exposure class XD3. As explained in Section 11 of *How to design concrete building structures using Eurocode 2*[19], 'Car park decks, ramps, splash zones and external areas subject to freezing and deicing salts' are taken to be subject to primarily XD3 but also to XF4. The soffits of car park slabs are not cited specifically.

However, there is growing acceptance that car parks are specialist structures. Generally, they are well drained, well ventilated and de-icing salts are not applied directly. In such cases, the following exposure classes are recommended in a new publication from The Concrete Centre[20]:

- Top surfaces:
  - generally XD1
  - entry levels, XD3
  - where exposed to freezing, XF2 and XD1
  - where exposed to freezing and deicing salts, XF4 and XD3
- Soffits and vertical elements: XC3/4.

Therefore, apart from coastal locations where exposure class XS1 (airborne chlorides originating from salt water) should be applied, soffits may be regarded as being `not subject to chlorides'. So decompression is not considered to be an issue for prestressing steel at the bottom of precast units, and the charts and data may be used directly. However, where exposure class XS1 is applicable to the soffits of precast prestressed slabs, depths may be estimated by multiplying the maximum span data given in the charts and data by approximately 0.85. Nonetheless, designers of car parks in coastal locations who intend to use precast prestressed units are advised to consult specialist literature or to contact suppliers to confirm suitability and economic depths.

4.1.5 Composite solid prestressed soffit slabs



Solid prestressed slabs act compositely with a structural topping (generally grade C30/37 with a light fabric) to create a robust composite floor. The units, usually 600 mm, 1200 mm or 2400 wide, act as fully participating formwork, which is usually propped during construction.

The slab depths given in the charts include the topping.

Advantages/disadvantages

These robust slabs are quick to construct, providing a structurally efficient floor that requires no formwork. But propping is usually required and the spans and capacities are limited.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes. All propped at mid-span.
Fire and durability – Fire resistance 1 hour; exposure class XC1. For car parks see Section 4.1.4.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Ultimate loads assume a reaction factor of 1.0 to internal supports and 0.5 to end supports (see Section 8.3.2).
Concrete – Precast C45/55. Topping 50 mm thick (100 mm for 250 mm depth), C30/37 ($f_{ck,i}$ = 25 MPa), 25 kN/m^3, 20 mm gravel aggregate. Installation at 28 days. Imposed loading 28 days after topping cast. E_{cm} = 36,300 MPa.
Reinforcement – Strand, f_{pk} = 1770 N/mm^2 stressed to 70%. c_{nom} = 20 mm (for indoor exposure).

Key
Characteristic imposed load (IL)
- 2.5 kN/m^2
- 5.0 kN/m^2
- 7.5 kN/m^2
- 10.0 kN/m^2

Single span
Propped

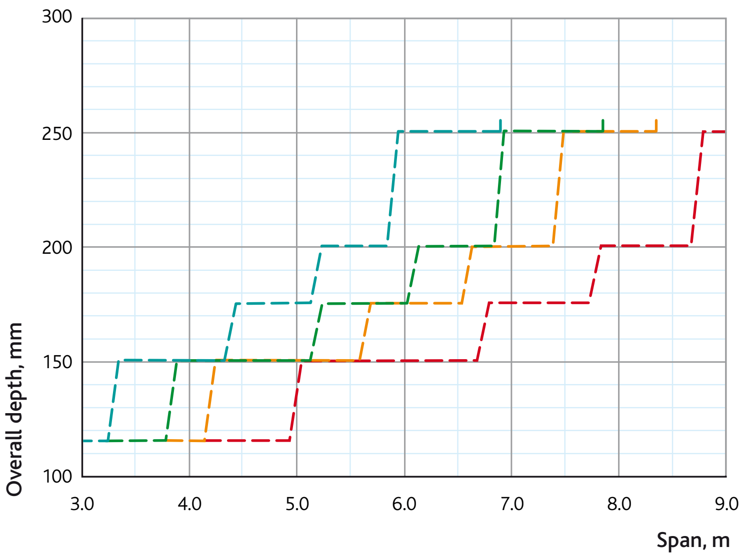


Figure 4.1a
Span:depth chart for composite solid prestressed soffit slabs

Table 4.1a
Data for composite solid prestressed soffit slabs

SINGLE span, m	3.0	4.0	5.0	6.0	7.0	8.0	9.0
Overall depth, mm, propped							
IL = 2.5 kN/m^2	115	115	150	150	175	200	250
IL = 5.0 kN/m^2	115	115	150	175	200	250	
IL = 7.5 kN/m^2	150	150	150	175	250		
IL = 10.0 kN/m^2	150	150	175	250			
Ultimate load to supporting beams, internal (end), kN/m							
IL = 2.5 kN/m^2	28 (14)	37 (18)	52 (26)	62 (31)	78 (39)	95 (48)	121 (60)
IL = 5.0 kN/m^2	39 (19)	52 (26)	70 (35)	89 (45)	109 (55)	138 (69)	
IL = 7.5 kN/m^2	53 (27)	71 (36)	89 (45)	112 (56)	147 (73)		
IL = 10.0 kN/m^2	66 (33)	88 (44)	115 (57)	153 (76)			

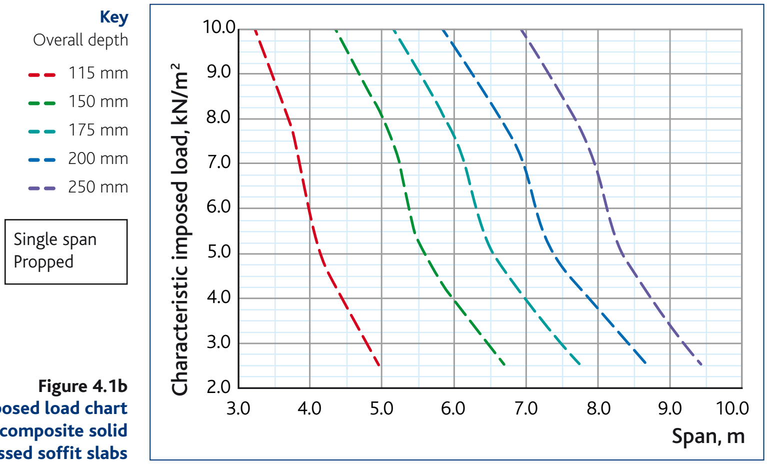
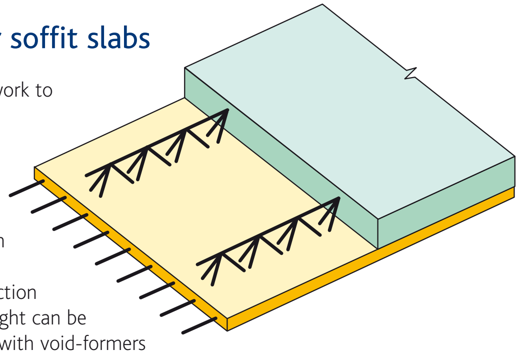


Figure 4.1b
Span:imposed load chart for composite solid prestressed soffit slabs

Table 4.1b
Data for for composite solid prestressed soffit slabs

Imposed load (IL), kN/m^2, and ψ_2	2.5 $\psi_2 = 0.3$	5.0 $\psi_2 = 0.6$	7.5 $\psi_2 = 0.6$	10.0 $\psi_2 = 0.8$
Maximum span, m, propped				
Overall depth = 115 mm	4.95	4.15	3.80	3.25
Overall depth = 150 mm	6.70	5.60	5.15	4.35
Overall depth = 175 mm	7.75	6.55	6.05	5.15
Overall depth = 200 mm	8.70	7.40	6.85	5.85
Overall depth = 250 mm	9.45	8.35	7.85	6.95

4.1.6 Composite lattice girder soffit slabs



Precast units act as permanent formwork to high-capacity, composite floor slabs.

The units are usually 2.4 m wide and precast to include most, if not all, of the bottom reinforcement required. Top reinforcement is fixed on site and cast into the in-situ topping. The lattice girders give the precast section strength during construction. Self-weight can be reduced by having the units supplied with void-formers bonded to the upper surface.

Advantages/disadvantages

These robust slabs are quick to construct, providing a safe working platform that requires little or no formwork. The soffit is of good quality. Continuity is commonly achieved and 2-way action is feasible. However propping and craneage is usually required.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes. All propped at mid-span.
Multiple spans – Units are erected in single spans and propped at mid-span in the temporary condition. Only two-span data is given here; overall depths for three spans or more may be up to 25 mm shallower (see data in Section 7.2.1).
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Ultimate loads assume a reaction factor of 0.5 at end supports and 1.0 at internal supports (see Section 8.3.2).
Concrete – Precast C45/55. In-situ C30/37 ($f_{ck,i}$ = 25 MPa), 25 kN/m^3, both use 20 mm gravel aggregate. Thickness of precast section = 75 mm or 100 mm. Precast installation at 28 days. Imposed loading at 28 days after topping cast. E_{cm} = 36,300 MPa.
Reinforcement – f_{yk} = 500 N/mm^2. c_{nom} = 20 mm for indoor exposure. 4 lattices per 2.4 m wide unit. Each lattice has 1 bar at top, 2 bars at bottom. To comply with deflection criteria, service stress, σ_s, may have been reduced.

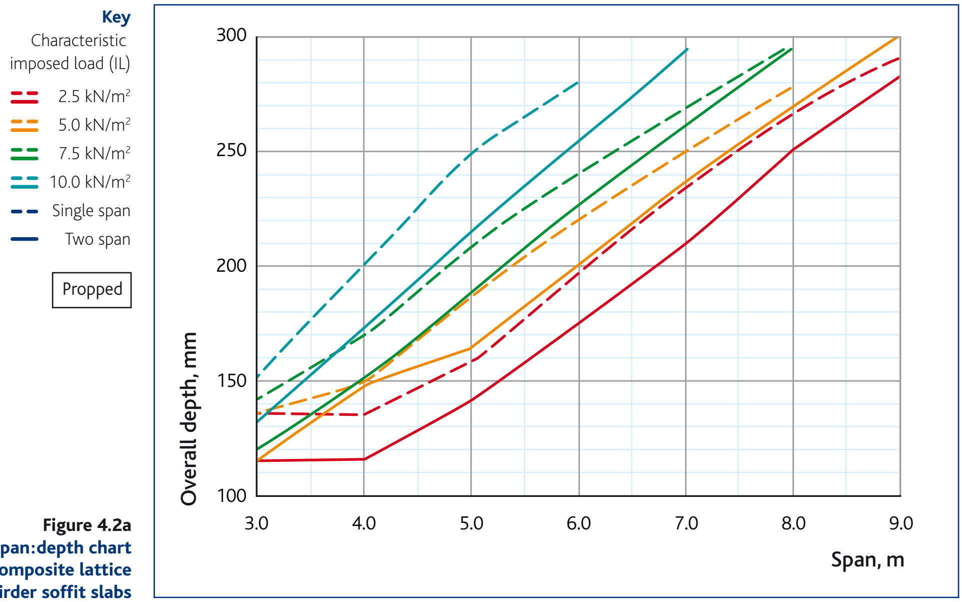


Figure 4.2a
Span:depth chart for composite lattice girder soffit slabs

Table 4.2a
Data for composite lattice girder soffit slab

SINGLE span, m	3.0	4.0	5.0	6.0	7.0	8.0	9.0
Overall depth, mm, propped							
IL = 2.5 kN/m²	135	135	158	197	234	267	291
IL = 5.0 kN/m²	135	149	186	220	250	277	
IL = 7.5 kN/m²	142	170	209	241	269	296	
IL = 10.0 kN/m²	153	203	250	280			
Ultimate load to supporting beams, internal (end), kN/m							
IL = 2.5 kN/m²	n/a (15)	n/a (20)	n/a (27)	n/a (36)	n/a (46)	n/a (56)	n/a (67)
IL = 5.0 kN/m²	n/a (21)	n/a (29)	n/a (38)	n/a (49)	n/a (61)	n/a (73)	
IL = 7.5 kN/m²	n/a (27)	n/a (37)	n/a (50)	n/a (62)	n/a (76)	n/a (90)	
IL = 10.0 kN/m²	n/a (33)	n/a (47)	n/a (62)	n/a (77)			

TWO span, m	3.0	4.0	5.0	6.0	7.0	8.0	9.0
Overall depth, mm, propped							
IL = 2.5 kN/m²	115	115	140	174	209	250	282
IL = 5.0 kN/m²	115	147	163	200	236	269	300
IL = 7.5 kN/m²	119	149	187	225	260	294	
IL = 10.0 kN/m²	131	171	213	254	293		
Ultimate load to supporting beams, internal (end), kN/m							
IL = 2.5 kN/m²	28 (14)	37 (19)	51 (26)	67 (34)	86 (43)	108 (54)	130 (65)
IL = 5.0 kN/m²	39 (20)	56 (28)	73 (37)	94 (47)	118 (59)	143 (72)	169 (85)
IL = 7.5 kN/m²	51 (26)	72 (36)	95 (48)	121 (61)	149 (75)	179 (90)	
IL = 10.0 kN/m²	63 (32)	89 (45)	118 (59)	149 (75)	183 (92)		

Key
Overall depth
115 mm
135 mm
150 mm
200 mm
250 mm
300 mm
Single span
Two span

Propped

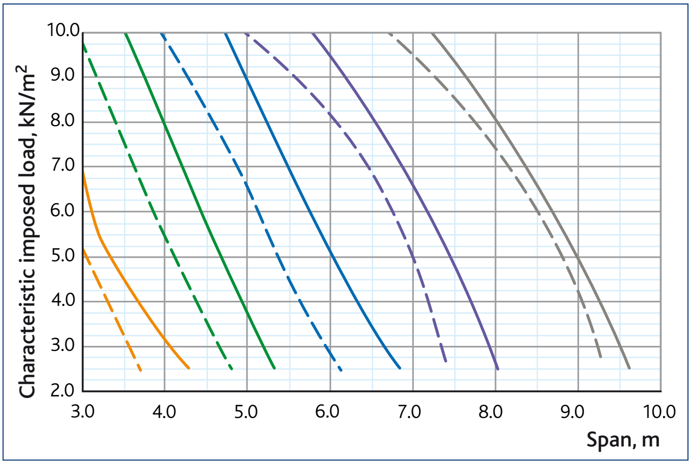
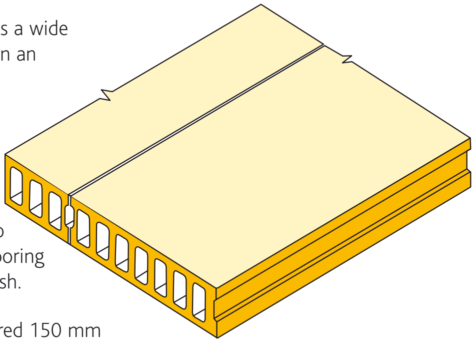


Figure 4.2b
Span:imposed load chart for composite lattice girder soffit slabs

Table 4.2b
Data for composite lattice girder slabs

Imposed load (IL), kN/m², and ψ_2	2.5 $\psi_2 = 0.3$	5.0 $\psi_2 = 0.6$	7.5 $\psi_2 = 0.6$	10.0 $\psi_2 = 0.8$
Maximum span, single span (two span), m, propped				
Overall depth = 115 mm	n/a (4.25)	n/a (3.30)	n/a (2.90)	n/a (2.60)
Overall depth = 135 mm	3.70 (n/a)	3.00 (n/a)	2.60 (n/a)	2.25 (n/a)
Overall depth = 150 mm	4.80 (5.30)	4.10 (4.65)	3.50 (4.05)	2.95 (3.50)
Overall depth = 200 mm	6.10 (6.80)	5.35 (6.00)	4.75 (5.35)	3.95 (4.70)
Overall depth = 250 mm	7.45 (8.00)	7.00 (7.40)	6.30 (6.70)	5.00 (5.90)
Overall depth = 300 mm	9.35 (9.60)	8.75 (9.00)	7.90 (8.20)	6.70 (7.20)

4.1.7 Precast hollowcore slabs, no topping



Hollowcore floor units are economic across a wide range of spans and loadings and are used in an extensive range of buildings.

In the UK available depths range in increments from 110 mm to 450 mm. Span:load capacities vary slightly between manufacturers; but widths are generally 1200 mm. The top is designed to receive a levelling screed or appropriate flooring system. The soffit provides a utilitarian finish.

Where enhanced sound resistance is required 150 mm deep units with special heavy cross-sections are available.

Advantages/disadvantages

These structurally efficient slabs are quick to construct, and require little or no formwork or propping. They can provide a range of spans with high load capacities and flat utilitarian soffits that can be used for passive cooling . However, craneage is usually required and allowance has to be made for camber. In very long spans, shrinkage and axial creep can give problems.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes.
Fire and durability – Fire resistance 1 hour; exposure class XC1. For car parks see Section 4.1.4.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Ultimate loads assume a reaction factor of 0.5 at end supports and 1.0 at internal supports (see Section 8.3.2).
Concrete – C45/55; 25 kN/m^3; 20 mm gravel aggregate. Precast installation at 28 days. Imposed loading at 28 days after topping cast. E_{cm} = 36,300 MPa.
Reinforcement – Strand and/or wire, f_{pk} = 1770 N/mm^2 stressed to 70%. c_{nom} = 20 mm to wire, 30 mm to strand (for indoor exposure).

Key
Characteristic imposed load (IL)
2.5 kN/m^2
5.0 kN/m^2
7.5 kN/m^2
10.0 kN/m^2

Single span
Unpropped

Note
For 150 mm heavy, refer to Figure 4.3b and Table 4.3b

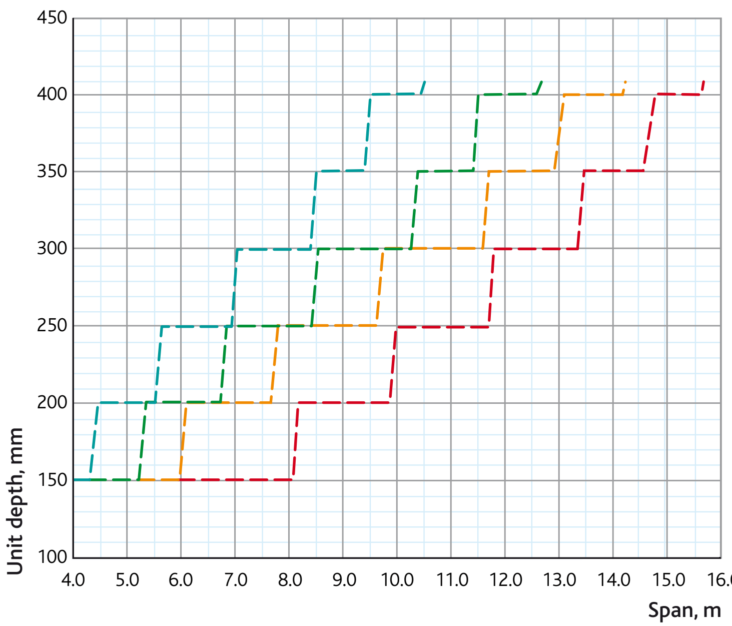


Figure 4.3a
Span:depth chart for precast hollowcore slabs, no topping

Table 4.3a
Data for precast hollowcore slabs, no topping

SINGLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Unit depth, mm, (unpropped)									
IL = 2.5 kN/m^2	150	150	150	200	250	250	300	300	350
IL = 5.0 kN/m^2	150	200	250	250	300	300	350	400	
IL = 7.5 kN/m^2	200	250	250	300	300	350	400		
IL = 10.0 kN/m^2	250	300	300	350	400				
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	55 (27)	64 (32)	73 (37)	89 (44)	105 (53)	116 (58)	135 (68)	146 (73)	168 (84)
IL = 5.0 kN/m^2	77 (39)	95 (48)	114 (57)	128 (64)	150 (75)	165 (83)	189 (95)	215(107)	
IL = 7.5 kN/m^2	104 (52)	126 (63)	144 (72)	169 (84)	188 (94)	215(107)	243(122)		
IL = 10.0 kN/m^2	134 (67)	162 (81)	185 (92)	215(108)	247(124)				

Note
400 mm units supporting IL = 2.5 kN/m^2 will span 15.65 m and impart an ultimate load to supporting beams of 200 kN/m to internal beams (and 100 kN/m to end beams)

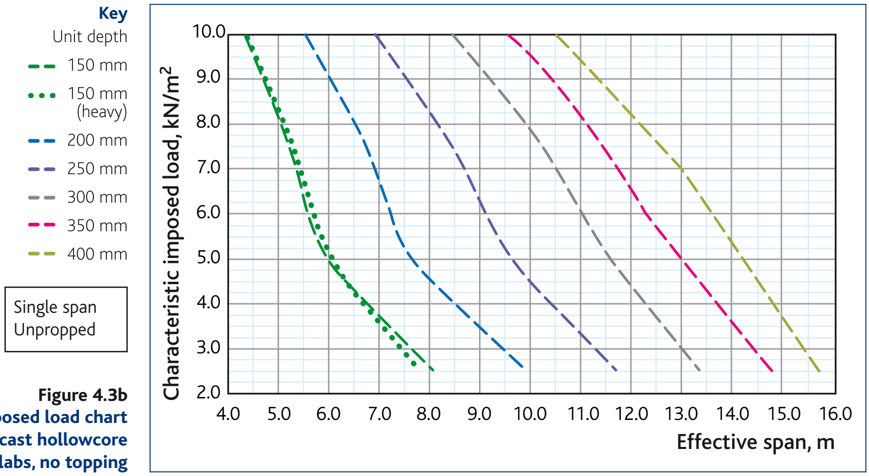
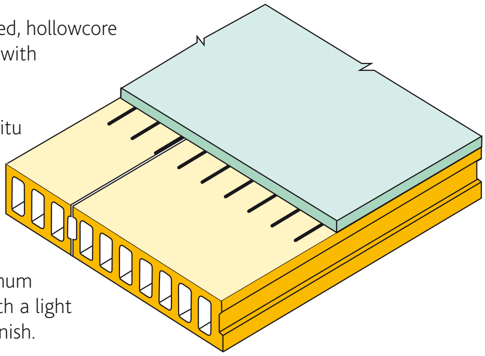


Figure 4.3b
Span:imposed load chart for precast hollowcore slabs, no topping

Table 4.3b
Data or precast hollowcore slabs, no topping

Imposed load (IL), kN/m^2, and ψ_2	2.5 $\psi_2 = 0.3$	5.0 $\psi_2 = 0.6$	7.5 $\psi_2 = 0.6$	10.0 $\psi_2 = 0.8$
Maximum span, m, unpropped				
Unit depth = 150 mm	8.10	6.00	5.25	4.35
Unit depth = 200 mm	9.90	7.70	6.75	5.55
Unit depth = 250 mm	11.70	9.65	8.45	6.95
Unit depth = 300 mm	13.35	11.60	10.25	8.45
Unit depth = 350 mm	14.70	12.95	11.35	9.40
Unit depth = 400 mm	15.65	14.25	12.70	10.50
Heavy hollowcore units (re. Building Regulations Part E, Sound)				
Unit depth = 150 mm	7.80	6.10	5.30	4.40

4.1.8 Composite hollowcore slabs, 50 mm topping



Where enhanced performance is required, hollowcore floor slabs may be used in conjunction with a structural topping.

The units act compositely with the in-situ structural topping to create a robust, high-capacity composite floor. The structural topping overcomes possible differential camber between units. The topping is usually a grade C30/37 normal weight concrete, minimum 50 mm thick at midspan, reinforced with a light fabric. The soffit provides a utilitarian finish.

Advantages/disadvantages

These structurally efficient, robust slabs are quick to construct, and require little or no formwork and generally no propping unless there is a requirement to increase span or reduce depth. They can provide a range of spans with high load capacities. The flat utilitarian soffits can be used for passive cooling. Craneage is usually required, and allowance has to be made for camber. In very long spans, shrinkage and axial creep should be considered.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes.
Fire and durability – Fire resistance 1 hour; exposure class XC1. For car parks see Section 4.1.4.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Ultimate loads assume a reaction factor of 0.5 at end supports and 1.0 at internal supports (see Section 8.3.2).
Concrete – Precast C45/55, in-situ C30/37 ($f_{ck,i}$ = 25 MPa), 25 kN/m^3, 20 mm gravel aggregate. Precast installation at 28 days. Imposed loading at 28 days after topping cast. E_{cm} = 36,300 MPa.
Reinforcement – Strand and/or wire, f_{pk} = 1770 N/mm^2 stressed to 70%. c_{nom} = 20 mm to wire, 30 mm to strand (for indoor exposure).

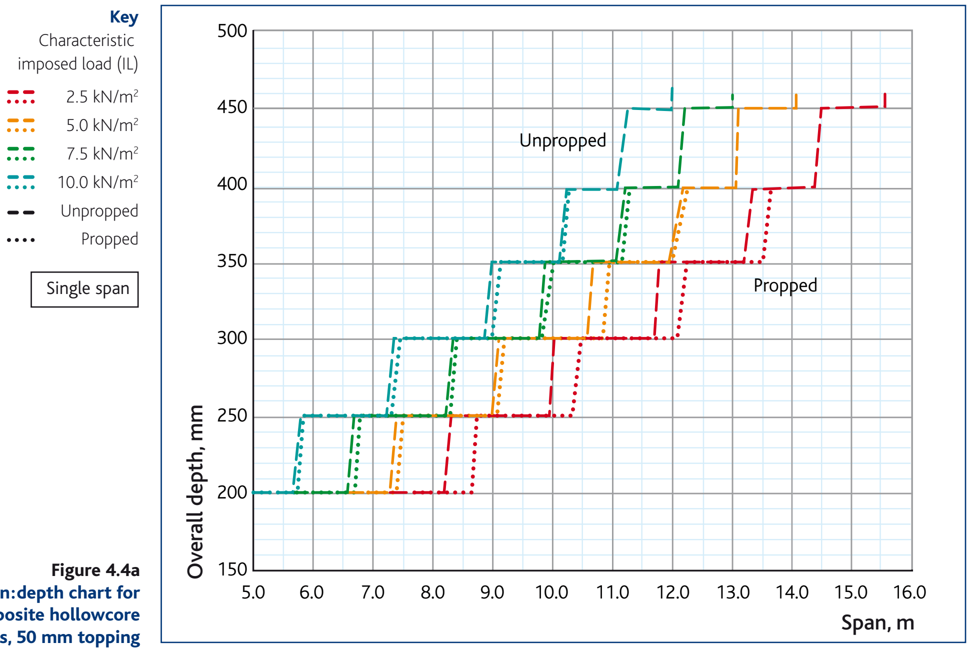


Figure 4.4a
Span:depth chart for composite hollowcore slabs, 50 mm topping

Table 4.4a
Data for composite hollowcore slabs, 50 mm topping

Span, m	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm, unpropped (propped)										
IL = 2.5 kN/m^2	200(200)	200(200)	200(200)	200(200)	250(250)	300(250)	300(300)	350(300)	350(350)	400(400)
IL = 5.0 kN/m^2	200(200)	200(200)	200(200)	250(250)	300(250)	300(300)	350(350)	350(350)	400(400)	450(450)
IL = 7.5 kN/m^2	200(200)	200(200)	250(250)	250(250)	300(300)	350(350)	350(350)	400(400)	450(450)	
IL = 10.0 kN/m^2	200(200)	250(250)	250(250)	300(300)	350(300)	350(350)	400(400)	450(450)		
Ultimate load to supporting beams, internal (end), kN/m										
IL = 2.5 kN/m^2	53 (27)	64 (32)	75 (37)	86 (43)	103 (51)	121 (60)	133 (66)	154 (77)	167 (83)	190 (95)
IL = 5.0 kN/m^2	72 (36)	87 (43)	101 (51)	122 (61)	142 (71)	158 (79)	182 (91)	199 (99)	225(113)	253(126)
IL = 7.5 kN/m^2	91 (45)	109 (55)	133 (66)	152 (76)	176 (88)	203(102)	223(112)	253(126)	284(142)	
IL = 10.0 kN/m^2	112 (56)	140 (70)	163 (82)	192 (96)	223(112)	248(124)	282(141)	317(158)		

Key
Overall depth
200 mm
250 mm
300 mm
350 mm
400 mm
450 mm
Unpropped
Propped

Single span

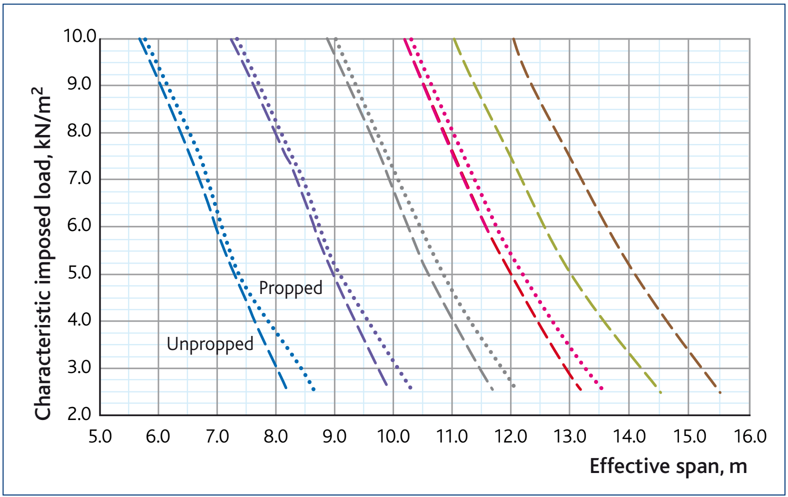


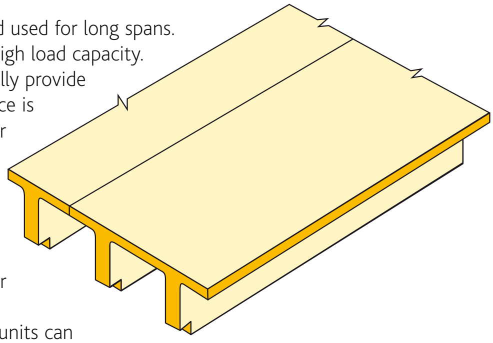
Figure 4.4b
Span:imposed load chart for composite hollowcore slabs, 50 mm topping

Table 4.4b
Data for composite hollowcore slabs, 50 mm topping

Imposed load (IL), kN/m^2, and ψ_2	2.5 $\psi_2 = 0.3$	5.0 $\psi_2 = 0.6$	7.5 $\psi_2 = 0.6$	10.0 $\psi_2 = 0.8$
Maximum span, m, unpropped (propped)				
Unit depth = 200 mm	8.20 (8.65)	7.20 (7.40)	6.60 (6.70)	5.70 (5.75)
Unit depth = 250 mm	9.95 (10.35)	8.90 (9.10)	8.15 (8.30)	7.25 (7.35)
Unit depth = 300 mm	11.70 (12.10)	10.60 (10.85)	9.80 (9.90)	8.90 (9.00)
Unit depth = 350 mm	13.20 (13.60)	12.00 (12.20)	11.05 (11.20)	10.20 (10.30)
Unit depth = 400 mm	14.40 (14.40*)	13.05 (13.05*)	12.05 (12.05*)	11.10 (11.10*)
Unit depth = 450 mm	15.60 (15.60*)	14.15 (14.15*)	13.00 (13.00*)	12.00 (12.00*)

Key
*At high spans and high imposed loads, the requirement for deflection after application of finishes < span/500 is critical. Therefore, propping has little or no influence

4.1.9 Precast double-tees, no topping



Precast double-tees are prestressed and used for long spans. They are relatively lightweight with a high load capacity. The units can be left exposed and usually provide two hours fire resistance. The top surface is intended to receive a levelling screed or flooring system, but un-topped units are occasionally used for long-span roofing.

Load sharing between units is achieved by the use of intermittent welded shear connectors. The double-tee is normally 2.4 m wide. Special narrow or tapered units can be produced to suit changes in the structural grid.

Advantages/disadvantages

These units are structurally efficient and are used in long-span, high-load flooring applications. With a low dead weight compared with other precast floor units, they are quick to erect and require little or no formwork or propping. The soffit has a pleasing ribbed finish. Unit weights of up to 15 tonnes may make craneage critical.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes.
Fire and durability – Fire resistance 2 hours; exposure class XC1. For car parks see Section 4.1.4.
Loads – No allowance for finishes or services is included. Ultimate loads assume a reaction factor of 0.5 at end supports and 1.0 at internal supports (see Section 8.3.2).
Concrete – C50/60, 25 kN/m^3; 20 mm gravel aggregate; h_f = 90 mm; b_{wmin} = 140 mm. Precast installation at 28 days. Imposed loading at 28 days after topping cast. E_{cm} = 37,300 MPa.
Reinforcement – 12.5 mm strand and/or wire, f_{pk} = 1770 N/mm^2 stressed to 70%. c_{nom} = 20 mm to wire, 30 mm to strand (for indoor exposure).

Key
Characteristic imposed load (IL)
- 2.5 kN/m^2
- 5.0 kN/m^2
- 7.5 kN/m^2
- 10.0 kN/m^2

Single span
Unpropped

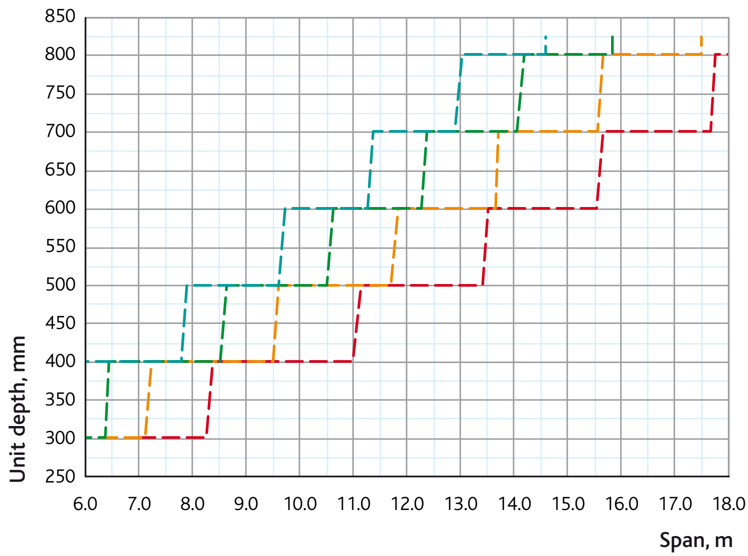


Figure 4.5a
Span: depth chart for precast double-tees, no topping

Table 4.5a
Data for precast double-tees, no topping

SINGLE span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Unit depth, m, unpropped									
IL = 2.5 kN/m^2	300	400	400	400	500	500	600	600	700
IL = 5.0 kN/m^2	400	400	500	500	600	600	700	700	800
IL = 7.5 kN/m^2	400	500	500	600	600	700	700	800	
IL = 10.0 kN/m^2	500	500	600	600	700	800	800		
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	75 (38)	89 (45)	99 (49)	109 (54)	124 (62)	135 (67)	152 (76)	162 (81)	181 (90)
IL = 5.0 kN/m^2	109 (55)	123 (61)	141 (71)	155 (78)	175 (87)	190 (95)	211(105)	226(113)	248(124)
IL = 7.5 kN/m^2	139 (70)	161 (80)	179 (89)	202(101)	220(110)	244 (122)	263(132)	289(144)	
IL = 10.0 kN/m^2	177 (89)	199(100)	226(113)	249(125)	278(139)	308(154)	331(166)		

Key
Unit depth
300 mm
400 mm
500 mm
600 mm
700 mm
800 mm

Single span
Unpropped

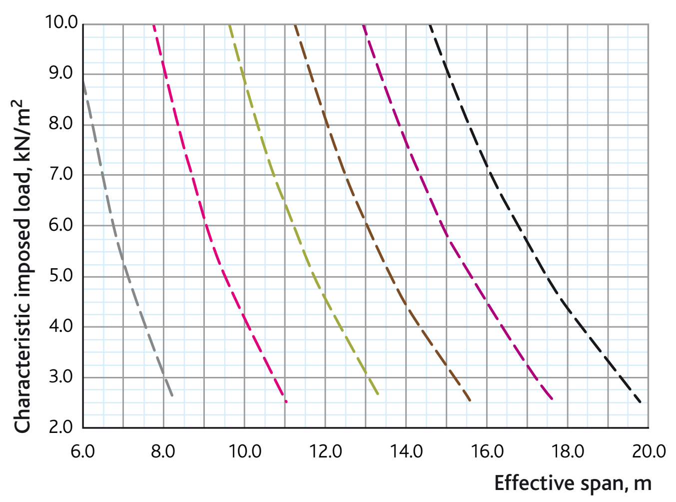


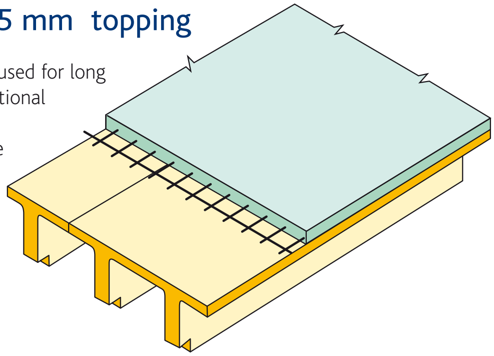
Figure 4.5b
Span:imposed load chart for precast double-tees, no topping

Table 4.5b
Data for precast double-tees, no topping

Imposed load (IL), kN/m^2, and ψ_2	2.5 ψ_2 = 0.3	5.0 ψ_2 = 0.6	7.5 ψ_2 = 0.6	10.0 ψ_2 = 0.8
Maximum span, m, unpropped				
Unit depth = 300 mm	8.30	7.15	6.40	5.80
Unit depth = 400 mm	11.05	9.55	8.55	7.80
Unit depth = 500 mm	13.45	11.75	10.55	9.65
Unit depth = 600 mm	15.60	13.65	12.30	11.30
Unit depth = 700 mm	17.70	15.60	14.10	12.95
Unit depth = 800 mm	19.80	17.50	15.85	14.60

4.1.10 Composite double-tees, 75 mm topping

Precast double-tees are prestressed and used for long spans. A structural topping provides additional capacity, robustness and buildability. The topping is usually grade C25/30 concrete and reinforced with fabric. The surface finish of the prestressed double-tee is left rough to provide sufficient bond. The units usually have two hours fire resistance and the soffit can be left exposed.



Advantages/disadvantages

These units are structurally efficient and are used in long-span, high-load flooring applications. The structural topping provides additional capacity, robustness and buildability. The units are quick to erect and require little or no formwork or propping. The soffit has a pleasing ribbed finish. Unit weights of up to 15 tonnes may make craneage critical.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes. Double-tees may be propped or unpropped at mid-span during construction.
Fire and durability – Fire resistance 2 hours; exposure class XC1. For car parks see Section 4.1.4.
Loads – No allowance for finishes or services is included. Ultimate loads assume a reaction factor of 0.5 at end supports and 1.0 at internal supports (see Section 8.3.2).
Concrete – Precast C50/60; 25 kN/m^3; 20 mm gravel aggregate, h_f = 90 mm, b_{wmin} = 140 mm. In-situ topping min. 75 mm thick at mid span, C30/37 ($f_{ck,i}$ = 30 MPa), 25 kN/m^3, 10 mm aggregate. Precast installation at 28 days. Imposed loading at 28 days after topping cast. E_{cm} = 37,300 MPa.
Reinforcement – 12.5 mm strand and/or wire, f_{pk} = 1770 N/mm^2 stressed to 70%.
c_{nom} = 20 mm to wire, 30 mm to strand (for indoor exposure).

Key
Characteristic imposed load (IL)
2.5 kN/m²
5.0 kN/m²
7.5 kN/m²
10.0 kN/m²
Unpropped
Propped

Single span

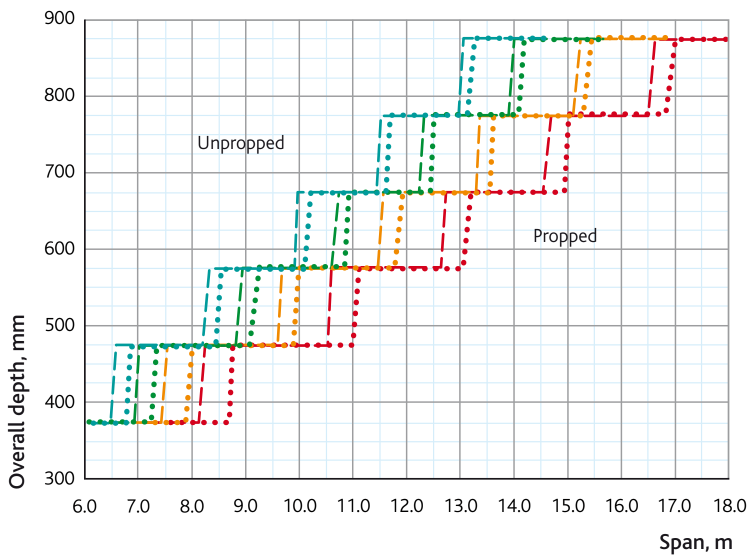


Figure 4.6a
Span: depth chart for composite double-tees, 75 mm topping

Table 4.6a
Data for composite double-tees, 75 mm topping

SINGLE span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Overall depth, m, unpropped (propped)									
IL = 2.5 kN/m^2	375(375)	475(475)	475(475)	575(475)	675(575)	675(575)	675(675)	775(775)	775(775)
IL = 5.0 kN/m^2	475(475)	475(475)	575(575)	575(575)	675(675)	675(675)	775(775)	775(775)	875(875)
IL = 7.5 kN/m^2	475(475)	575(475)	575(575)	675(675)	675(675)	775(775)	875(775)	875(875)	
IL = 10.0 kN/m^2	575(475)	575(575)	675(575)	675(675)	775(775)	875(775)	875(875)		
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	82 (41)	94 (47)	110 (55)	122 (61)	135 (67)	152 (76)	165 (83)	184 (92)	198 (99)
IL = 5.0 kN/m^2	109 (54)	128 (64)	144 (72)	165 (82)	181 (90)	203(102)	220(110)	243(122)	261(130)
IL = 7.5 kN/m^2	138 (69)	158 (79)	182 (91)	202(101)	227(114)	248(124)	275(137)	296(148)	324(162)
IL = 10.0 kN/m^2	169 (85)	197 (99)	222(111)	252(126)	277(138)	308(154)	340(170)	367(183)	

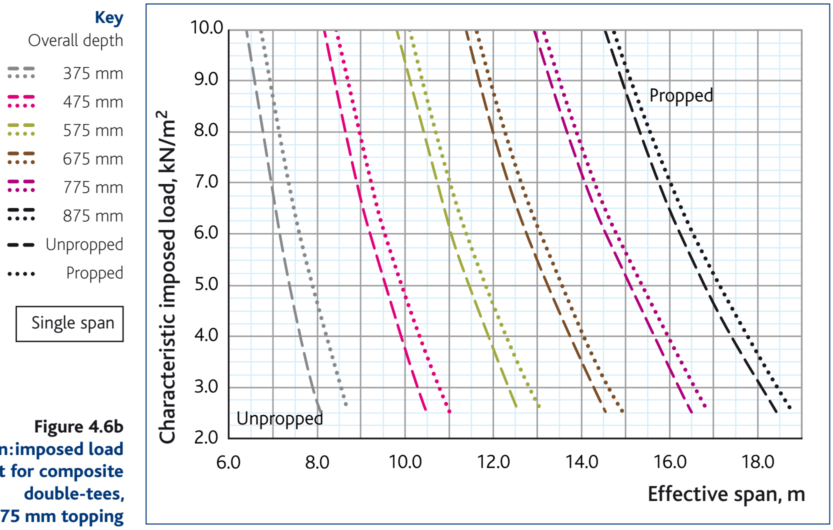
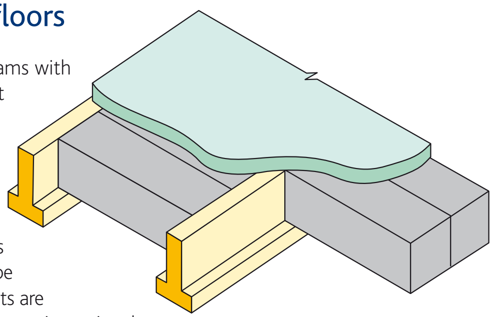


Figure 4.6b
Span:imposed load chart for composite double-tees, 75 mm topping

Table 4.6b
Data for composite double-tees, 75 mm topping

Imposed load (IL), kN/m^2, and ψ_2	2.5 ψ_2 = 0.3	5.0 ψ_2 = 0.6	7.5 ψ_2 = 0.6	10.0 ψ_2 = 0.8
Maximum span, m, unpropped (propped)				
Overall depth = 375 mm	8.10 (8.70)	7.40 (7.90)	6.90 (7.25)	6.45 (6.75)
Overall depth = 475 mm	10.50 (11.00)	9.55 (9.90)	8.80 (9.10)	8.20 (8.45)
Overall depth = 575 mm	12.60 (13.10)	11.45 (11.80)	10.60 (10.85)	9.85 (10.10)
Overall depth = 675 mm	14.55 (14.95)	13.25 (13.55)	12.20 (12.45)	11.40 (11.60)
Overall depth = 775 mm	16.50 (16.90)	15.10 (15.35)	13.90 (14.10)	1 2.95 (13.15)
Overall depth = 875 mm	18.45 (18.80)	16.80 (17.10)	15.55 (15.80)	14.55 (14.70)

4.1.11 Precast beam and block floors



These systems combine prestressed beams with either solid concrete blocks or specialist blocks, made, for example, of expanded polystyrene for insulated ground floors. Beam and block floors are widely used in the domestic market, but by using beams at close centres they can be used for commercial loadings for limited spans. Diaphragm action can be assured by using a structural topping. Units are manhandleable and ideal for use where access is restricted.

Flush soffits can be achieved by using specially shaped blocks. Holes can be formed by omitting blocks. Shallow slip tiles facilitate incorporation of service runs or solid sections of in-situ concrete.

Advantages/disadvantages

These smaller lighter units can be manhandled, and there is no need for formwork or propping, but the spans and capacities are limited.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes.
Fire and durability – Fire resistance 30 minutes; exposure class XC1.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Ultimate loads assume a reaction factor of 0.5 at end supports and 1.0 at internal supports (see Section 8.3.2).
Concrete – Precast beam C45/55. Gravel aggregates. Block density1480 kg/m^3. Precast installation at 28 days. Imposed loading at 28 days after topping cast. E_{cm} = 36,300 MPa.
Reinforcement – Strand, f_{pk} = 1770 N/mm^2 stressed to 70%. c_{nom} = 25 mm (for indoor exposure).

Key
Characteristic imposed load (IL)
1.0 kN/m^2
2.5 kN/m^2
5.0 kN/m^2
7.5 kN/m^2
10.0 kN/m^2

Single span
Unpropped

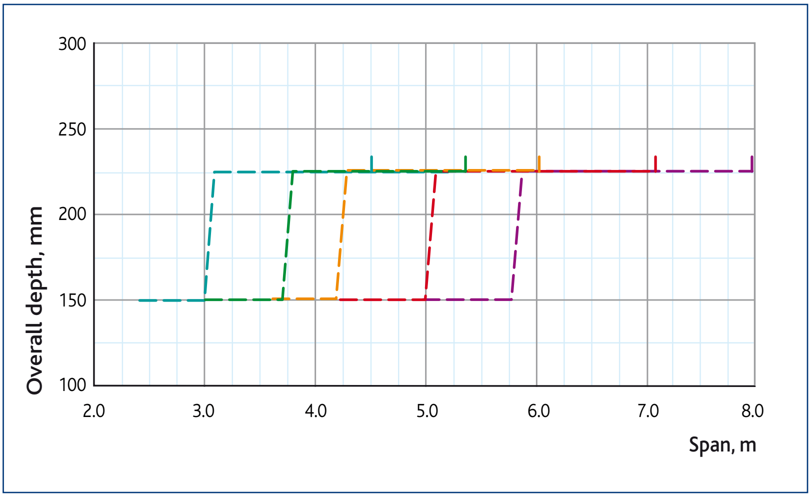


Figure 4.7a
Span: depth chart for precast beam and block floors

Table 4.7a
Data for precast beam and block floors

SINGLE span, m	3.0	4.0	5.0	6.0	7.0	8.0
Overall depth, mm, unpropped						
IL = 1.0 kN/m^2	150	150	150	225	225	225
IL = 2.5 kN/m^2	150	150	150	225	225	
IL = 5.0 kN/m^2	150	150	225	225		
IL = 7.5 kN/m^2	150	225	225			
IL = 10.0 kN/m^2	150	225				
Ultimate load to supporting beams, internal (end), kN/m						
IL = 1.0 kN/m^2	17 (9)	23 (11)	30 (15)	37 (19)	48 (24)	55 (28)
IL = 2.5 kN/m^2	24 (12)	33 (17)	42 (21)	55 (28)	64 (32)	
IL = 5.0 kN/m^2	35 (18)	48 (24)	65 (32)	78 (39)		
IL = 7.5 kN/m^2	47 (24)	64 (32)	83 (42)			
IL = 10.0 kN/m^2	59 (29)	82 (41)				

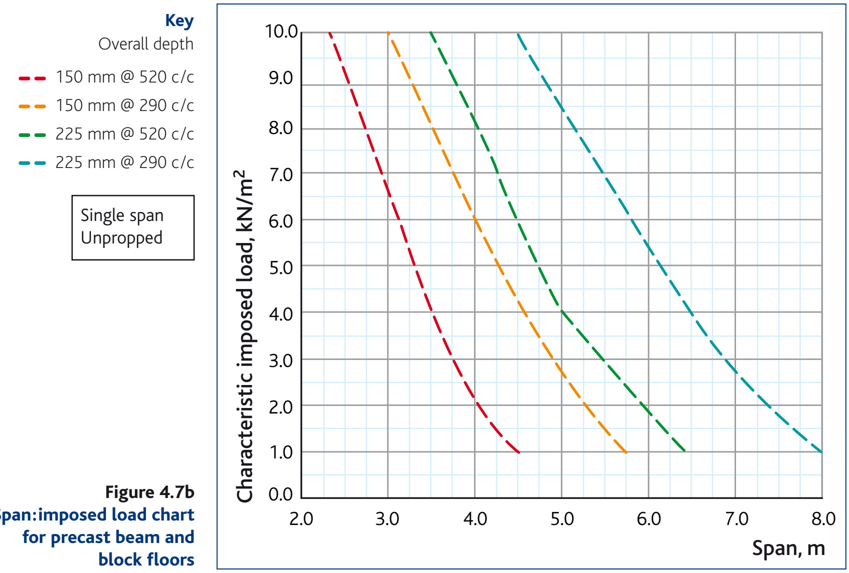


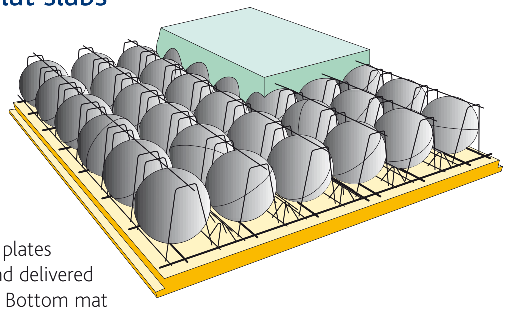
Figure 4.7b
Span:imposed load chart for precast beam and block floors

Table 4.7b
Data for precast beam and block floors

Imposed load (IL), kN/m^2, and ψ_2	1.0 $\psi_2 = 0.3$	2.5 $\psi_2 = 0.3$	5.0 $\psi_2 = 0.6$	7.5 $\psi_2 = 0.6$	10.0 $\psi_2 = 0.8$
Maximum span, m, unpropped					
Overall depth = 150 mm, beams at 520 mm c/c	4.55	3.90	3.30	2.90	2.40
Overall depth = 150 mm, beams at 290 mm c/c	5.75	5.05	4.21	3.70	3.05
Overall depth = 225 mm, beams at 520 mm c/c	6.40	5.60	4.70	4.15	3.50
Overall depth = 225 mm, beams at 290 mm c/c	8.00	7.10	6.05	5.40	4.50

Note
Depths and spacings of beams may vary between manufacturers

4.1.12 Composite biaxial voided flat slabs



This system, which uses plastic spheres as void formers, enables true two-way flat slab design, fast construction and a high quality flat soffit. Spans of up to 17 m are possible.

Precast plates act as permanent formwork and are provided with two-way reinforcement and shear links cast in. The plates are typically aligned with the slab span and delivered in panels up to 12 m long by 2.4 m wide. Bottom mat splice steel and all top steel is provided and fixed by the contractor before the topping is cast.

Advantages/disadvantages

This system is fast to construct and the flat soffits allow easy service installation. Enhanced shear strength is provided around columns by removing void formers. The voids reduce the self-weight by 20% to 25% compared with solid flat slab floors but tend to increase deflection. Early supplier involvement is required to gain maximum benefit.

Design assumptions

Supported by – Columns. Refer to column charts and data to estimate sizes. All propped at mid-span.
Fire and durability – Fire resistance 2 hours; exposure class XC1. For car parks see Section 4.1.4.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Ultimate loads assume a reaction factor of 0.5 at end supports and 1.0 at internal supports (see Section 8.4.5).
Concrete – Infill concrete is normally C30/37 grade, with the precast typically C45/50. Precast installation at 28 days. Imposed loading at 28 days after topping cast.
Reinforcement – f_{yk} = 500 N/mm^2. c_{nom} = 20 mm (for indoor exposure).

Key
Characteristic imposed load (IL)
- 2.5 kN/m^2
- 5.0 kN/m^2
- 7.5 kN/m^2
- 10.0 kN/m^2

Multiple span
Propped

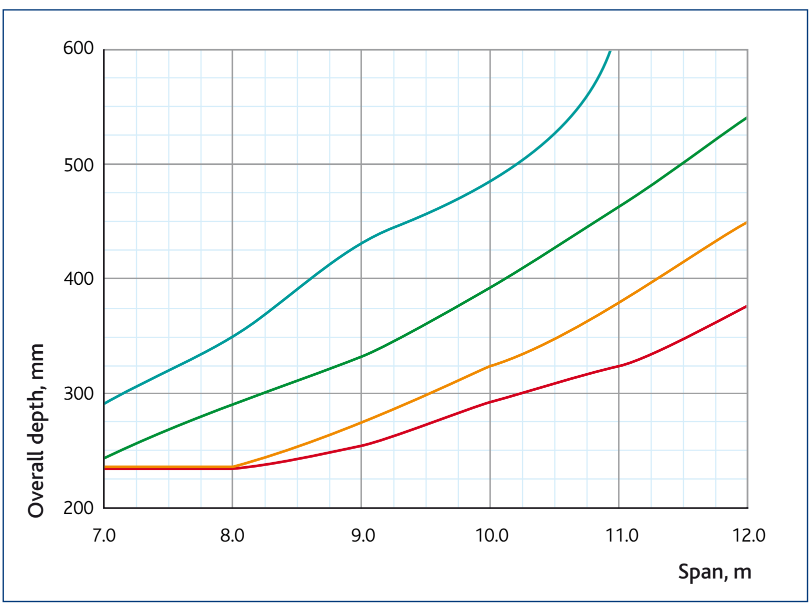


Figure 4.8
Span: depth chart for composite biaxial voided flat slabs

Table 4.8
Data for composite biaxial voided flat slabs

MULTIPLE span, m	7.0	8.0	9.0	10.0	11.0	12.0
Overall depth, mm, propped						
IL = 2.5 kN/m^2	230	230	252	292	334	375
IL = 5.0 kN/m^2	230	230	275	319	374	454
IL = 7.5 kN/m^2	245	296	335	389	458	524
IL = 10.0 kN/m^2	293	351	430	484	605	
Ultimate load to supporting columns, internal (edge*), kN; * excludes cladding loads						
IL = 2.5 kN/m^2	540 (270)	710 (355)	940 (470)	1250 (625)	1640 (820)	2090 (1045)
IL = 5.0 kN/m^2	720 (360)	940 (470)	1270 (635)	1680 (840)	2190 (1095)	2880 (1440)
IL = 7.5 kN/m^2	910 (455)	1270 (635)	1680 (840)	2200 (1100)	2860 (1430)	3640 (1820)
IL = 10.0 kN/m^2	1200 (600)	1660 (830)	2270 (1135)	2930 (1465)	3920 (1960)	

4.2 Precast beams

4.2.1 Using precast beams

Factory-engineered precast concrete frames are used in offices, car parks, and commercial and industrial developments of all types. Precast beams facilitate speed of erection by eliminating formwork, propping and, in many cases, site-applied finishes and follow-on trades. They have inherent fire resistance, durability and the potential for a vast range of integral and applied finishes.

Manufacturers produce a wide range of preferred cross-sections based on 50 mm increments. Designs with other cross-sections are easily accommodated. However, the economics of precasting beams depend on repetition, as a major cost item is the manufacture of the base moulds. Manufacturers should be consulted at the earliest opportunity.

4.2.2 The charts and data

The charts and data cover reinforced and prestressed precast beams. They include a range of web widths and ultimate applied uniformly distributed loads (uaudl). The types of beams covered are:

- Rectangular beams, e.g. isolated or upstand beams
- L-beams or single booted beams, e.g. perimeter beams supporting hollowcore floor units
- (Inverted) T-beams or double booted beams, e.g. internal beams supporting hollowcore floor units

Some benefit may be gained by using flange action and/or temporary propping, but for simplicity, the charts and data in this publication assume that the beams are simply supported and non-composite.

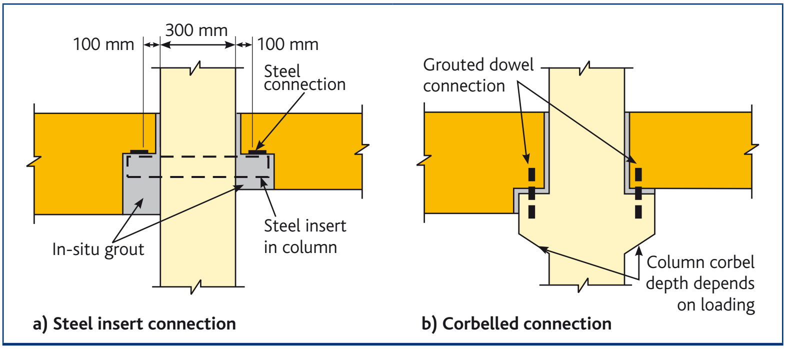


Figure 4.D
Typical precast beam support details

The economic depths of precast beams were determined using effective spans (centreline of support to centreline of support). The centreline of support was assumed to be 250 mm from the centre of the columns (by assuming 300 mm wide columns and 100 mm from edge of the column to the centreline of support each end: see Figure 4.D). The centreline column to centreline column dimension (span col. c/c) is highlighted in the charts and data and this 'full' span dimension should be used in assessing loads to supports and columns.

From the appropriate chart(s), use the maximum span (col. c/c) and appropriate ultimate applied uniformly distributed loads to determine overall depth. The user is expected to interpolate between values given in the charts and data, and round up both the depth and loads to supports in line with normal modular sizing and his or her confidence in the design criteria used.

4.2.3 Design assumptions for reinforced precast beams

Support
Precast beams are assumed to be simply supported by precast columns with compatible connection details. Refer to column charts and data to estimate sizes.

Loads
Ultimate loads to columns assume elastic reaction factors of 1.0 to internal columns and 0.5 to end columns. A dead to imposed load ratio of 1 to 1 has been assumed, with a quasi-permanent load factor, ψ_2, of 0.6.

Fire and durability
Fire resistance 1 hour; exposure class up to XD1.

Span
The economic depths of precast beams were determined using effective spans (centreline of support to centreline of support). Note that the 'full' span (col. c/c) dimension should be used in assessing loads to supports and columns.

Ledge size
Where appropriate, the ledge (or boot) width has been taken to be 125 mm. This allows 75 mm bearing, 10 mm fixing tolerance and 40 mm for in-situ infill. The ledge depth has been taken as 150 mm.

Concrete
C40/50; 25 kN/m^3; 20 mm aggregate. Fair-faced finish. Please note that concrete grades up to C60/80 are commonly used to facilitate early removal from moulds.

Reinforcement
Main bars: maximum H32 top and bottom, minimum H12 top and bottom at simply supported ends, minimum links H8.

Cover
Cover to link, c_{nom} = 25 mm. In accordance with Clause 4.4(N) of Eurocode 2[2] and its UK National Annex[2a], Δc_{dev} for precast units has been taken as 0 mm (non-conforming units are rejected).

4.2.4 Design assumptions for prestressed precast beams

Support
Precast beams are assumed to be simply supported by precast columns with compatible connection details. Refer to column charts and data to estimate sizes.

Loads
Ultimate loads to columns assume elastic reaction factors of 1.0 to internal columns and 0.5 to end columns. A dead to imposed load ratio of 1 to 1 has been assumed, with a quasi-permanent load factor, ψ_2, of 0.8.

Fire and durability
Fire resistance 1.5 hours; exposure class up to XC1.

Span
The economic depths of precast beams were determined using effective spans (centreline of support to centreline of support). Note that the 'full' span (col. c/c) dimension should be used in assessing loads to supports and columns.

Ledge size
For inverted T-beams, ledge widths of 125 mm and depths of 150 mm have been assumed.

Concrete
Grade C50/60, 25 kN/m^3, 20 mm gravel aggregate. Fair-faced finish. E_{cm} = 37,300 MPa.

Reinforcement
Strand, f_{pk} = 1770 MPa. Initially stressed to 70%. Losses assumed to be 25%.

Cover
A nominal cover to the strand of 40 mm has been assumed.

4.2.5 Rectangular precast beams, single span, 300 mm wide

Design assumptions

Design and dimensions – See Section 4.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XD1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

Key
Ultimate applied udl (uaudl)
- 25 kN/m
- 50 kN/m
- 75 kN/m
- 100 kN/m
- 150 kN/m

Single span

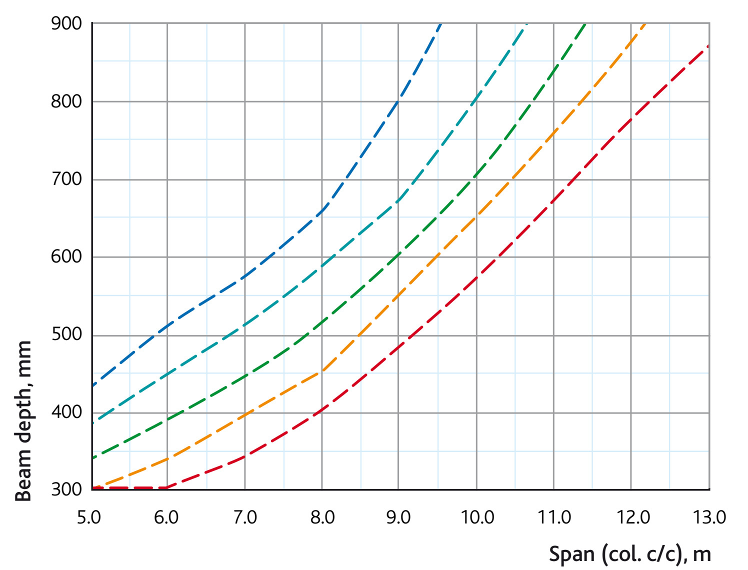


Figure 4.9
Span:depth chart for single-span rectangular precast beams, 300 mm wide

Table 4.9
Data for rectangular single-span, precast beams, 300 mm wide

Span col. c/c, m (effective span, m)	5.0 (4.5)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)
Depth, mm									
uaudl = 25 kN/m	300	300	342	403	483	572	678	775	872
uaudl = 50 kN/m	300	340	399	452	553	652	757	875	993
uaudl = 75 kN/m	332	406	451	500	592	719	846	987	
uaudl = 100 kN/m	385	453	510	591	672	807	954		
uaudl = 150 kN/m	433	512	577	661	798	990			
Ultimate load to supports/columns, internal (end), kN									
uaudl = 25 kN/m	139 (70)	167 (83)	197 (99)	230 (115)	266 (133)	304 (152)	345 (172)	387 (194)	431 (216)
uaudl = 50 kN/m	264 (132)	319 (160)	376 (188)	434 (217)	497 (248)	561 (281)	628 (314)	698 (349)	771 (386)
uaudl = 75 kN/m	391 (195)	473 (236)	555 (277)	638 (319)	725 (362)	817 (409)	912 (456)	1011 (506)	
uaudl = 100 kN/m	518 (259)	625 (313)	733 (367)	844 (422)	957 (478)	1076 (538)	1198 (599)		
uaudl = 150 kN/m	770 (385)	929 (464)	1088 (544)	1250 (625)	1417 (709)	1593 (796)			
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	13 (148)	19 (212)	25 (243)	24 (200)	24 (169)	26 (149)	32 (155)	33 (140)	33 (125)
uaudl = 50 kN/m	26 (286)	27 (263)	33 (274)	40 (298)	38 (230)	40 (203)	46 (201)	46 (174)	46 (153)
uaudl = 75 kN/m	27 (273)	28 (232)	46 (343)	61 (404)	63 (383)	82 (458)	47 (187)	42 (128)	
uaudl = 100 kN/m	27 (235)	36 (262)	41 (268)	45 (253)	52 (256)	52 (215)	52 (183)		
uaudl = 150 kN/m	44 (335)	45 (293)	60 (347)	68 (344)	57 (237)	55 (185)			
Variations: for uaudl = 50 kN/m									
2 hours fire	300	361	419	489	593	708	833	970	1119
4 hours fire	351	427	506	609	756	938	1153		

4.2.6 Rectangular precast beams, single span, 450 mm wide

Design assumptions

Design and dimensions – See Section 4.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
75 kN/m
100 kN/m
150 kN/m

Single span

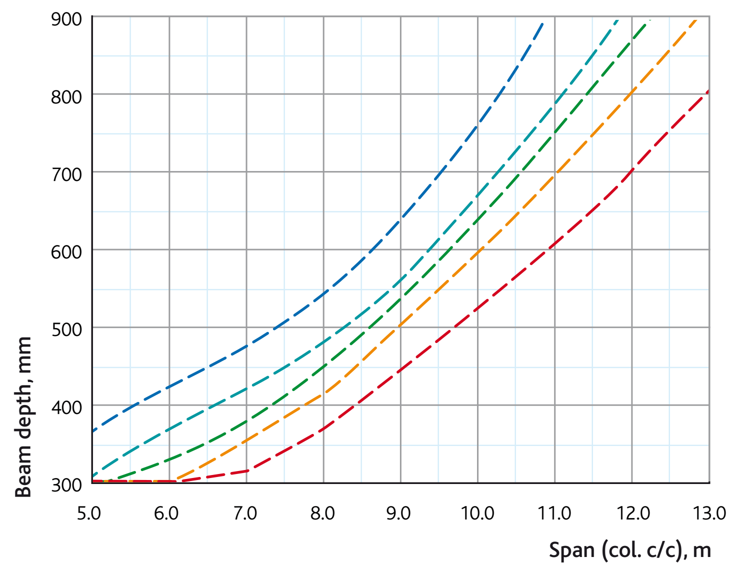
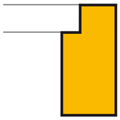


Figure 4.10
Span:depth chart for single-span rectangular precast beams, 450 mm wide

Table 4.10
Data for single-span rectangular precast beams, 450 mm wide

Span col. c/c, m (effective span, m)	5.0 (4.5)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)
Depth, mm									
uaudl = 25 kN/m	300	300	318	374	448	528	613	703	805
uaudl = 50 kN/m	300	305	357	419	509	600	697	808	916
uaudl = 75 kN/m	300	334	394	435	545	650	755	867	989
uaudl = 100 kN/m	312	377	430	475	561	682	796	913	
uaudl = 150 kN/m	372	425	487	552	623	777	918		
Ultimate load to supports/columns, internal (end), kN									
uaudl = 25 kN/m	146 (73)	175 (88)	206 (103)	242 (121)	282 (141)	324 (162)	370 (185)	419 (209)	472 (236)
uaudl = 50 kN/m	271 (136)	326 (163)	385 (193)	447 (224)	514 (257)	584 (292)	658 (329)	736 (368)	817 (409)
uaudl = 75 kN/m	396 (198)	478 (239)	564 (282)	649 (324)	744 (372)	841 (421)	942 (471)	1046 (523)	1156 (578)
uaudl = 100 kN/m	522 (261)	632 (316)	742 (371)	853 (427)	971 (486)	1096 (548)	1223 (612)	1354 (677)	
uaudl = 150 kN/m	776 (388)	936 (468)	1098 (549)	1262 (631)	1429 (714)	1609 (805)	1792 (896)		
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	13 (96)	18 (137)	25 (173)	28 (164)	31 (152)	31 (132)	42 (151)	37 (117)	39 (108)
uaudl = 50 kN/m	19 (141)	34 (251)	36 (223)	41 (215)	47 (207)	49 (182)	49 (158)	51 (139)	56 (135)
uaudl = 75 kN/m	29 (212)	43 (286)	44 (250)	57 (290)	54 (221)	64 (219)	64 (188)	65 (166)	71 (159)
uaudl = 100 kN/m	39 (277)	40 (235)	56 (290)	75 (352)	76 (301)	72 (234)	77 (214)	77 (187)	
uaudl = 150 kN/m	40 (237)	61 (317)	74 (340)	81 (327)	97 (346)	85 (243)	86 (207)		
Variations: for uaudl = 50 kN/m									
2 hours fire	300	315	363	427	515	606	703	814	922
4 hours fire	300	351	401	457	549	642	740	844	958

4.2.7 Precast L-beams, single span, 300 mm overall width



Design assumptions

Design and dimensions – See Section 4.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XD1.
ψ_2 factor – ψ_2 = 0.6.
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
75 kN/m
100 kN/m
150 kN/m

Single span

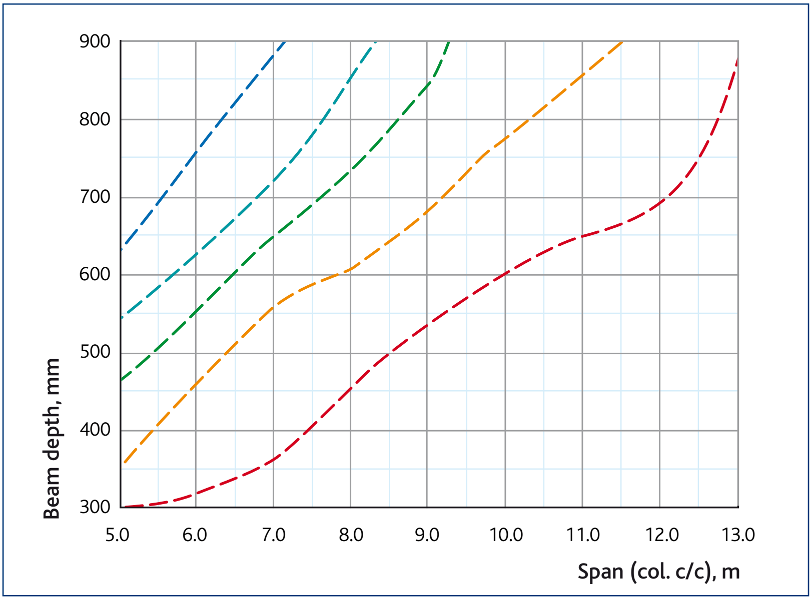
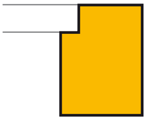


Figure 4.11
Span:depth chart for single-span precast L-beams, 300 mm overall width

Table 4.11
Data for single-span precast L-beams, 300 mm overall width

Span col. c/c, m (effective span, m)	5.0 (4.5)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)
Depth, mm									
uaudl = 25 kN/m	300	307	357	459	541	595	647	699	876
uaudl = 50 kN/m	347	463	559	617	689	778	867	940	
uaudl = 75 kN/m	463	568	649	727	848	933			
uaudl = 100 kN/m	546	632	720	854	950				
uaudl = 150 kN/m	628	753	889						
Ultimate load to supports/columns, internal (end), kN									
uaudl = 25 kN/m	136 (68)	164 (82)	194 (97)	230 (115)	265 (133)	300 (150)	335 (168)	372 (186)	424 (212)
uaudl = 50 kN/m	263 (132)	323 (161)	383 (191)	442 (221)	503 (251)	567 (284)	633 (316)	699 (349)	
uaudl = 75 kN/m	394 (197)	478 (239)	563 (282)	650 (325)	741 (371)	832 (416)			
uaudl = 100 kN/m	523 (261)	632 (316)	743 (372)	859 (430)	975 (487)				
uaudl = 150 kN/m	777 (388)	939 (469)	1104 (552)						
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	15 (171)	19 (207)	20 (190)	21 (152)	23 (141)	26 (143)	26 (136)	33 (155)	34 (128)
uaudl = 50 kN/m	19 (165)	22 (175)	24 (167)	26 (148)	27 (133)	31 (137)	32 (128)	49 (173)	
uaudl = 75 kN/m	22 (158)	24 (142)	28 (144)	31 (144)	31 (124)	36 (130)			
uaudl = 100 kN/m	23 (142)	28 (150)	32 (148)	32 (126)	37 (131)				
uaudl = 150 kN/m	30 (157)	33 (148)	34 (127)	39 (130)					
Variations: for uaudl = 50 kN/m									
2 hours fire	355	511	608	709	790	837	908	979	
4 hours fire	Section not wide enough								

4.2.8 Precast L-beams, single span, 450 mm overall width



Design assumptions

Design and dimensions – See Section 4.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
75 kN/m
100 kN/m
150 kN/m

Single span

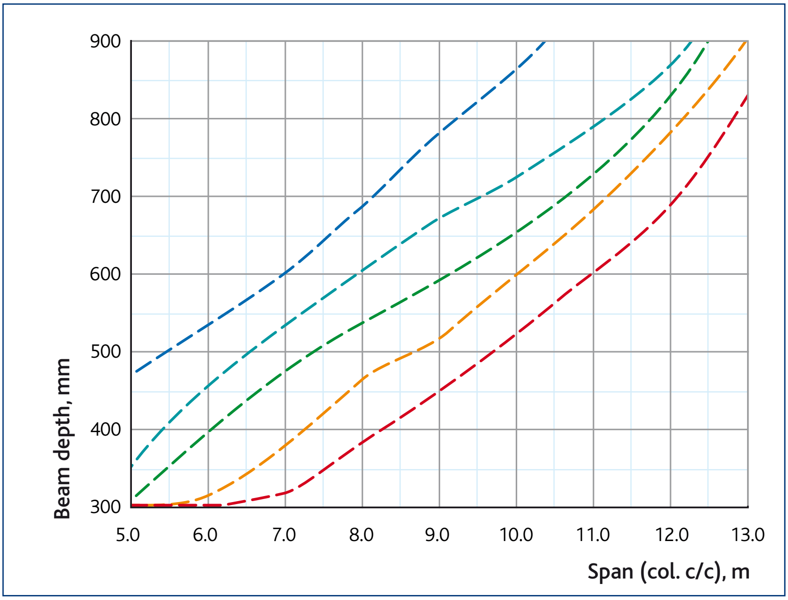
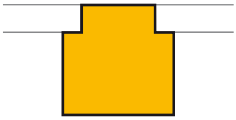


Figure 4.12
Span:depth chart for single-span precast L-beams 450 mm overall width

Table 4.12
Data for single-span precast L-beams, 450 mm overall width

Span col. c/c, m (effective span, m)	5.0 (4.5)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)
Depth, mm									
uaudl = 25 kN/m	300	300	319	377	450	527	615	698	842
uaudl = 50 kN/m	300	315	387	466	510	595	689	797	903
uaudl = 75 kN/m	315	396	481	532	597	661	730	833	976
uaudl = 100 kN/m	360	461	529	602	666	739	794	868	
uaudl = 150 kN/m	470	541	600	690	781	860	947		
Ultimate load to supports/columns, internal (end), kN									
uaudl = 25 kN/m	143 (72)	172 (86)	202 (101)	238 (119)	277 (138)	318 (159)	364 (182)	411 (205)	471 (236)
uaudl = 50 kN/m	268 (134)	323 (162)	384 (192)	448 (224)	509 (255)	578 (289)	650 (325)	727 (364)	807 (404)
uaudl = 75 kN/m	394 (197)	480 (240)	568 (284)	655 (328)	745 (373)	837 (419)	931 (466)	1034 (517)	1146 (573)
uaudl = 100 kN/m	522 (261)	635 (318)	748 (374)	863 (432)	979 (490)	1098 (549)	1216 (608)	1339 (670)	
uaudl = 150 kN/m	780 (390)	942 (471)	1105 (552)	1273 (636)	1444 (722)	1615 (808)	1790 (895)		
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	16 (117)	20 (147)	25 (174)	28 (166)	28 (138)	31 (133)	32 (115)	37 (118)	36 (95)
uaudl = 50 kN/m	23 (170)	48 (337)	37 (223)	46 (245)	45 (200)	46 (170)	51 (165)	53 (150)	54 (131)
uaudl = 75 kN/m	46 (326)	34 (192)	42 (192)	49 (204)	50 (188)	57 (190)	65 (197)	74 (198)	74 (168)
uaudl = 100 kN/m	33 (202)	48 (232)	50 (210)	51 (190)	55 (184)	61 (183)	72 (202)	83 (212)	
uaudl = 150 kN/m	44 (208)	53 (219)	55 (203)	65 (209)	70 (199)	70 (182)	80 (188)		
Variations: for uaudl = 50 kN/m									
2 hours fire	300	325	396	473	537	600	706	808	926
4 hours fire	300	351	433	500	584	653	742	845	964

4.2.9 Precast inverted T-beams, single span, 600 mm overall width



Design assumptions

Design and dimensions – See Section 4.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XD1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.
Reinforcement – $f_{yk} = 500$ MPa.

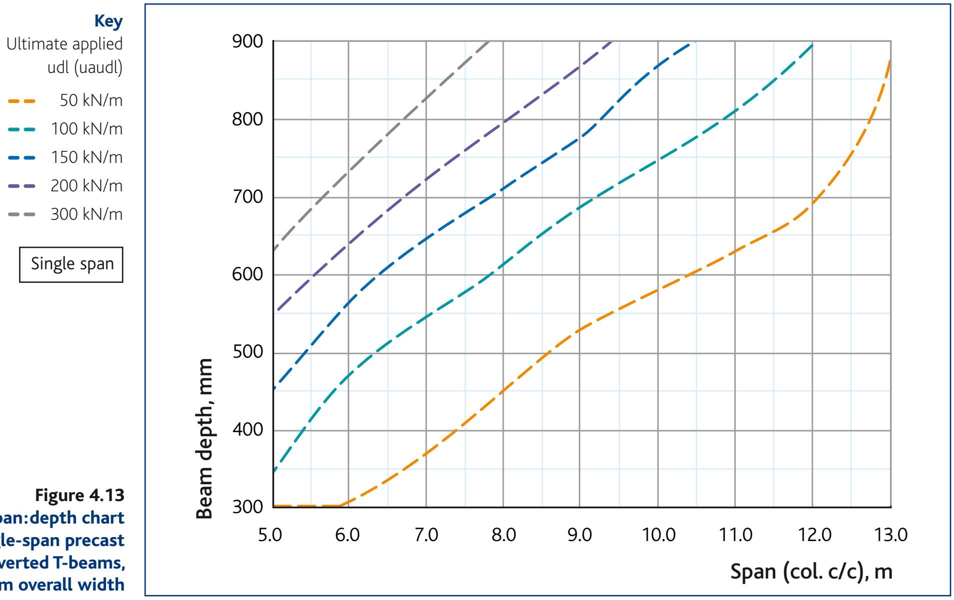
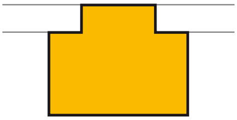


Figure 4.13
Span:depth chart for single-span precast inverted T-beams, 600 mm overall width

Table 4.13
Data for single-span precast inverted T-beams, 600 mm overall width

Span col. c/c, m (effective span, m)	5.0 (4.5)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)
Depth, mm									
uaudl = 50 kN/m	300	307	360	443	539	584	635	691	867
uaudl = 100 kN/m	346	454	556	615	674	736	815	878	
uaudl = 150 kN/m	450	565	643	717	780	864	926	999	
uaudl = 200 kN/m	556	636	720	793	888	953			
uaudl = 300 kN/m	634	736	833	926	991				
Ultimate load to supports/columns, internal (end), kN									
uaudl = 50 kN/m	272 (136)	328 (164)	389 (195)	457 (229)	530 (265)	598 (299)	668 (334)	741 (371)	846 (423)
uaudl = 100 kN/m	527 (263)	644 (322)	765 (382)	883 (441)	1003 (502)	1126 (563)	1255 (628)	1383 (692)	
uaudl = 150 kN/m	786 (393)	957 (478)	1126 (563)	1298 (649)	1471 (736)	1650 (825)	1828 (914)	2011(1005)	
uaudl = 200 kN/m	1046 (523)	1265 (632)	1486 (743)	1710 (855)	1939 (970)	2167(1083)			
uaudl = 300 kN/m	1554 (777)	1876 (938)	2201(1101)	2530(1265)	2857(1428)				
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	26 (143)	45 (246)	51 (235)	48 (182)	48 (150)	59 (167)	65 (171)	66 (159)	63 (121)
uaudl = 100 kN/m	45 (219)	44 (162)	53 (158)	55 (148)	62 (154)	72 (163)	72 (147)	82 (156)	
uaudl = 150 kN/m	46 (170)	53 (155)	61 (158)	71 (165)	74 (158)	77 (148)	83 (150)	93 (156)	
uaudl = 200 kN/m	46 (138)	55 (144)	72 (167)	75 (158)	83 (156)	93 (162)			
uaudl = 300 kN/m	57 (150)	76 (173)	80 (160)	89 (160)	108 (182)				
Variations: for uaudl = 100 kN/m									
2 hours fire	350	458	566	636	694	755	826	893	1021
4 hours fire	376	485	630	720	792	849	911	991	1104

4.2.10 Precast inverted T-beams, single span, 750 mm overall width



Design assumptions

Design and dimensions – See Section 4.2.3.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.6$.
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
150 kN/m
200 kN/m
300 kN/m

Single span

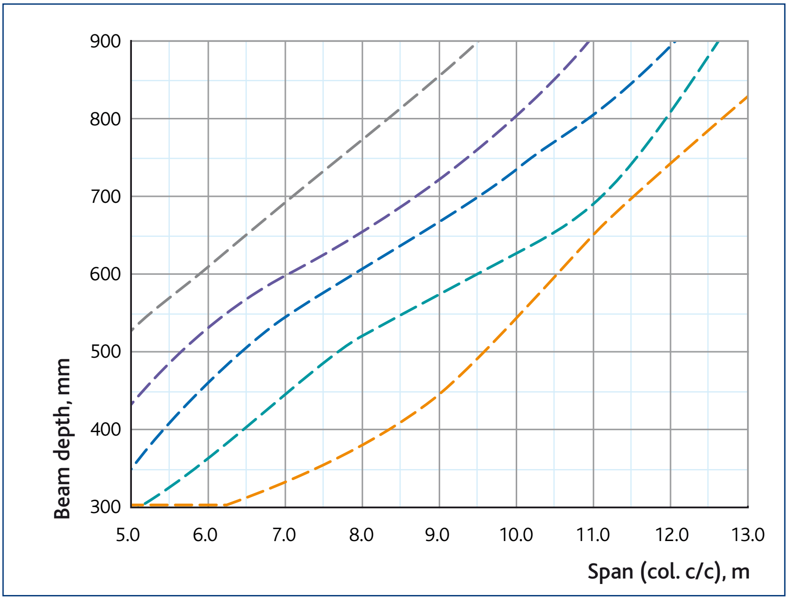


Figure 4.14
Span:depth chart for single-span precast inverted T-beams, 750 mm overall width

Table 4.14
Data for single-span precast inverted T-beams, 750 mm overall width

Span col. c/c, m (effective span)	5.0 (4.5)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)
Depth, mm									
uaudl = 50 kN/m	300	300	336	383	445	556	643	725	823
uaudl = 100 kN/m	300	357	450	528	583	627	694	815	979
uaudl = 150 kN/m	356	468	546	602	664	735	799	895	967
uaudl = 200 kN/m	432	535	600	669	729	810	906		
uaudl = 300 kN/m	533	613	698	783	869	931			
Ultimate load to supports/columns, internal (end), kN									
uaudl = 50 kN/m	279 (140)	335 (168)	397 (198)	462 (231)	533 (267)	619 (309)	703 (351)	790 (395)	886 (443)
uaudl = 100 kN/m	529 (265)	643 (322)	766 (383)	890 (445)	1012 (506)	1135 (568)	1266 (633)	1415 (708)	1583 (792)
uaudl = 150 kN/m	786 (393)	959 (479)	1131 (566)	1304 (652)	1480 (740)	1661 (830)	1843 (922)	2038(1019)	2229(1115)
uaudl = 200 kN/m	1045 (522)	1268 (634)	1490 (745)	1716 (858)	1943 (972)	2178(1089)	2421(1210)		
uaudl = 300 kN/m	1557 (778)	1879 (940)	2206(1103)	2537(1269)	2873(1436)	3206(1603)			
Reinforcement, kg/m (kg/m³)									
uaudl = 50 kN/m	27 (120)	39 (172)	47 (188)	60 (208)	61 (183)	54 (129)	65 (135)	77 (141)	73 (118)
uaudl = 100 kN/m	45 (201)	52 (193)	59 (174)	71 (179)	76 (175)	91 (194)	94 (181)	95 (155)	91 (124)
uaudl = 150 kN/m	52 (195)	60 (170)	73 (177)	78 (172)	105 (211)	97 (176)	105 (176)	116 (172)	133 (183)
uaudl = 200 kN/m	57 (175)	64 (160)	87 (192)	107 (214)	95 (173)	101 (166)	109 (160)		
uaudl = 300 kN/m	67 (167)	86 (187)	102 (194)	105 (179)	111 (171)	135 (193)			
Variations: for uaudl = 100 kN/m									
2 hours fire	301	362	454	547	598	645	705	814	954
4 hours fire	322	427	538	597	653	701	764	830	968

4.2.11 Rectangular precast prestressed beams, single span, 300 mm wide

Design assumptions

Design and dimensions – See Section 4.2.4.
Fire and durability – Fire resistance 1.5 hours; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.8$.
Concrete – C50/60; 25 kN/m^3; 20 mm gravel aggregate.
Reinforcement – Strand, f_{pk} = 1770 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
75 kN/m
100 kN/m
150 kN/m

Single span

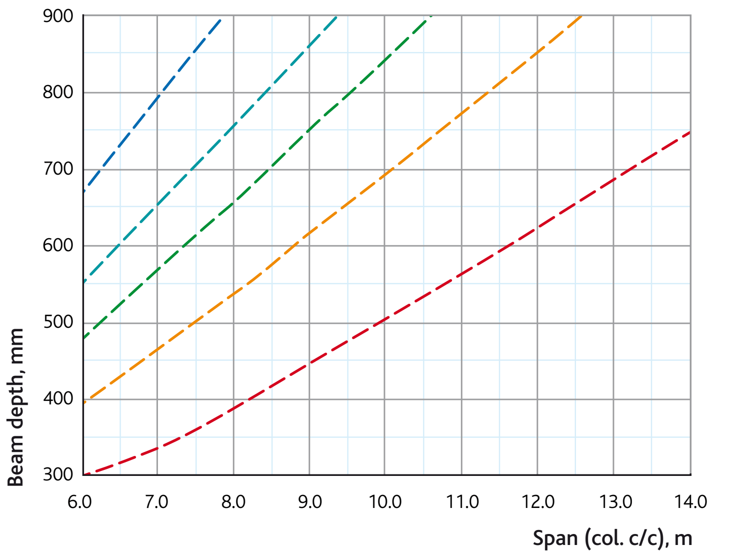


Figure 4.15
Span:depth chart for single-span rectangular precast prestressed beams, 300 mm wide

Table 4.15
Data for single-span rectangular precast prestressed beams, 300 mm wide

Span col. c/c, m (effective span, m)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)	14.0 (13.5)
Depth, mm									
uaudl = 25 kN/m	300	335	390	450	505	565	625	690	750
uaudl = 50 kN/m	395	465	540	620	695	775	855	940	
uaudl = 75 kN/m	480	570	660	755	845	940			
uaudl = 100 kN/m	550	655	760	865	970				
uaudl = 150 kN/m	670	795	920						
Ultimate load to supports/columns, internal (end), kN									
uaudl = 25 kN/m	167 (83)	197 (98)	229 (115)	263 (131)	297 (149)	333 (167)	370 (185)	409 (205)	448 (224)
uaudl = 50 kN/m	322 (161)	381 (190)	441 (220)	502 (251)	565 (283)	630 (315)	696 (348)	765 (382)	
uaudl = 75 kN/m	477 (239)	562 (281)	650 (325)	739 (369)	829 (415)	922 (461)			
uaudl = 100 kN/m	631 (315)	743 (371)	857 (429)	973 (486)	1091 (545)				
uaudl = 150 kN/m	938 (469)	1102 (551)	1269 (635)						
Prestressing strand, kg/m (kg/m³) excluding links/carriers									
uaudl = 25 kN/m	5 (55)	6 (55)	6 (55)	7 (55)	8 (55)	9 (55)	10 (55)	11 (55)	12 (55)
uaudl = 50 kN/m	7 (55)	8 (55)	9 (55)	10 (55)	11 (55)	13 (55)	14 (55)	15 (55)	
uaudl = 75 kN/m	8 (55)	9 (55)	11 (55)	12 (55)	14 (55)	15 (55)			
uaudl = 100 kN/m	9 (55)	11 (55)	13 (55)	14 (55)	16 (55)				
uaudl = 150 kN/m	11 (55)	13 (55)	15 (55)						

4.2.12 Rectangular precast prestressed beams, single span, 450 mm wide

Design assumptions

Design and dimensions – See Section 4.2.4.
Fire and durability – Fire resistance 1.5 hours; exposure class XC1.
ψ_2 factor – ψ_2 = 0.8.
Concrete – C50/60; 25 kN/m^3; 20 mm gravel aggregate.
Reinforcement – Strand, f_{pk} = 1770 MPa.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
75 kN/m
100 kN/m
150 kN/m

Single span

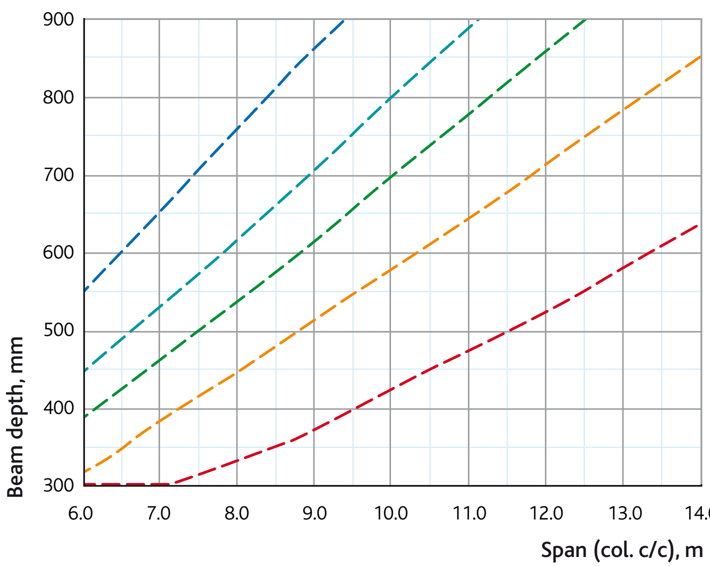
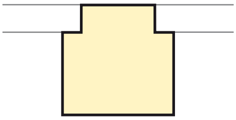


Figure 4.16
Span:depth chart for single-span rectangular precast prestressed beams, 450 mm wide

Table 4.16
Data for single-span rectangular precast prestressed beams, 450 mm wide

Span col. c/c, m (effective span, m)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)	14.0 (13.5)
Depth, mm									
uaudl = 25 kN/m	300	300	335	375	425	475	525	580	635
uaudl = 50 kN/m	320	385	450	515	580	645	715	785	855
uaudl = 75 kN/m	390	465	540	620	700	780	860	940	
uaudl = 100 kN/m	450	535	620	710	800	890	980		
uaudl = 150 kN/m	550	655	760	865	970				
Ultimate load to supports/columns, internal (end), kN									
uaudl = 25 kN/m	175 (88)	205 (102)	238 (119)	272 (136)	310 (155)	348 (174)	389 (194)	431 (216)	475 (238)
uaudl = 50 kN/m	327 (164)	388 (194)	451 (225)	515 (258)	582 (291)	650 (325)	721 (360)	794 (397)	868 (434)
uaudl = 75 kN/m	483 (241)	571 (285)	661 (330)	753 (377)	848 (424)	946 (473)	1045 (523)	1147 (573)	
uaudl = 100 kN/m	638 (319)	753 (376)	870 (435)	990 (495)	1113 (556)	1238 (619)	1365 (683)		
uaudl = 150 kN/m	946 (473)	1114 (557)	1286 (643)	1459 (730)	1636 (818)				
Reinforcement, kg/m (kg/m³)									
uaudl = 25 kN/m	7 (55)	7 (55)	8 (55)	9 (55)	11 (55)	12 (55)	13 (55)	14 (55)	16 (55)
uaudl = 50 kN/m	8 (55)	10 (55)	11 (55)	13 (55)	14 (55)	16 (55)	18 (55)	19 (55)	21 (55)
uaudl = 75 kN/m	10 (55)	11 (55)	13 (55)	15 (55)	17 (55)	19 (55)	21 (55)	23 (55)	
uaudl = 100 kN/m	11 (55)	13 (55)	15 (55)	18 (55)	20 (55)	22 (55)	24 (55)		
uaudl = 150 kN/m	14 (55)	16 (55)	19 (55)	21 (55)	24 (55)				

4.2.13 Precast prestressed inverted T-beams, single span, 600 mm overall width



Design assumptions

Design and dimensions – See Section 4.2.4.
Fire and durability – Fire resistance 1.5 hours; exposure class XC1.
ψ_2 factor – $\psi_2 = 0.8$.
Concrete – C50/60; 25 kN/m^3; 20 mm gravel aggregate.
Reinforcement – Strand, $f_{pk} = 1770$ MPa.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
150 kN/m
200 kN/m
300 kN/m

Single span

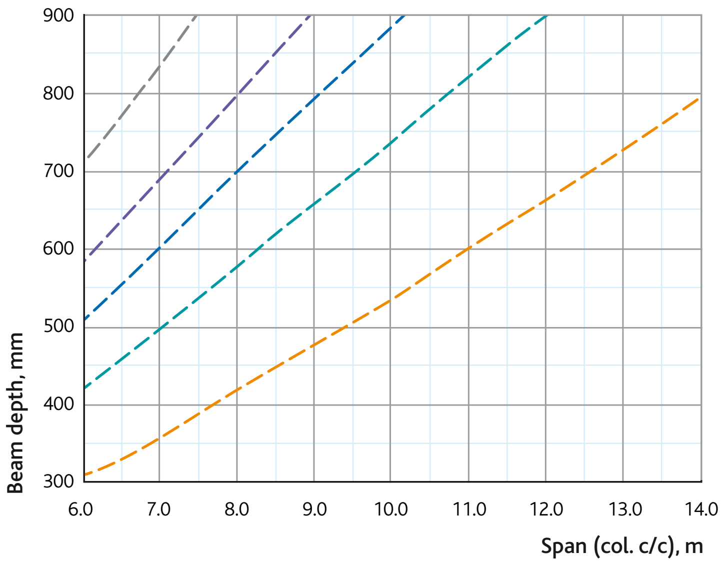
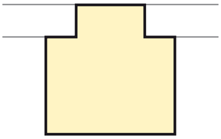


Figure 4.17
Span:depth chart for single-span precast prestressed inverted T-beams 600 mm overall width

Table 4.17
Data for single-span precast prestressed inverted T-beams, 600 mm overall width

Span col. c/c, m (effective span, m)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)	14.0 (13.5)
Depth, mm									
uaudl = 50 kN/m	310	360	420	480	540	605	665	730	795
uaudl = 100 kN/m	420	500	580	660	740	825	905	990	
uaudl = 150 kN/m	510	605	700	795	890	990			
uaudl = 200 kN/m	585	695	800	910					
uaudl = 300 kN/m	710	840	970						
Ultimate load to supports/columns, internal (end), kN									
uaudl = 50 kN/m	328 (164)	389 (195)	454 (227)	520 (260)	590 (295)	662 (331)	736 (368)	813 (406)	892 (446)
uaudl = 100 kN/m	640 (320)	757 (379)	878 (439)	1001 (500)	1127 (564)	1257 (629)	1390 (695)	1526 (763)	
uaudl = 150 kN/m	950 (475)	1121 (561)	1296 (648)	1474 (737)	1655 (828)	1841 (921)			
uaudl = 200 kN/m	1259 (629)	1483 (742)	1711 (855)	1943 (972)					
uaudl = 300 kN/m	1873 (936)	2202(1101)	2536(1268)						
Prestressing strand, kg/m (kg/m³) excluding links/carriers									
uaudl = 50 kN/m	10 (64)	11 (64)	14 (63)	16 (62)	18 (62)	20 (61)	22 (61)	24 (60)	26 (60)
uaudl = 100 kN/m	14 (63)	16 (62)	19 (61)	22 (61)	24 (60)	27 (60)	30 (59)	33 (59)	
uaudl = 150 kN/m	17 (62)	20 (61)	23 (60)	26 (60)	30 (59)	33 (59)			
uaudl = 200 kN/m	19 (61)	23 (60)	26 (60)	30 (59)					
uaudl = 300 kN/m	23 (60)	28 (60)	32 (59)						

4.2.14 Precast prestressed inverted T-beams, single span, 750 mm overall width



Design assumptions

Design and dimensions – See Section 4.2.4.
Fire and durability – Fire resistance 1.5 hours; exposure class XC1.
ψ_2 factor – ψ_2 = 0.8.
Concrete – C50/60; 25 kN/m^3; 20 mm gravel aggregate.
Reinforcement – Strand, f_{pk} = 1770 MPa.

Key
Ultimate applied uad (uaudl)
50 kN/m
100 kN/m
150 kN/m
200 kN/m
300 kN/m

Single span

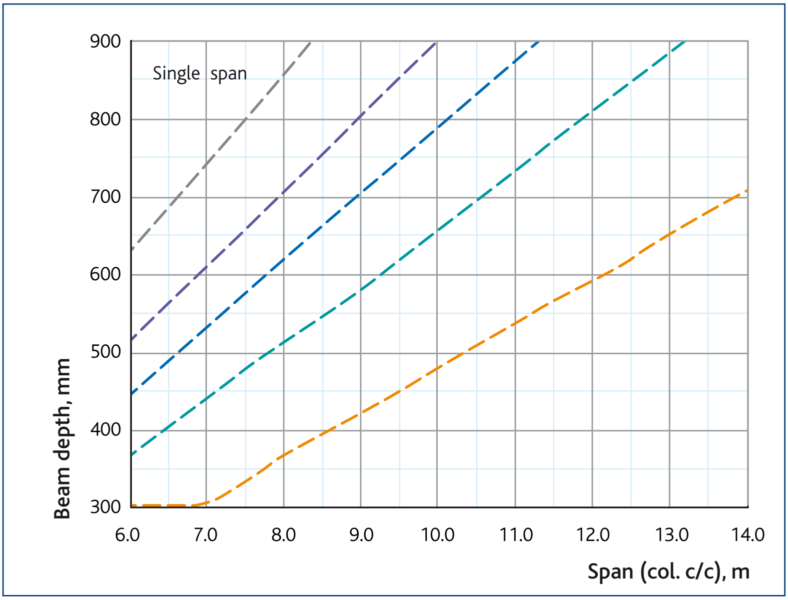


Figure 4.18
Span:depth chart for for single-span precast prestressed inverted T-beams, 750 mm overall width

Table 4.18
Data for single-span precast prestressed inverted T-beams, 750 mm overall width

Span col. c/c, m (effective span, m)	6.0 (5.5)	7.0 (6.5)	8.0 (7.5)	9.0 (8.5)	10.0 (9.5)	11.0 (10.5)	12.0 (11.5)	13.0 (12.5)	14.0 (13.5)
Depth, mm									
uaudl = 50 kN/m	300	310	370	425	480	540	595	655	710
uaudl = 100 kN/m	370	445	515	585	660	735	810	885	960
uaudl = 150 kN/m	450	535	620	705	790	875	960		
uaudl = 200 kN/m	520	615	710	810	905				
uaudl = 300 kN/m	630	745	860	980					
Ultimate load to supports/columns, internal (end), kN									
uaudl = 50 kN/m	335 (168)	393 (196)	460 (230)	529 (265)	601 (300)	676 (338)	753 (377)	834 (417)	917 (458)
uaudl = 100 kN/m	645 (323)	765 (382)	887 (444)	1013 (506)	1143 (571)	1277 (638)	1414 (707)	1554 (777)	1699 (849)
uaudl = 150 kN/m	956 (478)	1130 (565)	1307 (653)	1488 (744)	1673 (837)	1863 (931)	2056 (1028)	2255 (1127)	2458 (1229)
uaudl = 200 kN/m	1266 (633)	1493 (746)	1724 (862)	1960 (980)	2200 (1100)	2446 (1223)	2697 (1348)	2953 (1477)	
uaudl = 300 kN/m	1882 (941)	2214 (1107)	2552 (1276)	2896 (1448)	3245 (1622)	3600 (1800)			
Prestressing strand, kg/m (kg/m³) excluding links/carriers									
uaudl = 50 kN/m	12 (62)	12 (62)	15 (62)	17 (61)	20 (61)	22 (60)	24 (60)	27 (59)	29 (59)
uaudl = 100 kN/m	15 (62)	18 (61)	21 (60)	24 (60)	27 (59)	30 (59)	33 (59)	37 (58)	40 (58)
uaudl = 150 kN/m	18 (61)	22 (60)	26 (60)	29 (59)	33 (59)	36 (59)	40 (58)		
uaudl = 200 kN/m	21 (60)	25 (60)	29 (59)	33 (59)	37 (58)				
uaudl = 300 kN/m	26 (60)	31 (59)	36 (59)	41 (58)					

4.3 Precast columns

4.3.1 Using precast columns

Precast columns facilitate speed of erection by eliminating formwork and, in many cases, site-applied finishes and follow-on trades. They have inherent fire resistance, durability and the potential for a vast range of integral and applied finishes.

In multi-storey buildings, precast columns are often precast three or four storeys high, typically up to 600 mm square. The maximum sizes are usually dictated by craneage. Where splices are required in taller buildings, these are normally located at or near points of contraflexure. In smaller buildings typical precast column sizes are 300 mm square for two-storey buildings and 350 mm square for three-storey buildings. Smaller columns may be possible using higher grades of concrete and higher percentages of reinforcement than those indicated in the charts and data. In such cases reference should be made to manufacturers, as handling and connections, details of which are usually specific to individual manufacturers, may make smaller sections difficult to use. Manufacturers tend to produce preferred cross-sections based on 50 mm increments. Nonetheless, designs with other cross-sections and bespoke finishes are easily accommodated; for instance, corbels are common in precast concrete car parks.

The economics of precast construction depends on repetition. As far as possible, the same section should be used throughout.

4.3.2 The charts and data

The column charts give square sizes against axial load for internal, edge and corner braced columns, for a range of steel contents.

Knowing the total ultimate axial load (N_{Ed}), the size of internal columns can be derived from a load:size chart. The charts for edge and corner columns operate on total ultimate axial load (N_{Ed}) and 1st order design moments about the z direction (M_z). Please note that the 1st order moment, M, used should never be less than M_{min} indicated on the moment:load charts. (M_{min} = 0.02 N_{Ed} to allow for imperfections.) The ultimate axial load (N_{Ed}) should be calculated from first principles for the lowest level of column under consideration (see Section 8.4). However, it may suffice to estimate the load in accordance with Section 2.7 by summing the reactions of beams and self-weight of columns at each level; see Section 2.11.4.

The charts for edge and corner columns operate in a similar fashion to those for in-situ columns except that the user is expected to determine the 1st order design moment, M, from the eccentricity moment, eV; (see Section 4.3.3). The load:size chart for internal columns, Figure 4.19a, assumes 'normal' moments, i.e. equal adjacent spans in each direction. As an alternative, where the moment is eccentric, internal sizes may be estimated from the moment:load chart, Figure 4.19b. Some iteration on the design moment may be required. The charts and data for edge columns assume roughly equal spans in the direction parallel with the edge. If these spans are unequal by more than, say, 15%, consider treating edge columns as corner columns.

The user is expected to interpolate between values given in the charts and data and round up both the load and size derived in line with normal modular sizing and his or her confidence in the design criteria used. The thickness of any specialist finishes required should be added to the sizes given. Column design depends on ultimate axial load and ultimate design moment. Design moments are specific to a project and should not be generalised. The sizes of columns shown in the charts and data should be considered as being indicative only, until they can be confirmed at scheme design by a specialist engineer or contractor.

4.3.3 Estimating the 1st order design moment, *M*

Internal columns

The load:size chart and data for internal columns assume nominal moments only, i.e. equal beam or slab spans and equal loads in each orthogonal direction (i.e. $l_{y1} = l_{y2}$ and $l_{z1} = l_{z2}$). If spans are unequal by more than, say, 10%, then the design moment should be calculated and the load:moment chart, Figure 4.19b, should be used.

Perimeter columns

The eccentricity moment, *eV*, is caused by the beam reaction, *V*, acting at an eccentricity, *e*, from the centreline of the column, see Figure 4.E. The beam reaction should be calculated or, conservatively, the end ultimate load to support/columns from the precast beam tables may be used. The eccentricity moment should be calculated (or estimated from Figure 4.F) and distributed to the column above and below according to relative stiffnesses. This gives the 1st order design moment, *M*, about the z axis. Once axial load, N_{Ed} and *M* have been determined, the assumed size of column should be checked for suitability. The percentage of reinforcement required may be determined from the appropriate charts. Amounts of reinforcement may be determined from Figure 3.45.

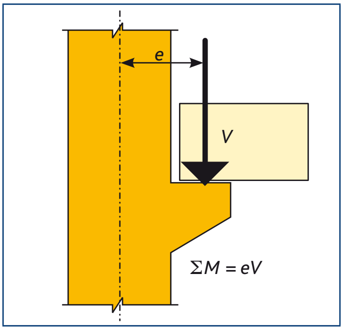


Figure 4.E
Detail at joint

Edge column charts have been adjusted to allow for biaxial bending with M_y/M_z = 0.2, and corner columns with M_y/M_z = 0.5. These ratios should generally prove adequate, but if they are exceeded, slightly larger columns may be required.

Key
Eccentricity, *e*
- 200 mm
- 300 mm
- 400 mm
- 500 mm
- 600 mm

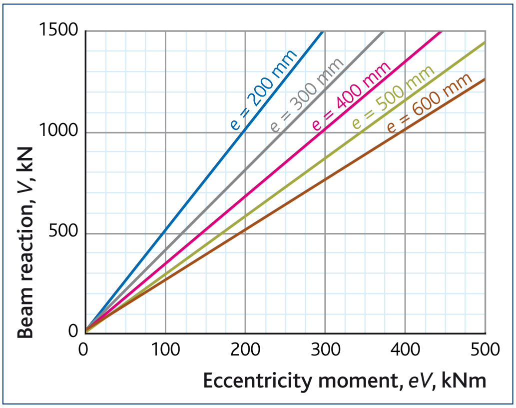


Figure 4.F
Chart to estimate eccentricity moment

4.3.4 Design assumptions

Design assumptions for precast columns are described in the relevant section. Other assumptions are given below.

Reinforcement

Main bars and links: f_{yk} = 500 MPa. Link size, maximum main bar size/4 (min. H8). Maximum bar size H32. Minimum 4 no. H12. Percentage of column area as indicated.

Column lengths

Column effective lengths have been taken as the clear height between floors (i.e. a *K* factor of 1.0). A storey height of 3750 mm has been assumed. A greater storey height will reduce moment capacity, but the applied moment will be smaller.

4.3.5 Precast internal columns

Design assumptions

Design – See Section 4.3.3.
Fire and durability – Fire resistance 1 hour; exposure class XC3 or XC4, both with XF3.
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.
Reinforcement – f_{yk} = 500 MPa. Percentage of column area as indicated.

Key
Percentage reinforcement
Min.
1.0%
2.0%
3.0%
4.0%
4.0% C50/60

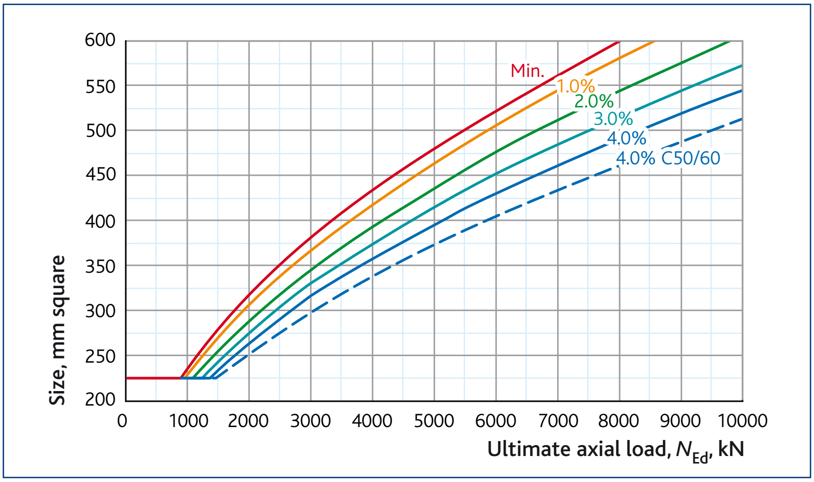


Figure 4.19a
Load:size chart for precast internal columns

Table 4.19
Data for precast internal columns

Ultimate axial load, N_{ed}, kN	1000	1500	2000	3000	4000	5000	6000	8000	10000
Size, mm square									
Min % reinf. C40/50	237	281	316	381	434	481	523	599	669
1.0% reinf. C40/50	228	270	306	367	419	464	505	580	647
2.0% reinf. C40/50	225	255	289	346	393	438	476	544	605
3.0% reinf. C40/50	225	243	275	330	375	414	451	516	573
4.0% reinf. C40/50	225	234	263	316	357	396	430	491	545
Variation									
4.0% reinf. C50/60	225	225	249	297	337	372	404	461	512

Key
Column size
225 mm sq.
300 mm sq.
400 mm sq.
500 mm sq.
600 mm sq.

Note
Use this chart if loads to internal columns are eccentric

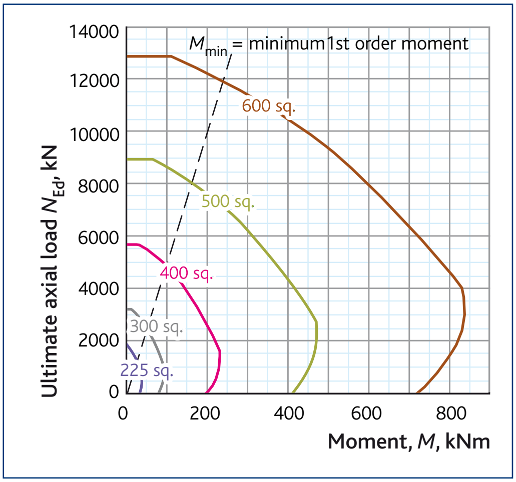
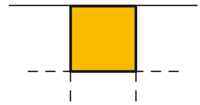


Figure 4.19b
Moment:load chart for precast internal columns with 4% reinforcement

4.3.6 Precast edge columns



Design assumptions

Design – Curves have been adjusted to allow for biaxial bending with $M_y/M_z = 0.2$, and for any additional buckling moments (2nd order). See also Section 4.3.4.

Fire resistance – 1 hour.

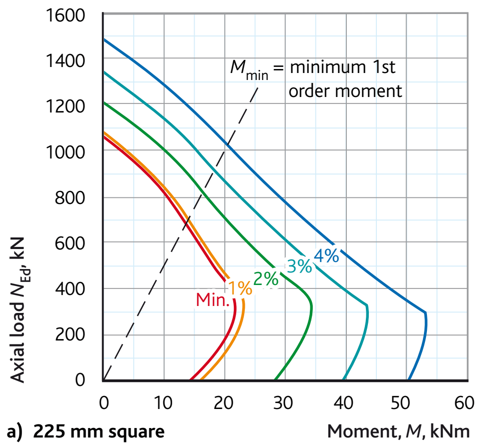
Exposure class – XC3 or XC4, both with XF3.

Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.

Reinforcement – $f_{yk} = 500$ MPa. Percentage of column area as indicated.

Key
Percentage reinforcement

Min. 2.0% 4.0%
1.0% 3.0%



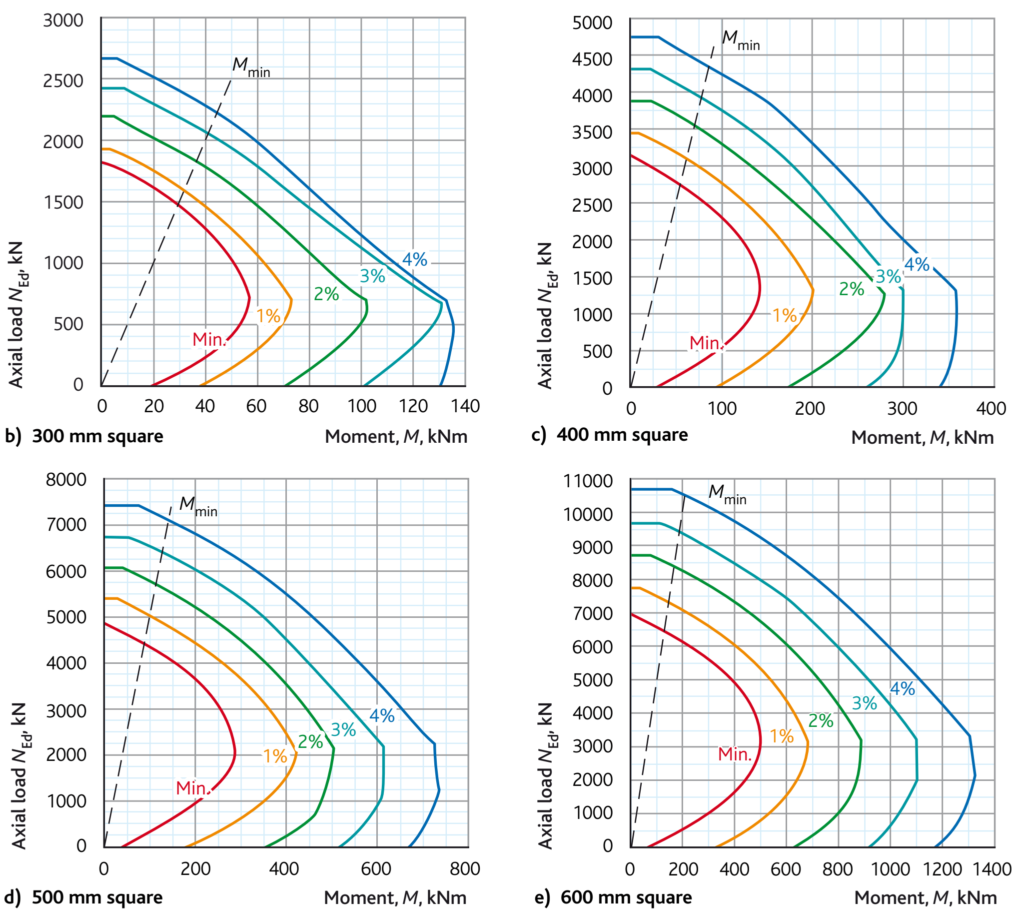


Figure 4.20
Moment:load charts for precast edge columns

4.3.7 Precast corner columns

Design assumptions

Design – Curves have been adjusted to allow for biaxial bending with $M_y/M_z = 0.5$, and for any additional buckling moments (2nd order). See also Section 4.3.3.

Fire resistance – 1 hour.

Exposure class – XC3 or XC4, both with XF3.

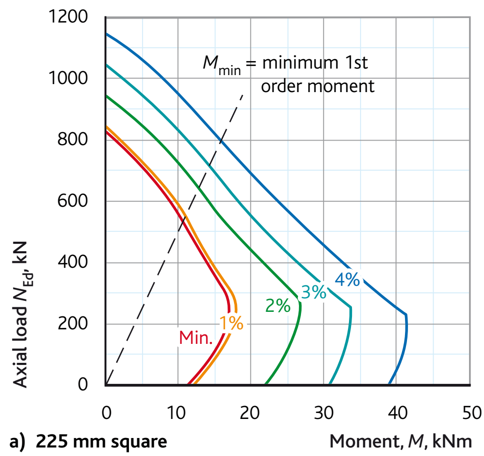
Concrete – C40/50; 25 kN/m^3; 20 mm aggregate.

Reinforcement – f_{yk} = 500 MPa. Percentage of column area as indicated.

Key

Percentage reinforcement

Min | 2.0% | 4.0%
1.0% | 3.0%



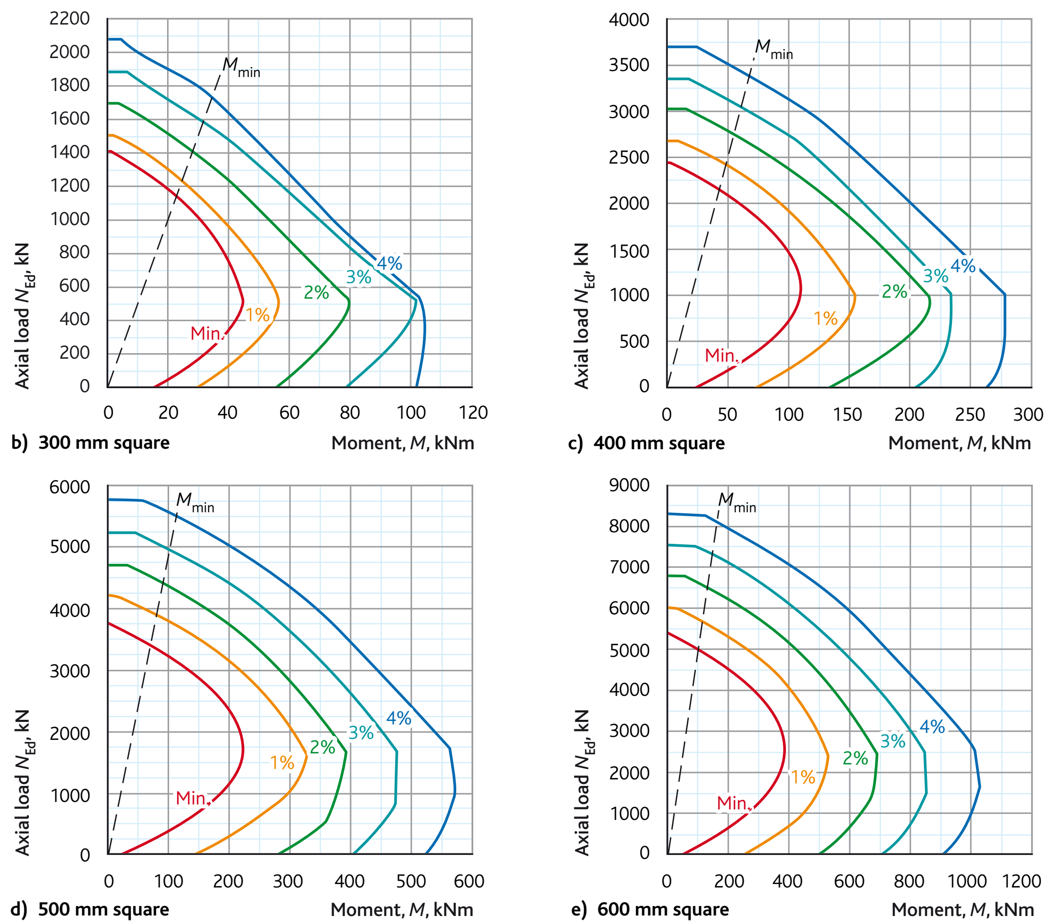


Figure 4.21
Moment:load charts for precast corner columns

5 Post-tensioned concrete construction

Figure 5.A
Cardinal Place, Victoria, London SW1 (Esso Glen) is a visually striking development on the Stag Estate. The high specification, 11-storey, commercial development uses post-tensioned upper level floor slabs.
Photo courtesy of Byrne Bros

5.1 Post-tensioning

Prestressing concrete, using tensioned high-strength steel to compress it, reduces or even eliminates tensile stresses and cracks. This gives rise to a range of benefits that exceed those found in normally reinforced concrete sections. Benefits include increased spans, stiffness and watertightness, and reduced construction depths, self-weights and deflections. Prestressing can be carried out before or after casting the concrete. Tensioning the prestressing steel before casting, (i.e. pre-tensioning) tends to be done in factories, e.g. in producing precast floor units. Post-tensioning is more usually carried on site. 'Normal' reinforcement (hereafter referred to as 'reinforcement') is used to supplement the prestressing steel and to provide against bursting forces at the anchorages.

In floors, post-tensioning is often achieved using bonded tendons in flat multi-strand ducts. Typically four or five tendons of 12.9 mm diameter (or sometimes three or four of 15.7 mm diameter) are inserted into flat galvanised metal or plastic ducts 75 mm wide and 25 mm deep. The ducts are placed in the concrete section between anchorages and to a vertical profile. Once the concrete has been cast and achieves sufficient strength, the strands in the tendons are stressed using a simple hand-held jack and anchored off. Stressing or 'transfer' may be undertaken in stages and once stressing is complete the ducts are grouted[21].

Unbonded construction is similar. Here, the tendons are typically single strand, covered in grease within a protective sheath. The tendons are cast into the concrete to a profile and anchored off, then tensioned, perhaps in stages, when the concrete has developed sufficient strength.

In beams, where the level of prestress required tends to be higher, multi-strand bonded tendons are used at close centres. Each tendon may consist of three, four or five strands of 12.9 mm diameter in round or flattened galvanised or plastic ducts. These too are cast into the concrete to a profile and tensioned once the concrete has gained sufficient strength. The strands are then anchored off and, with a bonded system, the ducts are grouted.

Post-tensioning can also be used in one-way slabs or beams, where the designer wishes to avoid large amounts of normal untensioned reinforcement.

As post-tensioned slabs and beams are relatively easy to design and construct, they are compatible with fast construction techniques. They are also safe and adaptable. Concrete Society Technical Report 43, *Post-tensioned concrete floors – design handbook* [22], gives further details of design. The Concrete Centre's publication, *Post-tensioned concrete floors* [21] gives more general guidance. For specific applications, advice should be sought from specialist engineers and contractors. For example they may be able to advise on issues such as: CDM regulations, which oblige designers to consider demolition during initial design; the effects of restraint, which need to be assessed; and the use of detailed frame analysis, which can lead to significant economies.

It is recommended that designers always check the secondary effects of prestressing, such as induced frame moments, shears, and column displacements caused by member shortening.

5.1.1 The charts and data

The charts and data for slabs in Section 5.2 cover one-way solid, ribbed and flat slabs, and assume the use of bonded tendons. They give depths and other data against spans for a range of characteristic imposed loads. An allowance of 1.5 kN/m^2 has been made for superimposed dead loads (SDL). The charts and data assume frame action with nominal 250 mm square columns at supports. Data for unbonded construction is given under *Variations* in the tables (see also Section 5.1.5).

The first set of charts for post-tensioned beams in Section 5.3 assumes 1000 mm wide rectangular beams with no flange action. Rectangular beams with other web widths can be investigated on a pro-rata basis. Charts and data for 2400 mm wide T-beams are also presented. These assume full flange action. The beam charts 'work' on ultimate applied uniformly distributed loads (uaudl) in kN/m. The user must calculate or estimate this line load for each beam considered (see Section 8.3). The user is expected to interpolate between values given in the relevant charts and data, and round up both the loads and depth in line with normal modular sizing and his or her confidence in the design criteria used.

Please note that for any given load and span, there is a range of legitimate depths depending on the assumed amount of prestress (P/A). In practice, many post-tensioned elements are designed to make a certain depth work. By way of illustration, the slab charts show a range of viable prestress or in the case of ribbed slabs, viable options. The actual numbers given in the charts and data pertain to the criteria discussed in Section 7.3. The industry standard of using C32/40 concrete has been used, although some theoretical advantage may have been derived from using a higher strength concrete.

With regard to ultimate loads to supports, the $P\Delta$ effects have been ignored. The difference in tendon height between anchorages and internal supports can lead to considerable redistribution of loads. This is discussed in detail in Section 8.3.2.

Reinforcement and tendon quantities are approximate only (see Section 2.2.4). In the tables, the values given for reinforcement densities assume nominal amounts of 'normal' reinforcement throughout the top of all spans. Data for bonded construction is given under *Variations* in the tables. See also Section 5.1.5.

5.1.2 Design assumptions

Design assumptions for the individual types of slab and beams are described in the relevant section. Other assumptions made are described and discussed in Section 7.3. For multiple spans, the data has been derived from moment distribution analysis for two or three span conditions. The charts and data assume the use of bonded tendons in flat or oval multiple strand ducts and no restraint to movement.

It has been assumed that stressing takes place from one end only. For longer spans, stressing from both ends (strands stressed from each end in turn) may prove to be more economic.

Effects of movements on other elements (e.g. columns) should be considered separately.

5.1.3 Chlorides and car parks

As Section 4.1.4 explains, in exposure classes XD1, XD2, XD3, XS1, XS2 and XS3, Eurocode 2 requires bonded tendons or ducts to be contained within 25 mm of concrete in compression. Thus, in structures exposed to these conditions, such as multi-storey car parks, the use of unbonded tendons is to be preferred. In the slab data (Section 5.2) one of the variations shows depths for the option of using unbonded tendons for an imposed load of 2.5 kN/m^2.

5.1.4 Bonded vs unbonded construction

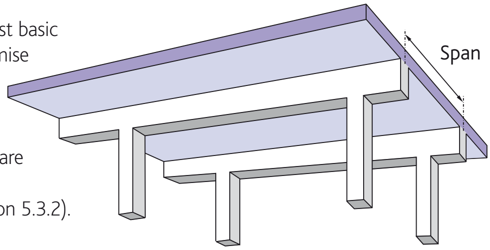
The charts and data assume the use of bonded tendons. However, they are also approximately valid for use with unbonded tendons, which is currently the less popular form of construction. When unbonded tendons are to be used, appropriate allowances have to be made as several design assumptions made in the derivation of the data for bonded tendons may become invalid (e.g. cover, effective depth, long-term stress losses). Generally sections with unbonded tendons will need slightly fewer tendons than those indicated for sections with bonded tendons. But, as shown by data for unbonded sections under *Variations*, economic depths may differ from those indicated for bonded tendons. The arguments for unbonded and bonded tendons are outlined below. (See also Zahn & Ganz, 1992[23].)

Table 5.A
Bonded versus unbonded tendons

Bonded tendons	Unbonded tendons
• Higher working stresses available • Flat duct systems allow good eccentricity • Bond forces reduce need for crack control reinforcement • Generally accommodates effects of variable pattern loading with less reinforcement • Multistrand systems allow for large forces using large units • The effects of accidental damage are localised • Does not depend on anchorages during working life • Can be demolished in the same way as reinforced concrete	• Smaller friction losses during tensioning • Smaller covers required for tendons, so providing maximum tendon eccentricity in small members • Easy to handle and place • Tendons can be prefabricated • Tendons can be deflected around obstructions easily • Corrosion protection applied at the factory • No grouting operation necessary • Demolition requires some care with detensioning

5.2 Post-tensioned slabs

5.2.1 Post-tensioned one-way solid slabs



One-way in-situ solid slabs are the most basic form of slab. Post-tensioning can minimise slab depth and control deflection and cracking. Generally used in office buildings and car parks, these slabs are economical in spans up to 12 m. They are particularly effective when used with post-tensioned band beams (See Section 5.3.2).

Advantages/disadvantages

One-way in-situ solid slabs are simple to construct, and provide robust floors with minimum overall depth. Deflection and cracking are controlled. Using wide post-tensioned band beams diminishes the effects associated with downstand beams such as slow formwork cycle times and greater storey heights.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes and reinforcement.
Fire and durability – Fire resistance 1 hour; exposure class XC2, XC3, XC4 (30 mm cover to all). Min. 37 mm cover to ducts.
Design basis – To Concrete Society Technical Report 43[22]. Transfer at 3 to 4 days. Maximum prestress (P/A) = 1.5 MPa, but limited by stresses at transfer and deflection (see Section 7.3.5). No restraint to movement. w_{max} = 0.2 mm.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Concrete – C32/40, 25 kN/m^3; 20 mm aggregate. $f_{ck(t)}$ at transfer = 20.8 MPa.
Tendons – Bonded 12.9 mm Superstrand (A_{ps} = 100 mm^2, f_{pk} = 1860 MPa) in T2 and B2. Maximum 7 no. per m width.
Reinforcement – f_{yk} = 500 MPa. Diameters as required. Main reinforcement in layers T2 and B2; distribution reinforcement in T1 and B1. Additional reinforcement at anchorages not included.

Key
Characteristic imposed load (IL)
2.5 kN/m^2
5.0 kN/m^2
7.5 kN/m^2
10.0 kN/m^2
Single span
Multiple span
Range for 5.0 kN/m^2 multiple span (P/A = 1.0–2.5 MPa)
P/A = prestressing force/area, MPa

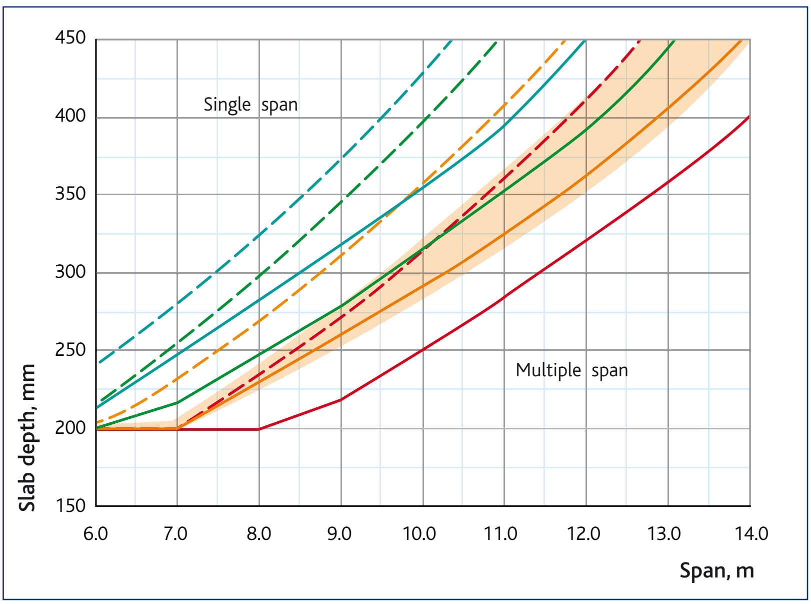


Figure 5.1
Span:depth chart for post-tensioned one-way solid slabs

Table 5.1a

Data for post-tensioned one-way solid slabs: single span

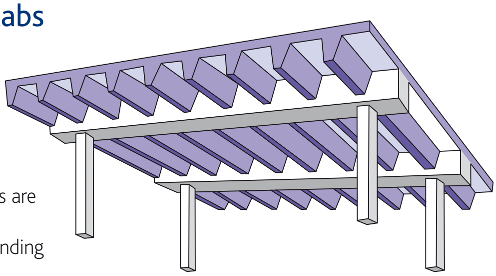
SINGLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm									
IL = 2.5 kN/m^2	200	200	234	271	314	360	414	471	
IL = 5.0 kN/m^2	200	230	269	311	358	410	467		
IL = 7.5 kN/m^2	214	254	298	346	396	454			
IL = 10.0 kN/m^2	241	282	324	373	429	488			
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	n/a (36)	n/a (42)	n/a (52)	n/a (63)	n/a (77)	n/a (93)	n/a (112)	n/a (134)	
IL = 5.0 kN/m^2	n/a (47)	n/a (58)	n/a (71)	n/a (86)	n/a (103)	n/a (122)	n/a (144)		
IL = 7.5 kN/m^2	n/a (59)	n/a (74)	n/a (90)	n/a (108)	n/a (127)	n/a (150)			
IL = 10.0 kN/m^2	n/a (75)	n/a (93)	n/a (112)	n/a (133)	n/a (158)	n/a (184)			
Reinforcement (tendons), kg/m^2 (kg/m^2)									
IL = 2.5 kN/m^2	10 (3)	15 (3)	17 (4)	21 (4)	24 (5)	27 (5)	31 (6)	35 (7)	
IL = 5.0 kN/m^2	14 (3)	17 (4)	21 (4)	24 (5)	27 (5)	31 (6)	34 (7)		
IL = 7.5 kN/m^2	16 (3)	18 (4)	22 (5)	24 (5)	28 (6)	32 (7)			
IL = 10.0 kN/m^2	19 (4)	22 (4)	27 (5)	30 (6)	35 (7)	39 (7)			
Variations: bonded tendons, overall depth, mm, for IL = 5.0 kN/m^2									
P/A = 1.0 MPa	210	252	298	351	411	470			
P/A = 2.0 MPa	200	228	263	298	335	372	430		
2 hours fire	200	235	272	314	362	412	467		
4 hours fire	216	253	293	337	384	438	496		
Variations: unbonded tendons, overall depth, mm, for IL = 5.0 kN/m^2 uno.									
Unbonded	200	237	284	326	363	415	472	535	619
Top XD3, bottom XD1, C45/55, IL = 2.5kN/m^2	233	233	233	268	306	350	397	450	509
XD1, C40/50	200	221	260	301	345	396	448	508	568

Table 5.1b

Data for post-tensioned one-way slabs: multiple span

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm									
IL = 2.5 kN/m^2	200	200	200	218	250	284	321	359	403
IL = 5.0 kN/m^2	200	200	230	260	291	323	361	405	452
IL = 7.5 kN/m^2	200	216	247	279	314	353	393	443	526
IL = 10.0 kN/m^2	212	247	282	318	354	394	450	532	
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	71 (36)	83 (42)	95 (47)	112 (56)	134 (67)	159 (80)	188 (94)	219 (109)	256 (128)
IL = 5.0 kN/m^2	94 (47)	109 (55)	132 (66)	157 (79)	185 (92)	214 (107)	248 (124)	286 (143)	329 (164)
IL = 7.5 kN/m^2	116 (58)	139 (70)	167 (83)	196 (98)	229 (115)	266 (133)	305 (152)	350 (175)	414 (207)
IL = 10.0 kN/m^2	145 (73)	178 (89)	212 (106)	250 (125)	290 (145)	334 (167)	387 (193)	455 (227)	
Reinforcement (tendons), kg/m^2 (kg/m^2)									
IL = 2.5 kN/m^2	5 (3)	7 (3)	10 (3)	12 (3)	13 (3)	15 (4)	17 (4)	18 (5)	21 (5)
IL = 5.0 kN/m^2	7 (3)	10 (3)	11 (3)	13 (3)	16 (4)	17 (4)	19 (5)	22 (6)	25 (6)
IL = 7.5 kN/m^2	8 (3)	12 (3)	14 (3)	16 (4)	18 (4)	20 (5)	22 (6)	26 (6)	31 (6)
IL = 10.0 kN/m^2	12 (3)	14 (3)	16 (4)	19 (5)	22 (5)	24 (6)	28 (6)	33 (6)	
Variations: bonded tendons, overall depth, mm, for IL = 5.0 kN/m^2									
P/A = 1.0 MPa	200	205	240	279	322	365	414	465	517
P/A = 2.5 MPa	200	200	225	255	286	318	351	396	452
2 hours fire	200	202	232	262	293	325	363	405	452
4 hours fire	248	248	248	272	307	345	383	424	483
Grade C40/50	200	200	217	245	274	308	345	386	428
Variations: unbonded tendons, overall depth, mm, for IL = 5.0 kN/m^2 uno.									
Unbonded	200	200	233	268	308	349	392	441	489
Top XD3, bottom XD1, C45/55, IL = 2.5 kN/m^2	233	233	233	238	272	307	347	387	430
XD1, C40/50	200	200	238	275	313	356	399	444	491
XD1, C40/50, IL = 2.5 kN/m^2	200	200	205	236	272	308	347	392	436

5.2.2 Post-tensioned ribbed slabs



Generally these slabs are employed in office buildings and car parks where long spans are required. Economical in spans in the range 8 m to 16 m, post-tensioned ribbed slabs are a very lightweight form of construction. Charts are based on 300 mm wide ribs, spaced at 1200 mm centres with solid areas extending up to span/9.6 from centre of supports.

Advantages/disadvantages

Compared with solid slabs, a slightly deeper section is required, but the stiffer floors facilitate longer spans and provision of holes. The soffit can be left exposed. The saving in materials is offset by the complication in formwork, reinforcement and post-tensioning operations. Sealing the tops of partitions can be difficult. These voided slabs are slower to construct but they provide a viable solution for spans between 8 m and 16 m.

Design assumptions

Supported by – Beams. Refer to beam charts and data to estimate sizes and reinforcement.
Fire and durability – Fire resistance 1 hour; exposure class XC2, XC3, XC4 (30 mm cover to all). Min. 37 mm cover to ducts.
Design basis – Transfer at 3 to 4 days. Maximum prestress, P/A = 1.5 MPa, but limited by number of tendons, and stresses at transfer and deflection. No restraint to movement. w_{max} = 0.2 mm.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Self-weight allows for slope on ribs and solid areas as indicated above. For multiple spans, results are from moment analysis for a three-span condition.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Dimensions – 300 mm ribs at 1200 mm c/c with 100 mm topping. Solid areas up to span/9.6 from supports.
Concrete – C32/40, 25 kN/m^3, 20 mm aggregate. $f_{ck(t)}$ at transfer = 20.8 MPa.
Tendons – Bonded 12.9 mm Superstrand (A_{ps} 100 mm^2, f_{pk} 1860 MPa) inside links. Max. 5 per rib.
Reinforcement – f_{yk} = 500 MPa. H8 links and weight of flange steel included.

Key
Characteristic imposed load (IL)
2.5 kN/m^2
5.0 kN/m^2
7.5 kN/m^2
10.0 kN/m^2
Single span
Multiple span
Range for 5.0 kN/m^2 multiple span
(P/A = 1.0–2.5 MPa)
P/A = prestressing force/area, MPa

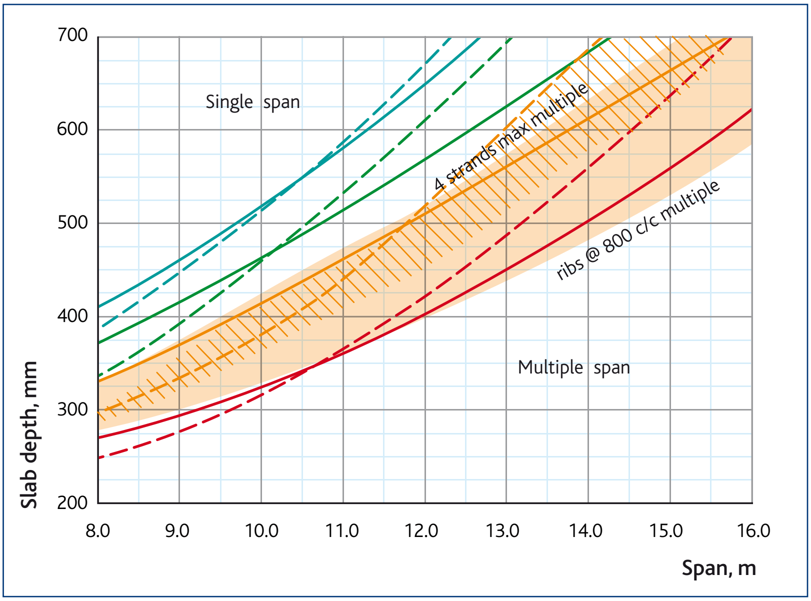


Figure 5.2
Span:depth chart for post-tensioned ribbed slabs

Table 5.2a

Data for post-tensioned ribbed slabs: single span

SINGLE span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Overall depth, mm									
IL = 2.5 kN/m^2	250	277	311	363	423	500	552	634	727
IL = 5.0 kN/m^2	298	334	383	439	519	605	691	788	845
IL = 7.5 kN/m^2	357	380	455	519	608	704	802	829	
IL = 10.0 kN/m^2	399	442	517	583	672	767	866		
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	n/a (44)	n/a (51)	n/a (59)	n/a (68)	n/a (78)	n/a (91)	n/a (103)	n/a (118)	n/a (135)
IL = 5.0 kN/m^2	n/a (61)	n/a (71)	n/a (82)	n/a (94)	n/a (108)	n/a (124)	n/a (141)	n/a (161)	n/a (178)
IL = 7.5 kN/m^2	n/a (79)	n/a (90)	n/a (105)	n/a (120)	n/a (137)	n/a (157)	n/a (178)	n/a (194)	
IL = 10.0 kN/m^2	n/a (99)	n/a (114)	n/a (132)	n/a (150)	n/a (171)	n/a (194)	n/a (219)		
Reinforcement (tendons), kg/m^2 (kg/m^2)									
IL = 2.5 kN/m^2	15 (5)	18 (5)	20 (5)	20 (5)	20 (5)	21 (5)	24 (5)	27 (5)	29 (5)
IL = 5.0 kN/m^2	23 (4)	20 (5)	21 (5)	23 (5)	24 (5)	26 (5)	28 (5)	31 (5)	38 (5)
IL = 7.5 kN/m^2	20 (4)	21 (5)	20 (5)	24 (5)	27 (5)	28 (5)	32 (5)	35 (5)	
IL = 10.0 kN/m^2	23 (4)	21 (5)	24 (5)	30 (5)	32 (5)	35 (5)	38 (5)		
Variations: bonded tendons, overall depth, mm, for IL = 5.0 kN/m^2									
Max. 4 strands/rib	298	354	418	458	535	613	699	790	890
Ribs at 800 mm c/c	284	322	365	408	453	513	578	626	714
4 hours fire	316	366	413	480	557	647	736	830	941
Variations: unbonded tendons, overall depth, mm, for IL = 5.0 kN/m^2 uno.									
Unbonded	286	329	371	401	448	493	568	645	713
Top XD3, bottom XD1, C45/55, IL = 2.5kN/m^2	250	281	313	354	406	464	513	592	667
XD1, C40/50	302	337	383	428	472	546	628	692	777

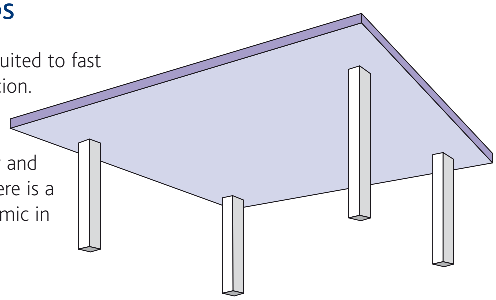
Table 5.2b

Data for post-tensioned ribbed slabs: multiple span

MULTIPLE span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Overall depth, mm									
IL = 2.5 kN/m^2	250	284	332	359	403	444	511	580	623
IL = 5.0 kN/m^2	327	375	413	445	507	580	621	675	690
IL = 7.5 kN/m^2	381	410	466	526	565	638	688	715	785
IL = 10.0 kN/m^2	400	466	510	593	639	718	801	888	
Ultimate load to supporting beams, internal (end), kN/m									
IL = 2.5 kN/m^2	89 (44)	103 (52)	120 (60)	135 (68)	154 (77)	173 (87)	198 (99)	225 (113)	249 (124)
IL = 5.0 kN/m^2	126 (63)	146 (73)	167 (84)	188 (94)	214 (107)	244 (122)	270 (135)	300 (150)	323 (161)
IL = 7.5 kN/m^2	161 (80)	184 (92)	211 (105)	240 (120)	268 (134)	302 (151)	335 (167)	364 (182)	403 (202)
IL = 10.0 kN/m^2	198 (99)	231 (115)	262 (131)	301 (150)	336 (168)	379 (189)	425 (212)	475 (237)	
Reinforcement (tendons), kg/m^2 (kg/m^2)									
IL = 2.5 kN/m^2	13 (4)	13 (4)	13 (5)	13 (5)	14 (5)	15 (5)	16 (5)	18 (5)	20 (5)
IL = 5.0 kN/m^2	15 (3)	15 (4)	15 (4)	16 (5)	16 (5)	22 (4)	20 (5)	26 (4)	26 (5)
IL = 7.5 kN/m^2	16 (3)	18 (3)	16 (5)	17 (5)	20 (5)	20 (5)	26 (4)	26 (5)	27 (5)
IL = 10.0 kN/m^2	18 (4)	18 (4)	19 (5)	20 (5)	23 (5)	26 (5)	27 (5)	30 (5)	
Variations: bonded tendons, overall depth, mm, for IL = 5.0 kN/m^2									
Max. 4 strands/rib			413	477	512	583	654	678	737
Ribs at 800 mm c/c	267	301	337	372	408	435	493	518	586
4 hours fire	384	384	419	463	505	580	643	716	790
Grade C40/C50	299	329	360	395	422	480	536	600	669
Variations: unbonded tendons, overall depth, mm, for IL = 5.0 kN/m^2 uno.									
Unbonded	304	327	351	351	369	376	419	468	511
Top XD3, bottom XD1, C45/55, IL = 2.5kN/m^2	289	327	365	398	434	464	507	575	640
XD1, C40/50	281	314	333	369	400	436	494	527	586
XD1, C40/50, IL = 2.5 kN/m^2	250	263	287	316	345	377	403	442	478

5.2.3 Post-tensioned flat slabs

Post-tensioned flat slabs are ideally suited to fast and economic multi-storey construction. Used in apartment blocks, office buildings, hospitals, hotels and other similar buildings, these slabs are easy and fast to construct especially where there is a regular column grid. Generally economic in multiple spans of 6 m to 13 m.



Advantages/disadvantages

The absence of beams allows lower storey heights and flexibility of both partition location and horizontal service distribution. It is easy to seal partitions for airtightness, acoustic isolation and fire containment. Punching shear and deflections are generally critical but edge beams to support cladding are usually unnecessary. The marking of tendon locations on soffits is usually required.

Design assumptions

Supported by – Columns. Refer to *Minimum column sizes* in data for minimum size for punching shear. Refer to other column charts and data to estimate sizes for axial load and moment.
Fire and durability – Fire resistance 1 hour; exposure class XC2, XC3, XC4 (30 mm cover to all). 37 mm cover to ducts.
Design basis – To Concrete Society Technical Report 43[22]. Transfer at 3 to 4 days. Maximum prestress (P/A) = 2.0 MPa, but limited by stresses at transfer and deflection. No restraint to movement. w_{max} = 0.2 mm.
Dimensions – Square panels, assuming three spans by three bays. Outside edge flush with columns. One 150 mm sq. hole adjacent to each internal column.
Loads – A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Perimeter load of 10 kN/m assumed.
ψ_2 factors – For 2.5 kN/m^2, ψ_2 = 0.3; for 5.0 kN/m^2, ψ_2 = 0.6; for 7.5 kN/m^2, ψ_2 = 0.6; and for 10.0 kN/m^2, ψ_2 = 0.8.
Concrete – C32/40; 25 kN/m^3; 20 mm aggregate. $f_{ck(t)}$ at transfer = 20.8 MPa.
Tendons – Bonded 12.9 mm. Superstrand (A_{ps} 100 mm^2, f_{pk} 1860 MPa) in T1, T2, B1 and B2. Max. 7 tendons per m width.
Reinforcement – f_{yk} = 500 MPa. Diameters as required. H8 links assumed. Nominal steel in top at intersection of middle strips.

Key
Characteristic imposed load (IL)
- 2.5 kN/m^2
- 5.0 kN/m^2
- 7.5 kN/m^2
- 10.0 kN/m^2
- Range for 5.0 kN/m^2 multiple span

(PA = 1.5–2.5 MPa)

P/A = prestressing force/area, MPa

Multiple span

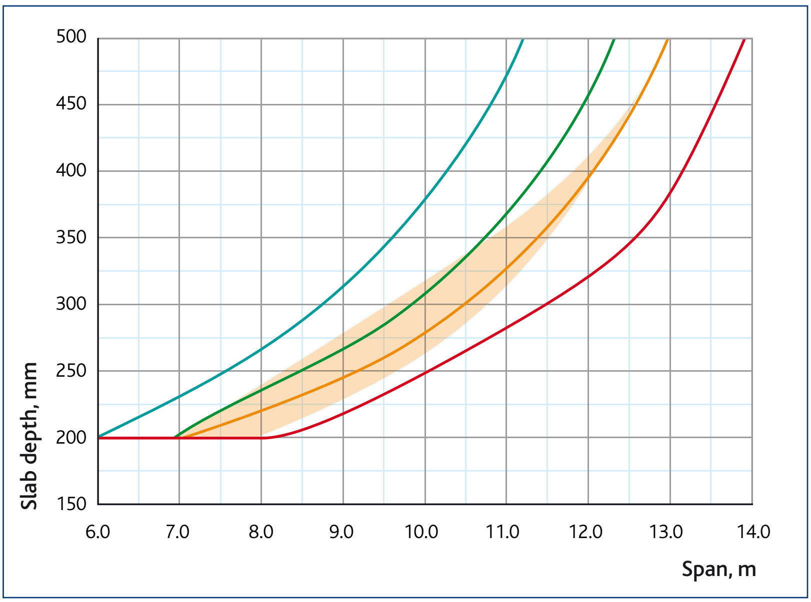


Figure 5.3 Span:depth chart for post-tensioned flat slabs: multiple span

Table 5.3
Data for post-tensioned flat slabs: multiple span

MULTIPLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm									
IL = 2.5 kN/m^2	200	200	200	217	249	283	318	387	515
IL = 5.0 kN/m^2	200	200	216	249	284	320	394	508	660
IL = 7.5 kN/m^2	200	202	235	270	307	364	464	596	
IL = 10.0 kN/m^2	200	232	270	308	378	478	604		
Ultimate load to supporting columns, kN, internal (edge*), per storey; * excludes cladding loads									
IL = 2.5 kN/m^2	430 (215)	580 (290)	760 (380)	1000 (500)	1340 (670)	1750 (875)	2240 (1120)	2990 (1500)	4320 (2160)
IL = 5.0 kN/m^2	560 (280)	765 (385)	1030 (515)	1390 (695)	1825 (910)	2340 (1170)	3120 (1560)	4260 (2130)	6220 (3110)
IL = 7.5 kN/m^2	700 (350)	950 (475)	1310 (655)	1750 (875)	2270 (1140)	2960 (1480)	3980 (1990)	5360 (2680)	
IL = 10.0 kN/m^2	856 (430)	1220 (610)	1670 (835)	2220 (1110)	2980 (1490)	4110 (2010)	5390 (2690)		
Reinforcement (tendons) kg/m^2 (kg/m^2)									
IL = 2.5 kN/m^2	5 (5)	7 (5)	7 (8)	7 (11)	8 (12)	9 (14)	11 (15)	14 (16)	20 (16)
IL = 5.0 kN/m^2	8 (5)	7 (7)	8 (11)	9 (13)	10 (14)	12 (16)	16 (16)	21 (16)	33 (16)
IL = 7.5 kN/m^2	6 (7)	7 (10)	9 (12)	13 (14)	14 (15)	18 (16)	24 (16)	30 (16)	
IL = 10.0 kN/m^2	9 (9)	11 (12)	14 (14)	16 (16)	21 (16)	26 (16)	33 (16)		
Minimum column sizes at stated slab depth, internal, mm square									
IL = 2.5 kN/m^2	280	280	343	407	461	522	582	626	664
IL = 5.0 kN/m^2	280	345	418	476	540	603	639	665	692
IL = 7.5 kN/m^2	315	415	477	545	611	659	683	703	
IL = 10.0 kN/m^2	381	452	524	597	637	668	697		
Variations: bonded tendons, overall depth, mm, for IL = 5.0 kN/m^2									
P/A = 1.5 MPa	200	204	241	280	320	363	408	507	660
P/A = 2.5 MPa	200	200	200	233	264	310	394	508	660
4 hours fire	221	227	264	302	343	385	431	539	686
Grade C40/50	200	200	210	243	277	312	366	463	588
Variations: unbonded tendons, overall depth, mm, for IL = 5.0 kN/m^2 uno.									
Unbonded	200	200	217	267	297	349	365	407	485
Top XD3, bottom XD1, C45/55, IL = 2.5kN/m^2	203	203	205	237	270	304	341	381	423
XD1, C40/50	200	200	212	246	282	318	359	401	463
XD1, C40/50, IL = 2.5 kN/m^2	200	200	200	212	245	280	317	356	398

5.3 Post-tensioned beams

5.3.1 Rectangular beams, 1000 mm wide

Prestressing of beams can provide great economic benefit for spans of 8 m to 16 m in a wide range of structures. Whilst the charts and data relate to 1000 mm wide rectangular beams, other widths can be investigated on a pro-rata basis. Post-tensioned beams are used for long spans, high loads or transferring point loads.

In line with the post-tensioned slab charts, the use of multi-strand bonded tendons in flat or oval ducts is assumed. In practice, however, consideration would also be given to using unbonded single-strand tendons in round ducts.

Advantages/disadvantages

The use of post-tensioned beams provides minimum floor depths and story heights, and means of controlling deflection and cracking. However, post-tensioning can be perceived as being a specialist operation and attention should be given to possible congestion at anchorages.

Design assumptions

Supported by – Columns.
Fire and durability – Resistance 1 hour; exposure class XC2, XC3, XC4 (30 mm cover to all).
Design basis – To Concrete Society Technical Report 43[22]. Transfer at 3 to 4 days. Maximum prestress (P/A) limited to 4 MPa or by stresses at transfer and deflection. See Section 7.3.5. No restraint to movement. w_{max} = 0.2 mm. Multiple layers with a maximum of 10 tendons per m and a maximum of 3 layers.
Loads – Ultimate applied uniformly distributed loads (uaudl) and ultimate loads to supports are per m width of beam web. Applied imposed load ≤ applied dead loads.
ψ_2 factor – Assumed ψ_2 = 0.6.
Concrete – C32/40; 25 kN/m^3; 20 mm aggregate. $f_{ck(t)}$ at transfer = 20.8 MPa.
Tendons – Bonded 12.9 mm Superstrand (A_{ps} 150 mm^2, f_{pk} 1860 MPa B2 and T2). Maximum 3 layers stressed from one end. For the same level of prestress slightly fewer unbonded tendons would be required.
Reinforcement – f_{yk} = 500 MPa. Diameters as required. Minimum H8 links.

Key
Ultimate applied udl (uaudl)
25 kN/m
50 kN/m
100 kN/m
200 kN/m
Single span
Multiple span

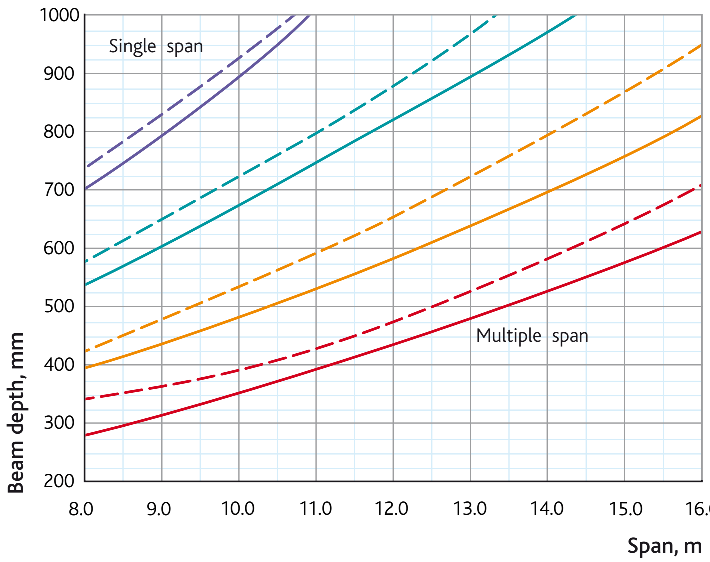


Figure 5.4
Span:depth chart for rectangular post-tensioned beams, 1000 mm wide

Table 5.4a
Data for rectangular post-tensioned beams, 1000 mm wide: single span

SINGLE span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
Overall depth, mm									
uaudl = 25 kN/m	314	329	344	365	391	426	473	524	582
uaudl = 50 kN/m	322	375	424	486	536	590	655	728	796
uaudl = 100 kN/m	425	506	582	647	713	805	878	964	1070
uaudl = 200 kN/m	548	644	737	833	928	1026			
Ultimate load to supports, each end, per metre web width, kN/m									
uaudl = 25 kN/m	104	123	143	164	186	211	239	269	302
uaudl = 50 kN/m	180	216	253	293	334	376	423	473	524
uaudl = 100 kN/m	340	405	473	541	611	688	765	846	934
uaudl = 200 kN/m	651	770	892	1017	1145	1276			
Reinforcement (tendons), kg/m^3 (kg/m^3)									
uaudl = 25 kN/m	43 (21)	42 (21)	45 (26)	49 (31)	46 (32)	49 (32)	45 (38)	47 (39)	49 (39)
uaudl = 50 kN/m	52 (31)	48 (27)	49 (27)	46 (28)	47 (31)	46 (34)	51 (33)	49 (34)	45 (37)
uaudl = 100 kN/m	55 (29)	50 (29)	46 (31)	47 (33)	51 (35)	45 (35)	50 (36)	53 (35)	51 (32)
uaudl = 200 kN/m	59 (35)	60 (31)	66 (31)	68 (30)	64 (32)	65 (33)			
Variations: bonded tendons, overall depth, mm, for uaudl = 100 kN/m									
2 hours fire	425	506	582	647	713	805	878	964	1070
$\psi_2 = 0.8$	425	506	582	647	713	805	878	964	1070
4 hours fire	444	518	600	670	743	827	914	988	1092
Variations: unbonded tendons, overall depth, mm, for uaudl = 100 kN/m									
Unbonded	447	520	605	690	761	841	928	1033	
Exp. XD1, C40/50	422	505	580	652	731	810	893	991	1096

Table 5.4b
Span:depth chart for rectangular post-tensioned beams, 1000 mm wide: multiple span

MULTIPLE span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Overall depth, mm									
uaudl = 25 kN/m	278	314	355	399	441	482	527	572	618
uaudl = 50 kN/m	389	435	491	534	584	632	697	759	819
uaudl = 100 kN/m	535	611	684	739	819	899	971	1052	
uaudl = 200 kN/m	705	797	899	1015					
Ultimate load to supporting beams, internal (end), kN/m									
uaudl = 25 kN/m	270 (135)	313 (157)	361 (180)	412 (206)	465 (233)	521 (260)	581 (290)	643 (322)	709 (355)
uaudl = 50 kN/m	497 (249)	572 (286)	653 (327)	734 (367)	819 (410)	907 (453)	1005 (502)	1106 (553)	1210 (605)
uaudl = 100 kN/m	934 (467)	1072 (536)	1214 (607)	1354 (677)	1507 (754)	1665 (833)	1825 (912)	1993 (997)	
uaudl = 200 kN/m	1776 (888)	2024 (1012)	2281 (1140)	2549 (1274)					
Reinforcement (tendons), kg/m^3 (kg/m^3)									
uaudl = 25 kN/m	50 (32)	49 (32)	47 (32)	45 (34)	46 (36)	55 (33)	52 (34)	51 (39)	55 (36)
uaudl = 50 kN/m	50 (49)	53 (49)	49 (46)	52 (46)	49 (46)	52 (46)	50 (45)	52 (43)	56 (40)
uaudl = 100 kN/m	51 (46)	51 (42)	47 (41)	53 (43)	49 (41)	52 (38)	59 (35)	64 (32)	
uaudl = 200 kN/m	76 (35)	71 (37)	66 (38)	68 (33)					
Variations: bonded tendons, overall depth, mm, for uaudl = 100 kN/m									
2 hours fire	535	611	684	739	819	899	971	1052	
4 hours fire	540	616	684	739	819	899	971	1052	
$\psi_2 = 0.8$	551	624	684	739	819	899	1009		
C40/50	505	577	635	707	771	843	937	1004	
Variations: unbonded tendons, overall depth, mm, for uaudl = 100 kN/m									
Unbonded	523	598	677	761	848	944	1043	1152	
Exp. XD1, C40/50	502	576	653	734	823	908	1000		

5.3.2 T-beams, 2400 mm wide

Together with an appropriate slab, wide, shallow, post-tensioned multiple-span T-beams maximise the benefits of minimum overall construction depth. These 'band beams' are used where long spans or irregular grids are required. They are economical for spans of 8 m to 16 m.

In line with the post-tensioned slab charts, the use of multi-strand bonded tendons in flat or oval ducts is assumed. In practice, however, consideration would also be given to using unbonded single-strand tendons in round ducts.

Advantages/disadvantages

The use of post-tensioned beams provides minimum floor depths, minimum storey heights, and a means of controlling deflection and cracking. However, post-tensioning can be perceived as being a specialist operation and attention may need to be given to possible congestion at anchorages.

Design assumptions

Supported by – Columns.
Fire and durability – Resistance 1 hour; exposure class XC2, XC3, XC4 (30 mm cover to all).
Design basis – To Concrete Society Technical Report 43[22]. 100 mm deep flange assumed. Transfer at 3 to 4 days. Maximum prestress (P/A) limited to 4 MPa or by stresses at transfer and deflection. No restraint to movement. w_{max} = 0.2 mm. See Section 7.3.5. Multiple layers with max. 16 tendons per layer.
Loads – Applied imposed load ≤ applied dead loads.
ψ_2 factor – Assumed = 0.6.
Concrete – C32/40; 25 kN/m^3; 20 mm aggregate. $f_{ck(t)}$ at transfer = 20.8 MPa.
Tendons – Bonded 12.9 mm Superstrand (A_{ps} 150 mm^2, f_{pk} 1860 MPa) B2 and T2. Maximum 3 layers stressed from one end. For the same level of prestress slightly fewer unbonded tendons would be required.
Reinforcement – f_{yk} = 500 MPa. Diameters as required. Minimum H8 links.

Key
Ultimate applied udl (uaudl)
50 kN/m
100 kN/m
200 kN/m
400 kN/m
Single span
Multiple span

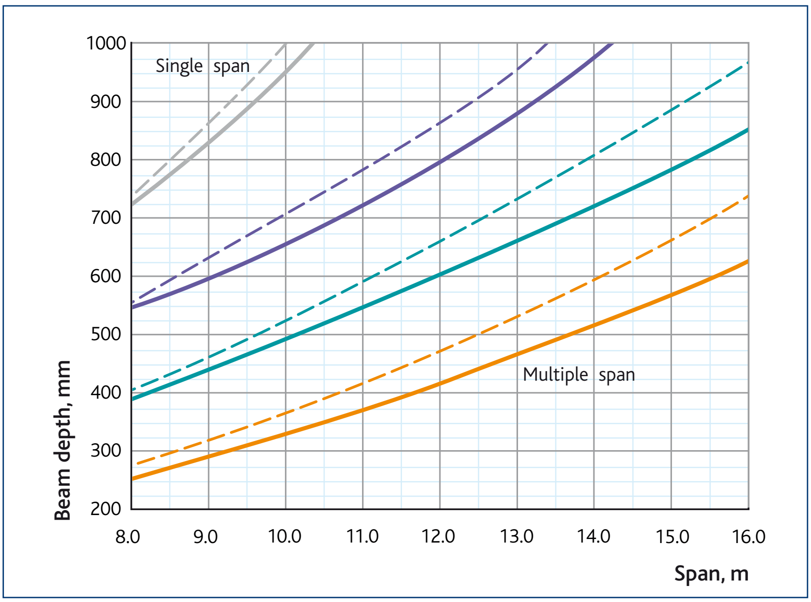


Figure 5.5
Span:depth chart for post-tensioned T-beams, 2400 mm web

Table 5.5a
Data for post-tensioned T-beams, 2400 mm web: single span

SINGLE span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Overall depth, mm									
uaudl = 50 kN/m	273	326	367	415	471	526	594	662	752
uaudl = 100 kN/m	406	464	523	591	660	744	803	896	956
uaudl = 200 kN/m	559	635	708	789	864	955	1072		
uaudl = 400 kN/m	738	864	998	1141					
Ultimate load to supports, each end, kN									
uaudl = 50 kN/m	252	301	350	405	467	533	609	691	791
uaudl = 100 kN/m	492	573	659	753	852	964	1069	1198	1314
uaudl = 200 kN/m	938	1081	1228	1384	1544	1717	1910		
uaudl = 400 kN/m	1791	2058	2337	2629					
Reinforcement (tendons), kg/m^3 (kg/m^3)									
uaudl = 50 kN/m	45 (26)	42 (23)	43 (27)	46 (28)	48 (27)	49 (27)	51 (24)	54 (21)	54 (19)
uaudl = 100 kN/m	44 (23)	44 (24)	45 (26)	48 (24)	49 (21)	52 (19)	58 (18)	60 (16)	65 (15)
uaudl = 200 kN/m	47 (23)	52 (21)	54 (20)	59 (18)	64 (16)	66 (15)	65 (13)		
uaudl = 400 kN/m	65 (19)	62 (16)	63 (14)	64 (12)					
Variations: bonded tendons, overall depth, mm, for uaudl = 100 kN/m									
2 hours fire	406	464	530	601	668	746	815	896	956
4 hours fire	430	488	550	620	680	768	843	916	976
C40/50	396	449	509	569	635	703	772	853	921
Variations: unbonded tendons, overall depth, mm, for uaudl = 100 kN/m									
Unbonded	393	454	509	566	637	709	782	868	926
Exp. XD1, C40/50	377	434	487	545	608	682	743	824	890

Table 5.5b
Data for post-tensioned T-beams, 2400 mm web: multiple span

MULTIPLE span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Overall depth, mm									
uaudl = 50 kN/m	254	288	329	372	423	466	513	566	623
uaudl = 100 kN/m	395	441	491	541	595	660	732	783	844
uaudl = 200 kN/m	538	601	669	724	792	869	972	1085	
uaudl = 400 kN/m	721	829	948	1085					
Ultimate load to supports, internal (end), kN									
uaudl = 50 kN/m	490 (245)	575 (290)	670 (335)	775 (385)	890 (445)	1010 (505)	1130 (565)	1270 (635)	1430 (715)
uaudl = 100 kN/m	975 (490)	1130 (565)	1290 (645)	1460 (730)	1650 (825)	1850 (925)	2060 (1030)	2270 (1130)	2490 (1250)
uaudl = 200 kN/m	1860 (930)	2140 (1070)	2430 (1210)	2720 (1360)	3020 (1510)	3350 (1680)	3720 (1860)	4110 (2050)	
uaudl = 400 kN/m	3570 (1790)	4090 (2050)	4640 (2320)	5210 (2610)					
Reinforcement (tendons), kg/m^3 (kg/m^3)									
uaudl = 50 kN/m	47 (28)	44 (28)	44 (30)	47 (29)	46 (27)	48 (27)	51 (27)	51 (25)	54 (23)
uaudl = 100 kN/m	44 (21)	47 (22)	48 (24)	49 (25)	50 (24)	53 (21)	54 (19)	58 (18)	59 (17)
uaudl = 200 kN/m	59 (17)	59 (19)	58 (19)	64 (19)	66 (18)	66 (16)	65 (14)	65 (13)	
uaudl = 400 kN/m	66 (19)	69 (17)	67 (15)	68 (13)					
Variations: bonded tendons, overall depth, mm, for uaudl = 100 kN/m									
2 hours fire	395	441	491	541	595	660	732	783	844
4 hours fire	395	441	491	541	595	660	732	783	844
C40/50	365	412	459	514	565	628	690	741	792
Variations: unbonded tendons, overall depth, mm, for uaudl = 100 kN/m									
Unbonded	372	399	442	485	532	587	647	711	772
Exp. XD1, C40/50	326	369	417	469	518	567	624	684	744

6 Walls and stairs

Figure 6.1
In-situ reinforced concrete walls are generally used to provide stability in addition to vertical load-bearing capacity.

6.1 Walls

Reinforced in-situ concrete walls are used principally to provide lateral stability to a structure. Whilst this publication is not intended to cover stability, the design of single plane walls is considered here briefly.

Several walls may, of course, be joined together to give, for example, L-, C-, U- or box-shapes in plan. These are stiffer than single-plane walls, but are beyond the scope of this publication. Nonetheless, the data should prove useful at the conceptual stage of design.

Walls providing lateral stability should be continuous throughout the height of the building or structure. In plan, the shear centre of the walls should coincide as much as possible with the centre of action of the applied horizontal loads (wind) in two orthogonal directions; otherwise twisting moments need to be considered.

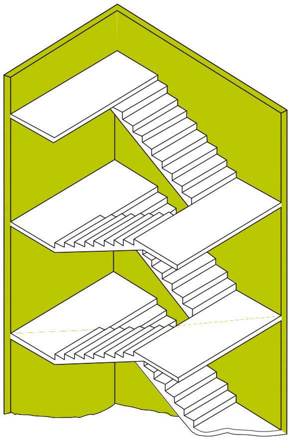
For an element to be considered as a wall, the breadth (b) must be at least four times the thickness (h). To be considered as being reinforced, a wall must have at least 0.002bh of high-yield reinforcement in the vertical direction, and horizontally, a minimum of 0.001bh or 25% of the vertical steel must be provided.

For walls required to resist fire, the maximum permitted ratio of clear height to thickness is 40. However, slender walls (l_0/b >20) should be avoided wherever possible. Derived from Eurocode 2 Section 5.8, factors for calculating the effective heights for braced columns and walls are shown in Table 6.1.

Walls that cannot go into tension do not necessarily have to be reinforced. They may be designed as 'plain' walls to Section 12 of Eurocode 2. The vertical load capacities of walls with nominal quantities of reinforcement are usually adequate for low-rise structures. Obviously the design of walls becomes more critical with increasing height of structures as both in-plane bending and axial loads increase.

6.1.1 In-situ walls

Walls should be checked for the worst combination of vertical loads, in-plane bending (stability against lateral loads) and bending at right angles to the plane of the wall (such as that induced by adjoining floors). In design, walls are usually considered to be a series of vertical strips that are designed as columns.



An effective height factor of 0.85 is commonly used for conceptual design of in-situ walls. In practice these requirements will usually result in the use of cantilever walls at least 170 mm thick in low rise multi-storey buildings. In-plane bending is often resolved into a couple that leads to additional axial forces in the extremities of the walls. The walls should be dispersed around the plan and, as far as possible, should be located in cores and stair areas and so as to minimise restraint to shrinkage. The information in Table 6.2 is given for guidance only.

Table 6.1
Effective height factors for walls

Condition	Description of condition at both ends	Factor
a	Walls connected monolithically to slabs either side that are at least 50% deeper than the wall is thick	0.67
b	Walls connected monolithically to slabs either side that are at least as deep as the wall is thick, or connected to a foundation able to carry moment	0.75
c	Walls connected monolithically to slabs either side whose depth is at least 75% of the wall thickness	0.90
d	Walls connected to members that provide no more than nominal restraint to rotation	1.00

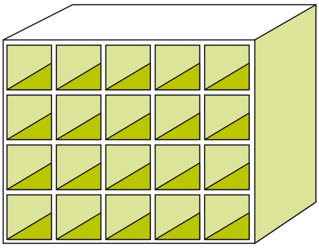
Table 6.2
Data for in-situ reinforced concrete walls

Thickness, mm	Fire period	Maximum clear height, m						ULS capacity		Reinforcement		
		Effective height factor, l_0/h						A_s, mm²/m	Capacity[a], kN/m	Typical arrangements		Density[b], kg/m³
		0.75	0.8	0.85	0.9	0.95	1.00			Vertical	Horizontal	
140	REI 60	3.73	3.50	3.29	3.11	2.95	2.80	393	1707	H10 @ 400 ef[c]	H8 @ 400 ef	43
150	–	4.00	3.75	3.53	3.33	3.16	3.00	419	1818	H10 @ 375 ef	H8 @ 400 ef	42
170	REI 90	4.53	4.25	4.00	3.78	3.58	3.40	449	1951	H10 @ 350 ef	H8 @ 400 ef	39
200	–	5.33	5.00	4.71	4.44	4.21	4.00	483	2101	H10 @ 325 ef	H8 @ 400 ef	35
220	REI 120	5.87	5.50	5.18	4.89	4.63	4.40	524	2276	H10 @ 300 ef	H8 @ 400 ef	33
270	REI 180	7.20	6.75	6.35	6.00	5.68	5.40	646	2809	H12 @ 350 ef	H8 @ 350 ef	33
350	REI 240	9.33	8.75	8.24	7.78	7.37	7.00	754	3278	H12 @ 300 ef	H10 @ 400 ef	31

Key
a Capacities are for ultimate vertical loads for the nominal reinforcement shown and nominal eccentricities only, with grade C30/37 concrete
b Includes 20% for laps and wastage etc
c ef = each face

Note
c_{nom} = 25 mm

6.1.2 Cellular structures



Tunnel form, crosswall and twinwall systems provide cellular structures suitable for residential, hotel, student, prison and other cellular accommodation. Essentially the slabs and walls are either in-situ or precast or a combination of both, and are designed in accordance with Eurocode 2.

Tunnel form

In-situ tunnel form construction creates cells which are 2.4 to 6.6 m wide. During the construction process, a structural tunnel is created by pouring concrete into steel formwork to make the floor and walls. Each 24 hours, the formwork is moved so that another tunnel can be formed. Typically 2.5 cells are created each day. The cells can be easily subdivided and longer spans (up to 11 m) are possible. The smooth face of the formwork results in a high quality finish that can be decorated directly. Particularly economic for projects with 100 cells or more, tunnel form has excellent inherent fire resistance and acoustic performance.

Figure 6.2
Student accommodation at the University of East Anglia, where tunnel form construction was used to create floors 250 mm thick. The separating walls were 180 mm thick with a 2 mm plaster skim finish. Pre-completion acoustic testing produced excellent results.
Photo courtesy of Grant Smith

Crosswall construction

Precast crosswall construction provides an efficient frame resulting in a structural floor zone 150 to 200 mm deep. Load-bearing walls perpendicular to the façade provide the means of primary vertical support and lateral stability. Longitudinal stability is achieved by external walls, panels or diaphragm action taken back to the lift cores or staircases. Structures up to 16 storeys have been completed in the UK using this system. Crosswall is fast to erect, durable and also has excellent inherent fire resistance and acoustic performance.

Figure 6.3
University of West of England, Bath, uses precast crosswall construction that may also be used for hotels and residential developments.
Photo courtesy of Buchan

Twinwall

Twinwall construction is a walling system that combines the speed of erection and quality of precast concrete with the structural integrity of in-situ concrete to provide a hybrid solution. The prefabricated panels comprise two slabs separated and connected by cast-in lattice girders. The units are placed, temporarily propped, then joined by reinforcing and concreting the cavity on site. Twinwall is usually employed in association with precast flooring systems.

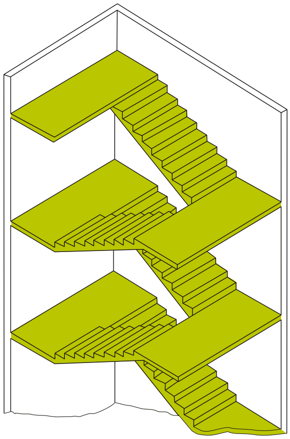
The panels are manufactured to exacting tolerances, have a high quality finish, and can incorporate cast-in cable ducts, electrical boxes and service ports. Installation rates are of up to 100 m^2 per hour. Twinwall has excellent inherent fire resistance and acoustic performance.

Figure 6.4
Twinwall combines precast with in-situ concrete to provide an ideal hybrid solution for hotels and residential units or, as pictured, in a basement.
Photo courtesy of Hanson

Sizing

The slab and wall thicknesses indicated in Tables 3.1a) & b), 4.1a) and b) and 6.2 may be used in initial scheme designs to size slabs and walls. However, acoustic considerations may govern thicknesses (see Section 9.4), and for the precast and hybrid solutions particularly, thicknesses should be confirmed with specialists at the earliest opportunity.

6.2 Stairs



There are many possible configurations of stair flights, landings and supports. The charts and data consider in-situ or precast concrete in parallel flights.

In-situ spans may be considered as being simply supported or continuous – depending upon the amount of continuity available. As illustrated in Figure 6.5, the span of the stair flight may be the flight only, or from landing to landing or through landings. Precast flights are usually considered as simply supported.

In-situ stairs provide robustness and may be useful to give continuity of work for formworkers. Precast stairs provide quality, speed of construction and early access.

The thickness of landings may be assessed from Figure 6.7 using ultimate loads to supports from the stairs data in Table 6.3. Generally, landings are treated as solid slabs.

Design assumptions

Supported by – Beams, walls or landings.
Dimensions – Flight assumed to be 70% of span, with a going of 250 mm and a rise of 180 mm. Span depends on support conditions as shown in Figure 6.5.
Fire and durability – Fire resistance 1 hour; exposure class XC1.
Loads – Superimposed load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) included. Ultimate loads assume elastic reaction factors of 0.5 to supports of single spans and of 1.1 and 0.46 to supports of continuous spans.
Imposed loads – 2.0 kN/m^2 for self-contained dwellings; 4.0 kN/m^2 for hotels, offices and other institutional buildings.
Concrete – C30/37; 25 kN/m^3; 20 mm aggregate.
Reinforcement – H10 to H16 as required with H10 distribution steel. Laps and bends included.

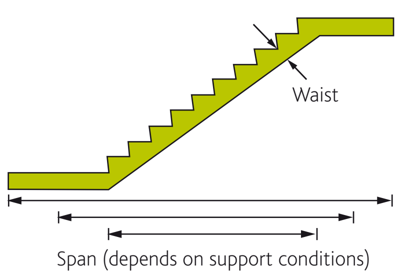


Figure 6.5
Spans of stair flights depend on configuration and available continuity

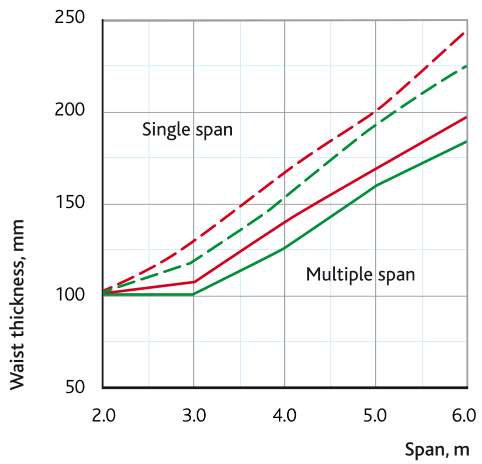


Figure 6.6
Stairs – span: waist thickness chart

Key
Characteristic imposed load (IL)
2.0 kN/m^2 — Single span
4.0 kN/m^2 — Multiple span

Table 6.3
Data for stairs

Configuration	Single spans, m					Multiple spans, m				
	2.0	3.0	4.0	5.0	6.0	2.0	3.0	4.0	5.0	6.0
Waist thickness, mm										
IL = 2.0 kN/m^2	100	119	153	194	226	100	100	126	159	183
IL = 4.0 kN/m^2	100	128	168	201	243	100	107	140	168	196
Ultimate load to internal (end) supports, kN/m										
IL = 2.0 kN/m^2	n/a (10)	n/a (16)	n/a (24)	n/a (34)	n/a (45)	12 (5)	18 (8)	27 (11)	37 (15)	51 (21)
IL = 4.0 kN/m^2	n/a (13)	n/a (21)	n/a (31)	n/a (42)	n/a (55)	15 (6)	24 (10)	35 (14)	46 (19)	59 (25)
Reinforcement, kg/m²										
IL = 2.0 kN/m^2	10	13	16	20	23	7	9	11	14	16
IL = 4.0 kN/m^2	13	17	19	21	27	9	12	13	14	18
Variations: waist thickness for IL = 4.0 kN/m^2										
R120	120	134	169	205	243	120	120	140	168	196
R240	175	175	196	234	271	175	175	175	195	224
XC3/4	102	137	174	208	249	100	117	145	171	202

Table 6.4
Data for landings

Configuration	Single spans, m				Multiple spans, m			
	2.0	3.0	4.0	5.0	2.0	3.0	4.0	5.0
Landing depth, mm								
uaudl = 10 kN/m	100	120	157	195	100	102	128	159
uaudl = 20 kN/m	100	133	172	209	100	111	141	173
uaudl = 30 kN/m	101	139	180	219	100	117	147	180
uaudl = 40 kN/m	106	147	186	232	100	121	155	186

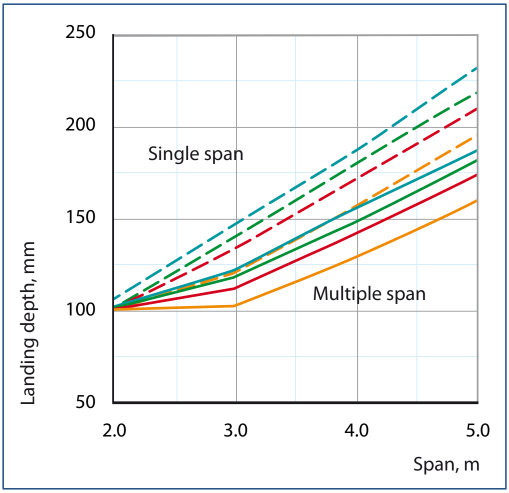


Figure 6.7
Landings – span:depth chart

Key
Ultimate applied udl (uaudl)
10 kN/m
30 kN/m
Single span
20 kN/m
40 kN/m
Multiple span

7 Derivation of charts and data

7.1 In-situ elements

7.1.1 Optimisation of depth

For a given load and span, slabs (or beams) can be designed at different depths. Thinner slabs require proportionally more reinforcement, but use less concrete, less perimeter cladding and require less support from columns and foundations. Each of these items can be ascribed a cost. The summation of these costs is a measure of overall construction cost. The minimum overall cost can be identified by designing an element at different depths and pricing the resulting quantities using budget rates and comparing total costs. In order to derive the charts and data in this publication, this process was automated using computer spreadsheets (derivatives of the RC Spreadsheets[24]).

For a particular span and load, elements were designed in accordance with BS EN 1992–1–1: 2004[2] and its UK National Annex[2a]. Unit rates were applied to the required quantities of concrete, reinforcement and formwork. Allowances were made for the costs of perimeter cladding and supporting the element's self-weight. The resulting budget costs were summed and the most economic valid depth identified, as illustrated in Figure 7.1.

This figure was based on the use of solid flat slabs on a 7.5 m square grid, with an imposed load of 5.0 kN/m^2, a superimposed dead load of 1.5 kN/m and allowance of 10 kN/m for perimeter cladding. A thickness of 242 mm appears to give best overall value. The data for this depth would have been identified and saved. Data for different spans and loads, and different forms of construction were obtained in a similar manner. This body of data forms the basis for all the information in this publication.

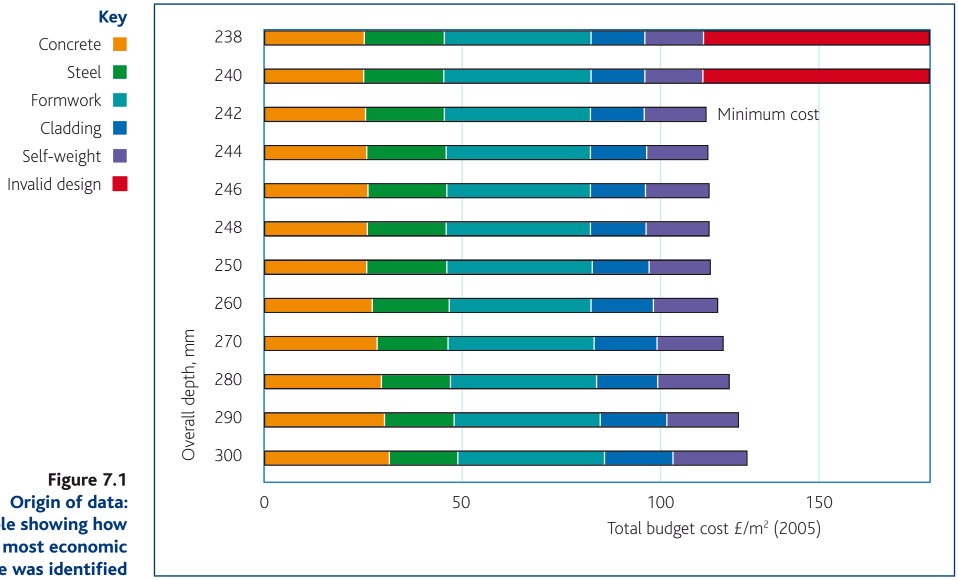


Figure 7.1
Origin of data: example showing how the most economic size was identified

The use of 1 mm increments in the data is not intended to instil some false sense of accuracy into the figures given. Rather, the user is expected to exercise engineering judgement and round up both loads and depths in line with normal modular sizing and his or her confidence in the design criteria being used. Thus, rather than using a 242 mm thick slab, it is intended that the user would actually choose a 250, 275 or even a 300 mm thick slab, confident in the knowledge that a 242 mm slab would work. Going up to, say, a 300 mm thick slab might add 10% to the overall cost of structure and cladding but might be warranted in certain circumstances.

In the example shown in Figure 7.1, the economic depth also happens to be the minimum depth that will produce a valid design. This is by no means always the case: often the economic depth is greater than the minimum depth but it would be unwise to choose a depth below the economic depth given.

The budget rates used in the optimisation are shown in Table 7.1

Table 7.1
Budget rates used for cost optimisation

Item	Cost
Concrete C30/37	£105/m^3
Horizontal formwork (plain)	£35/m^2
Horizontal formwork (ribbed)	£48/m^2
Vertical formwork	£36/m^2
Cladding	£330/m^2
Reinforcement	£800/tonne
Post-tensioning tendons	£4000/tonne
Allowance for self-weight	£2.50/kN

These rates have been arrived at via a limited industry survey undertaken in 2005, and are an update to the rates that were originally taken from the RCC's *Cost model study*[25], published in 1993. The rates have changed since 1993 and indeed they have changed since 2005 and will undoubtedly date further. However, the optimization process used in the derivation of the charts is not overly sensitive to actual rates or relative differences in rates. For instance using curtain wall cladding at, say, £750/m^2, would make little difference to the chart or data for flat slabs (but would probably improve the relative economics of using flat slabs compared with other forms of in-situ construction).

In some instances, had the optimisation process been carried out using concrete, reinforcement and formwork alone, slightly larger slab and beam sizes with lower amounts of reinforcement would have been found. However, whilst the concrete superstructure costs would have been less, the aggregate cost of the building, including cladding, foundations and vertical structure, would have been greater.

The allowance for self-weight is a measure of the additional cost in columns and foundations to support an additional 1 kN in slabs or beams. The figure used is derived from The Concrete Centre's *Cost model studies* [26] and is based on the difference in terms of £/kN of supporting three storeys rather than seven storeys. The foundations were assumed to be simple pad foundations (safe bearing pressure 200 kN/m^2). Using a higher cost per kN to allow for piling, rafts or difficult ground conditions would tend to make thinner slabs theoretically more economic, but would make their design more critical.

Project durations and differences attributable to different types of construction tend to be project specific and are difficult to model. Time costs were therefore not taken into account in the optimisation process.

It should be noted that, unlike BS 8110[5], the Eurocode 2[2] span-to-depth ratio method is not very responsive to large increases in tension reinforcement over that required at ULS. Nor is the method responsive to adding steel in the compression zone. However, deflection calculations can show that compression reinforcement does help to control deflection. Further it should be noted that economic depths can be susceptible to sudden changes in bar size or bar spacing with Eurocode 2 appearing to favour smaller bars at closer centres.

7.1.2 Design assumptions

Analysis

Apart from the data for two-way slabs on beams, which are based on yield-line tables, all charts and data tables have been generated by spreadsheets carrying out a full analysis at both ULS and at SLS. Three equal spans have been analysed to produce the multiple span charts, with 15% redistribution at internal supports. The condition where the central span was only 85% of the end (and declared) span was also checked. End spans or penultimate supports were normally found to be critical.

For slabs generally, knife-edged supports have been assumed. However, for beams, frame action has been assumed by considering that columns at supports form sub-frames: minimally sized columns were used (250 mm sq.)

Loads

In accordance with BS EN 1990[9] and its National Annex[9a] the worse case of using Expressions (6.10a) or (6.10b) is used in the derivation of slab and beam charts and data.

Partial factors for materials

Table 7.2 gives the partial factors for materials used in this publication.

Table 7.2
Partial factors for materials

Design situation	γ_C – for concrete	γ_S – for reinforcing and prestressing steel
ULS – persistent and transient	1.50	1.15
SLS	1.00	1.00

Concrete

With the exception of post-tensioned beams and slabs, all in-situ elements are assumed to use grade C30/37 concrete (f_{ck} = 30 MPa) and high yield steel (f_{yk} = 500 MPa). For post-tensioned beams and slab, the industry standard of grade C32/40 has been used (although grade C40/50 concrete may be found to give, in theory, more economic 'solutions'). Higher grades of concrete have been assumed for precast elements. Options on using higher grades of concrete are given in the data or charts, notably the charts for flat slabs and for in-situ columns.

A concrete density of 25 kN/m^3 to BS EN 1991–1–1[6] has been used.

As lightweight concretes are not always readily available, they were not considered for this publication; nonetheless, they might be an ideal solution for a particular project. Generally the structural provisions for lightweight concrete in Eurocode 2 are less onerous than those in BS 8110.

Exposure

As a minimum, internal exposure conditions (exposure class XC1) and a fire resistance of 1 hour (R60) have been assumed for the main charts and tables. Data for other exposure classes are given under *Variations*. For precast and post-tensioned elements, the higher grade of concrete required may already satisfy exposure requirements without any further adjustment.

Deflection

In many cases, particularly with slabs, deflection is critical to design. Span:depth ratio (L/d) checks in accordance with BS EN 1992–1–1, Cl. 7.4, have been used in the derivation of data. Additional tension reinforcement was provided to reduce the service stress due to quasi -permanent loads, σ_s, and thereby increase the permissible L/d ratio by 310/σ_s in accordance with Cl. 7.4.2(2). In most cases, σ_s was calculated from first principles. In accordance with Note 5 of Table NA.5 of the UK NA [2a], the ratio for $A_{s,prov}$ /$A_{s,req}$ has been restricted to 1.5: in effect this limited the factor 310/σ_s to 1.5.

As noted above, the charts and data in this publication have been derived using the span:depth method given in Eurocode 2[2] and the UK NA[2a] . The depths may be reproduced by using the TCC series in RC spreadsheets[24].

However, it is important to differentiate between the methods used in checking deformation as they will each give different answers. Three popular methods are discussed below in order of effort required.

The Concrete Centre method[7, 19]

The in-service stress of reinforcement, σ_s is used to determine a factor $310/\sigma_s$, which is used to modify the basic span:effective depth ratio as allowed in Cl.7.4.2(2) of BS EN 1992-1-1[2] and moderated by the National Annex[2a]. This method, highlighted as factor F3 in *Concise Eurocode 2*[7], was intended to be used in hand calculations to derive (conservative) values of σ_s from available ULS moments.

$$\sigma_s = (f_{yk}/\gamma_s)\ (w_{qp}/w_{ult})\ (A_{s,req}/A_{s,prov})\ /\Omega$$

where

σ_s = in-service stress of reinforcement

f_{yk} = characteristic strength of reinforcement = 500 MPa

γ_s = partial factor for reinforcement = 1.15

w_{qp} = quasi-permanent load (UDL assumed)

w_{ult} = ultimate load (UDL assumed)

$A_{s,req}$ = area of reinforcement required

$A_{s,prov}$ = area of reinforcement provided

Ω = redistribution ratio

RC Spreadsheets method[24]

The RC spreadsheets TCCxx.xls[24] use the span:depth method of checking deformation but use an accurate method for determining σ_s, which again is used to determine the moderating factor = $310/\sigma_s$.

The spreadsheets undertake separate analyses using quasi-permanent loads. For each span, an SLS neutral axis depth is determined and σ_c and σ_s are derived for the quasi-permanent load conditions. σ_s is used in accordance with BS EN 1992-1-1[2] and the current National Annex[2a], to modify the basic span:effective depth ratio. **The data in this publication is derived in this way.**

In the analysis of slabs, supports are assumed to be pinned but in reality some continuity, especially at end supports, will exist. Nominal top steel is assumed in the top of spans and is used in the determination of section properties.

Rigorous analysis

Rigorous analysis, such as that used in the series of RC Spreadsheets TCCxxR.xls, may be used to assess deformation in accordance with BS EN 1992-1-1, Cl. 7.4.3.

In these spreadsheets, sections at 1/20th points along the length of a span are checked to determine whether the flexural tensile stress in the section is likely to exceed the tensile strength of the concrete during either construction or service life: separate analyses are undertaken using frequent loads, quasi-permanent and temporary loads. Where flexural tensile strength is exceeded, the section is assumed to be cracked and remain cracked: cracked section properties are used to determine the radius of curvature for that 1/20th of span. If flexural tensile strength is not exceeded, uncracked section properties are used.

Radii of curvature are calculated for each 1/20th span increment of the element's span using the relevant properties and moments derived from analysis of quasi-permanent actions. Deformation is calculated from the increments' curvatures via numerical integration over the length of each span.

The method is in accordance with Concrete Society Technical Report 58[27].

During 2008, it became increasingly apparent (see Vollum[28]) that there is a disparity between the results given by the rigorous calculation method (method 3 above) and span:depth methods (methods 1 and 2 above) described in Eurocode 2.

In a similar manner to that experienced using BS 8110-2, using this rigorous method gives deflections that are far greater than would be expected from the assumptions stated for L/d methods, that is deflection limits of $L/250$ overall (see Cl. 7.4.1(4)) or $L/500$ after construction (see Cl. 7.4.1(5)). It is suspected that this disparity is the same as that experienced between span:depth and calculation methods in BS 8110. This disparity was recognised as long ago as 1971[29]. The rigorous method relies on many assumptions and is largely uncalibrated against real structures. As noted in Concrete Society Technical Report 58, there is an urgent need for data from actual structures so that methods may be calibrated.

With reference to his version of the rigorous method applied to continuous one-way slabs, Vollum[28] indicated that The Concrete Centre method results in deflections close to the stated Eurocode 2 limits but that slab thicknesses are only justifiable and economic if a nominal end-support restraining moment is present (where none is assumed in analysis). This situation may be addressed by ensuring that appropriate amounts of reinforcement are provided at end supports.

It should be noted that Vollum's observations were made using frequent loads where, in accordance with Eurocode 2, quasi-permanent loads are called for. Nonetheless, the NDP for Cl. 9.2.1.2(1) in the UK NA stipulates that 25% of end span moment should be used to determine end support reinforcement. This is usually accommodated by providing 25% of end span bottom steel as top steel at end supports. It is on this basis that the span charts and data in this publication are considered as being further substantiated.

Two spans

The charts and data for multiple spans assume a minimum of three spans. Unless subjected to more than 15% redistribution of support moments, two-span slab elements will be subject to greater support moments and shears than those assumed. Nonetheless, the sizes given in the charts and data can be used with caution for two-span conditions unless support moment or shear is considered critical. In this case two-span slabs should be justified by analysis and design. However, data for two-span lattice girders is given in Section 7.2.1.

Other assumptions made in the design spreadsheets are described more fully below and within the charts and data. The implications of variations to some of these assumptions are covered in the data. Other limitations of the charts and data, especially accuracy of reinforcement quantities, are covered in Section 2.2.

7.1.3 Slab charts and data

Slab charts give cost-optimised overall depths against spans for a range of characteristic imposed loads. An allowance of 1.5 kN/m^2 has been made for superimposed dead loads (finishes, services, etc.). For two-way slab systems (i.e. flat slabs, troughed slabs and waffle slabs), the design thickness and reinforcement quantities allow for a perimeter cladding load of 1.0 kN/m (but cladding loads are excluded from ultimate loads to perimeter columns).

The data accompanying each chart gives tabulated information on the economic overall depth, ultimate loads to supports and estimated reinforcement densities in kg/m^3 and kg/m^2 (see Section 2.2.4). Ultimate loads to supports include self-weight and assume a minimum of three spans with elastic reaction factors of 1.0 to internal supports and 0.5 to end supports.

Because of the way in which the data have been derived, the results are entirely valid only when the following conditions are verified:

- For multiple spans, the shortest span is at least 85% of the longest.
- The more onerous of BS EN 1990[9] loading Expressions (6.10a) and (6.10b) are applied throughout. For smaller imposed loads, Expression (6.10b) normally controls.
- Fixed values for ψ_2 (quasi-permanent proportion of imposed load) have been assumed. These values have been assumed to be:
  - 0.3 when IL ≤ 2.5 kN/m^2
  - 0.6 when 2.5 < IL ≤ 7.5 kN/m^2
  - 0.8 when 7.5 > IL ≤ 10 kN/m^2
  - (See Section 8.1)
- The characteristic imposed load, q_k, does not exceed 10 kN/m^2.

Early studies on whether to adopt BS EN 1990[9] Expression (6.10) or the more onerous of Expressions (6.10a) and (6.10b) suggested that adopting Expression 6.10 would lead to one-way slabs up to 2% thicker and 3% more expensive, and flat slabs up to 5 % thicker and 4% more expensive.

Flat slabs

For flat slabs, there should be at least three rows of panels of approximately equal span in each direction. If design parameters stray outside these limits, the sizes and data given should be used with caution. Unlike other two-way spanning slab systems, deflection checks on flat slabs are based on the longer span, so economic thicknesses are not affected by panel aspect ratio. In square bays, deflection in the edge strip in the B2 direction is critical.

Within the spreadsheet for flat slabs the equivalent frame method was used. Columns were assumed to be below only and their assumed sizes were as given in the data (generally column dimension = span/20). In accordance with BS EN 1992–1–1 Annex Cl. I.1.2(5), the end support moment was restricted to $M_{t,max}$ (= $0.17 b_e d^2 f_{ck}$), based on the column size noted. In normal situations this might lead to a possible consequential increase in end span moments. However, for the purposes of creating the flat slab data, 30 m long columns were assumed. This was done in order to minimise end support moments and so maximise end span moments in order to remain within the spirit of 'nominal end-support restraining moment (where none is assumed in analysis)' (See section 7.1.2).

It should be noted that the equivalent frame method is suitable for regular layouts but requires engineering judgment for application to irregular layouts. It is acknowledged that there are other methods of design such as those discussed in Concrete Society Technical Report 64[30]. Considering columns above and below normally would have made little difference as moment transfer at the end supports, is normally governed by $M_{t,max}$.

With respect to punching shear, β factors were calculated in accordance with Cl. 6.4.3(4) of Eurocode 2. In order to reflect typical construction, a 150 mm square hole was allowed for adjacent to each column. Consequently, punching shear, particularly around perimeter columns became critical and it was considered necessary to impose some upper limit on shear stress. Thus, a maximum of eight link perimeters was allowed and the ratio v_{Ed}/v_{Rd} was limited to 3.0* In some cases perimeter column head reinforcement was increased in order to enhance v_{Rd}. Checks indicated that the punching shear reinforcement required using βv_{Ed} calculated using minimised support moments equated to that required using βv_{Ed} calculated using a support moment of $M_{t,max}$ and consequent increase in v_{Rd}.

Flat slabs with heads

Generally slab data was derived in the same way as that for flat slabs. Sizes were found to be critical on punching shear at perimeter locations.

Flat slabs with drops

Flat slabs with drops were not included as they were regarded as being relatively uneconomic.

Ribbed slabs

For ribbed slabs it should be noted that in accordance with BS EN 1992–1–1[2], its UK NA[2a] and PD 6687[31], *L*/*d* ratios are based on ρ, which is defined as being the area of reinforcement required divided by the area of concrete above the centroid of the tension steel. Therefore ρ is higher than the equivalent in BS 8110[5] where traditionally the denominator was *bd*. Consequently *L*/*d* ratios are smaller and ribbed slabs are apparently less economic using design to BS EN 1992–1–1.

Reinforcement densities

Reinforcement densities assume that the areas or volumes of slabs are measured gross, e.g. slabs are measured through beams and the presence of voids in troughed and waffle slabs is ignored.

7.1.4 Beam charts and data

The beam charts and data give overall depths against span for a range of ultimate applied uniformly distributed loads (uaudl, see Section 8.3) and web widths. For multiple spans, the sizes given result from considering the end span of three. The charts and data were derived using essentially the same optimization process as for slabs.

Because of the way in which the data have been derived, the results are entirely valid only when the following conditions are met:

- Characteristic imposed loads, Q_k, do not exceed characteristic dead loads, G_k.
- Loads are substantially uniformly distributed over three or more spans.
- ψ_0, the combination value of ψ for imposed load, used in Expression (6.10a) = 0.7.
- ψ_2, the quasi-permanent value of ψ for imposed load, = 0.6.
- The more onerous of BS EN 1990 loading Expressions (6.10a) and (6.10b) are applied throughout.
- Variations in span length do not exceed 15% of the longest span.

Where the design parameters stray outside these limits, the sizes and data given should be used with caution. Where Expression (6.10) is to be applied and/or storage loads are envisaged ψ_0 = 1.0 and ψ_2 = 0.8, so the sizes indicated may not prove to be conservative. Early studies indicated that adopting Expression (6.10), T–beams, 450 wide would be up to 1% deeper and 5% more expensive.

The charts do not go above 800 mm depth as such beams are likely to be structurally significant and should be individually checked by design. Beams supporting point loads may be investigated by assuming uaudl = 2 x Σ(point loads)/span, in kN/m, but, again, such beams should be individually checked by design.

* In late 2008 a proposal was made for the UK National Annexe to include a limit of 2 or 2.5 on v_{Ed}/v_{Rdc} within punching shear requirements. It is apparent that this limitation could have major effects on flat slabs supported on relatively small columns (*L*/20) especially where edge columns have to accommodate service holes such as 150 mm holes for rain water outlets. This proposal has not gained universal support and until the UK NA is changed, no action can be taken.

In the optimization process there were slight differences in the allowances for cladding and the self-weight of beams compared with those for slabs. For the purposes of self-weight and perimeter cladding, the first 200 mm depth of beam was ignored, on the assumption that the applied load included the self-weight of a 200 mm solid slab and that only depths greater than this would affect the cost of cladding.

Particular attention is drawn to the need to check that there is adequate room for reinforcement at end supports. End support or column dimensions can have a major affect on the number and size of reinforcing bars that can be curtailed over the support. Hence, the size of the end support can have a major effect on the main bending steel and therefore size of beam. The charts assume that 25% of the end span reinforcement is provided at supports and that the end support/column size is based on edge columns with around 2% reinforcement supporting a minimum of three storeys or levels of similarly loaded beams. Smaller columns or narrower supports, particularly for narrow beams, may invalidate the details assumed and therefore size given (see Cl. 9.2.1.4 of BS EN 1992–1–1[2]).

Beam reinforcement densities relate to web width multiplied by overall depth.

7.1.5 Column charts

Column design depends on ultimate axial load N_{Ed} and ultimate design moment M_{Ed} that allows not only for 1st order moments from analysis, M, but also for the effects of imperfections, $e_i N_{Ed}$ and, in the case of slender columns, nominal 2nd order moments, M_2. (See *Concise Eurocode 2* [7], Section 5.6). Generally M_{Ed} should be considered in two directions M_{Edy} and M_{Edz}. Nonetheless, the column charts operate using 1st order moment, M.

For internal columns, design moments may generally be assumed to be nominal. Therefore the design chart for braced internal columns gives square sizes against total ultimate axial load for a range of reinforcing steel contents.

However, for perimeter (edge and corner) columns, moments are generally critical. Therefore moment derivation charts are provided so that 1st order moments in braced edge and corner columns may be estimated according to the assumed square size and whether they occur in beam-and-slab or flat slab construction. Opposite each moment derivation chart is the appropriate moment:load chart which gives the required reinforcement content for the assumed size according to the estimated ultimate axial load N_{Ed} and 1st order ultimate design moment, M. The moment:load charts presume that M occurs about one axis, z, and that the assumed ratio of M_y/M_z is not exceeded. The charts and tables allow for the effects of imperfections and, where appropriate, slenderness.

It should be noted that actual design moments depend on spans, loads and stiffnesses of members and are specific to a column or group of columns. Whilst the assumptions made, for instance in deriving the moment derivation charts, are considered to be conservative, they may not always be so. First order design moments, M, are also subject to a minimum value, M_{min}, which equates to the allowance for imperfections in Eurocode 2, which equals $0.02N_{Ed}$ for columns up to 600 mm square. Generally the sizes obtained should prove conservative but may not be so when fully analysed and designed. For instance analysis may show that column moments have been underestimated when less stiff beams or slabs, or very lightweight cladding, are used.

All data were derived from spreadsheets that designed square braced columns supporting either beam-and-slab construction or solid flat slabs. Floor-to-floor height was set at 3.75 m and panel aspect ratio was set at 1.00. In the case of flat slabs a 10 kN/m perimeter load was assumed. Checks were carried out over a limited range of panel aspect ratios.

Second order effects from slenderness have been incorporated into the charts. The columns are assumed to frame into beams-and-slabs or flat slabs and/or remote columns either end. Additional moments due to slenderness were based upon the stiffnesses of minimum depth (therefore minimum stiffness) of beams or flat slab for the span and load considered. Generally slenderness becomes an issue when the ratio of storey height to rectangular column dimension approaches 30 when the column is in double curvature, or when the ratio approaches 20 where the column is in single curvature i.e. has one pinned end.

A nominal cover of 30 mm or (main $\phi + \Delta c_{dev}$– link ϕ) has been assumed to all steel throughout. Δc_{dev} has been taken as 10 mm.

Internal columns

For internal columns, the following relatively conservative assumptions were made:

- Nominal column moments only.
- Flat slab construction, 200 mm deep.
- 7.5 m square panels.

The charts and data tables will be less accurate if unequal adjacent spans and/or loadings are envisaged, as this would produce higher than nominal column moments.

Perimeter columns

Moments in perimeter columns are generally critical. Therefore moment derivation charts are provided so that moments in edge and corner columns may be estimated according to whether they occur in beam-and-slab or flat slab construction. For an assumed column size, this moment and the ultimate axial load are used to interrogate moment:load charts – firstly to check the validity of the assumed column size and secondly to estimate the amount of reinforcement required in that column size. Some iteration may be required.

The charts are presented in pairs: moment derivation charts for braced edge columns in beam-and-slab construction are presented opposite moment:load charts for edge columns in beam-and-slab construction. Similarly for corner columns in beam-and-slab construction, for edge columns in flat slab construction and finally for corner columns in flat slab construction. In each case, the left hand chart allows estimation of moments and the right hand chart allows the assumed size to be checked and reinforcement to be estimated.

Moment derivation charts – For beam and slab construction these charts plot column moment against beam span for a range of applied uniformly distributed loads (uaudl) and column sizes based on the following assumptions:

- In the uaudl, $Q_k = G_k$.
- ψ_2, the quasi-permanent proportion of imposed load, ≤ 0.8.
- Beam sizes are derived from the charts and data in Section 3.2.

The moment derivation charts for flat slab construction plot column moment against slab span for a range of applied uniformly imposed loads (IL) and column sizes based on the following assumptions.

- Square panels (aspect ratio of 1.0).
- Values for ψ_2 are as assumed in Section 7.1.3.
- Slab thicknesses are derived from the data and charts in Section 3.1 for slabs.

Please note that these curves for perimeter columns supporting flat slabs are of an unusual shape due to the Eurocode limits to column transfer moments (see BS EN 1992–1–1 Annex, Cl. I.1.2 (5)).

The moment derivation charts for beam-and-slab construction assume economic beam sizes and slab thicknesses. For a given loading and span, the stiffness of different cost-optimised beams is quite similar so web width is not critical. Because the economic beam or slab depths will invariably be exceeded in practice and because the balancing effect of any perimeter loading is ignored, this approach should marginally overestimate column moments, and should therefore be conservative.

A storey height of 3.75 m was assumed when deriving column moments. Adjustment factors for other storey heights are tabulated below the charts.

Moment:load charts – These charts were derived from the design of square braced columns as described above. Additional moments due to imperfections and buckling were included as appropriate.

In order to evaluate the effects of biaxial bending, fixed values for M_y/M_z were assumed as below, where M_z is the look-up moment:

- Internal columns: $M_y/M_z = 1.0$
- Edge columns: $M_y/M_z = 0.2$
- Corner columns, beam and slab: $M_y/M_z = 0.5$
- Corner columns, two-way and flat slabs: $M_y/M_z = 1.0$

Concrete grade
The charts have been prepared for C30/37 and C50/60 concrete grades, allowing any intermediate grade to be interpolated. Smaller columns (higher concrete grades) occupy less lettable space. However, in low-rise buildings where buildability issues (e.g. different mixes on site, punching shear and reinforcement curtailment requirements) minimise potential gains, the lower grades may be more appropriate.

Reinforcement densities
Reinforcement densities assume 3.75 m storey heights with 40 diameter laps and H8 or H10 links.

Non-square columns
If non-square columns are required, it will normally be acceptable to use columns of the same area as those derived from the charts, as long as the ratio of sides is not greater than 2:1.

7.2 Precast and composite elements

7.2.1 Slabs

Generally the charts and data are derived from design spreadsheets to Eurocode 2[2], generally using grade C40/50 concrete or higher, with high-yield reinforcement (f_{yk} = 500 MPa) or high tensile strand or wire prestressing steel ($f_{pk} \geq$ 1770 MPa). The in-situ part of composite slabs is assumed to reach a strength of $f_{ck,i}$ before the precast part is de-propped.

The charts and data for proprietary precast and composite slab elements are based on data provided by industry in 2007/2008. The sizes given are selected, wherever possible, from those most commonly offered by manufacturers. The ultimate loads to supporting beams are derived from the self-weight quoted in Section 8.2.5 for the relevant size. For specific applications the reader should refer to manufacturers' current literature.

Precast and in-situ concrete can act together to provide composite sections that are efficient, economical and quick to construct. For slabs, these benefits are exploited in the range of composite floors available.

It should be noted, however, that load vs span data is limited in composite hollowcore slabs by the requirement to limit deflections to less than span/500 after the finishes have been applied, and therefore propping has little beneficial effect at the top end of the range of each depth of unit. By contrast, propping has beneficial effects when the composite slab is limited by service stresses, usually at the lower end of prestress.

Lattice girder slabs
Precast lattice girder slabs may be used to provide composite slabs. In single spans the units are placed onto supports either end and propped during concreting. Construction is similar for multiple-span applications and continuity is gained once props are removed in adjacent spans. It is customary not to use redistribution in the design of these slabs hence there is a difference in economic depths for two and three or more spans. For clarity in Section 4.1.6 only data for two spans is given. Overall depths and ultimate load data for three or more spans are given here.

Table 7.3
Data for composite lattice girder slabs of three or more spans

Three span, m-	3.0	4.0	5.0	6.0	7.0	8.0	9.0
Overall depth, mm, propped							
IL = 2.5 kN/m^2	115	115	124	146	184	224	262
IL = 5.0 kN/m^2	115	122	144	178	215	250	289
IL = 7.5 kN/m^2	115	133	162	200	242	281	
IL =10.0 kN/m^2	118	145	185	231	275		
Ultimate load to supporting beams, internal (end), kN/m							
IL = 2.5 kN/m^2	28 (14)	37 (19)	48 (24)	62 (31)	80 (40)	102 (51)	125 (63)
IL = 5.0 kN/m^2	39 (20)	53 (27)	70 (35)	90 (45)	113 (57)	138 (69)	166 (83)
IL = 7.5 kN/m^2	51 (26)	70 (35)	91 (46)	117 (59)	145 (73)	176 (88)	
IL =10.0 kN/m^2	62 (31)	86 (43)	114 (57)	145 (73)	179 (90)		

Double-tees

The charts and data for double-tees in Section 4.1.9 are based on the serviceability limits in Table NA.4 of the National Annex to Eurocode 2[2a], with a limiting crack width of 0.2 mm under the frequent load combination. Unscreeded double-tees with thickened top flanges are occasionally used for car parks. In these cases, depending upon exposure conditions, 'decompression' (see Section 4.1.4) should be checked for both the frequent and the quasi-permanent load combination. Designers are advised to consult specialist literature and suppliers.

7.2.2 Precast beams

The charts and data in this publication concentrate on unpropped non-composite beams. They cover a range of profiles, web widths and ultimate applied uniformly distributed loads (uaudl) using unstressed or prestressed reinforcement.

The data accompanying each chart give tabulated information on the economic overall depth of beam, ultimate loads to supports and estimated reinforcement densities in kg/m and kg/m^3. Ultimate loads to supports assume 'full' column centreline to column centreline (Col. c/c) spans and reaction factors of 1.0 to internal supports and 0.5 to end supports. For the purpose of calculating self-weight, the whole of the precast beam section has been taken.

The main complication with precast beams is the connections. The type of connection is usually specific to individual manufacturers and this detail can affect a beam's final cross-section. The sizes of beams given should therefore be considered as indicative only. Other aspects, including connection design and details, other components, columns, floors, walls, stairs, stability, structural integrity and overall economy, can influence final beam sizing.

For specific applications, the reader should refer to manufacturers and their current literature.

'Normally' reinforced precast beams

The charts for precast beams using unstressed (or 'normal') reinforcement were derived from spreadsheets that used the same optimization process as that used for in-situ beams. The design of precast beams was based on 'normal' reinforced concrete design principles as covered in Eurocode 2[2].

The economic depths of precast beams were determined using effective spans (centreline of support to centreline of support). However, in precast structures the centreline of support is often a distance from the centreline of a supporting column. So for presentation purposes, the centrelines of support have been assumed to be 250 mm from the centre of the columns (by assuming 'small' columns, 300 mm wide, and a distance of 100 mm from edge of the column to the centreline of support each end (see Figure 4.C). This 'full' column centreline to column centreline (Col. c/c span) dimension is highlighted in the charts and data and should be used in assessing loads to supports and columns. Where spans can be determined more accurately. 'effective spans' may be used to estimate the sizes of beams.

For L- and inverted T-beams, a ledge width of 125 mm was assumed. The upstanding concrete is therefore relatively wide and, for structural purposes, was considered part of the section. For the purpose of section analysis, in-situ concrete and the flange action of slabs and toppings were ignored. The depths of beams were minimised consistent with allowing suitable depth for precast floor elements. It is assumed that precast beams, unlike in-situ ones, are not loaded until 28 days after casting. Because of this, it was found that deflections affecting cladding and partitions tended to limit depths, rather than quasi-permanent deflections.

Manufacturers produce a wide range of preferred cross-sections based on 50 mm increments. Designs with other cross-sections can usually be accommodated. The economics of precast beams depend on repetition: a major cost is the manufacture of the base moulds. Reinforcement densities are quoted, but precasters may choose to use higher rebar densities to reduce deflections. Reinforcement densities should be restricted to a maximum of 350 kg/m^3.

Composite beams

Composite beams are not covered in this publication. During the construction of a composite beam (precast downstands acting with an in-situ topping), the precast element will often require temporary propping until the in-situ part has gained sufficient strength. The number of variables (e.g. construction stage loading, span, propped span, age at loading, flange width available) has, to date, precluded the preparation of adequate span:load charts and data for such beams. However, the combination of precast concrete with in-situ concrete (or hybrid concrete construction) has many benefits, particularly for buildability, and should not be discounted. In such circumstances, the beam charts should prove conservative.

Precast prestressed beams

The charts and data for unpropped non-composite precast prestressed beams were derived from design spreadsheets in accordance with Eurocode 2[2].

Again the economic depths of precast prestressed beams were determined using effective spans (centreline of support to centreline of support) and centrelines of support are assumed to be 250 mm from the centre of the columns. The centreline column to centreline column dimension is highlighted in the charts and data and this 'full' (Col. c/c span) dimension should be used in assessing loads to supports and columns.

The data accompanying each chart give tabulated information on the economic overall depth, ultimate loads to supports and estimated prestressing steel reinforcement densities in kg/m and kg/m^3 (excluding link reinforcement). The ultimate loads to supports assume 'full', Col. c/c span and, for the purposes of self-weight, ignore the top 200 mm of beam, and presume reaction factors of 1.0 to internal supports and 0.5 to end supports.

It should be noted that the quantity of prestressing tendons per cubic metre is almost identical for all prestressed beam sizes and spans. This is because the prestressing limits are set as constant. Therefore, the amount of prestressing steel per unit cross-sectional area is also constant.

7.2.3 Columns

These moment:load charts were derived from spreadsheets using the same process as that described for in-situ columns. The design of precast columns is based on the same reinforced concrete design principles as for in-situ columns as covered in BS EN 1992–1–1[2]. Column design depends on axial load, N_{Ed}, and design moment, M_{Ed}, induced. The charts and data operate on 1st order moments, M, where M has to be estimated (see Section 4.3.3). The charts and data for internal columns assume equal spans in each direction (i.e. $l_{y1} = l_{y2}$ and $l_{z1} = l_{z2}$) and, therefore, nominal moments (= M_{min}).

For edge and corner columns, a method for determining moments due to the eccentricity of connections is given in Section 4.3.3. Once the column 1st order moment, M, has been derived, size and reinforcement can be found from charts similar to those for in-situ columns. The moment:load charts allow for the effects of imperfections and, where appropriate, slenderness.

Conservatively, grade C40/50 has been used in the charts and data. However, grade C50/60 concrete suits factory production requirements and is commonly used for precast columns. Reinforcement densities are affected by connection details and are therefore not given.

Factory production and casting in a horizontal position may allow greater percentages of reinforcement to be used. However, connection details can limit the amounts of reinforcement that can be used. Higher percentages and higher or lower grades of concrete should be checked by a specialist engineer or contractor.

For specific applications, please refer to manufacturers.

7.3 Post-tensioned elements

7.3.1 General

The charts and data are derived from spreadsheets that designed the elements in accordance with BS EN 1992–1–1[2] and The Concrete Society Technical Report No. 43[22]. Reference was made to trade literature as required.

For the purposes of this publication, preliminary studies were undertaken to investigate the overall economics of slabs and beams versus amount of prestress. The studies suggested that high-strength concretes and high levels of prestress (e.g. 3.0, 4.0 and 5.0 MPa) in beams were, theoretically, increasingly more economic in overall terms. However, at these upper limits of stress (and span), problems of tendon and anchorage congestion and element shortening (due to prestress) become increasingly dominant. Theoretical economies had to be balanced against issues of buildability and serviceability and, so current UK custom and practice was reflected in the adoption of C32/40 concrete and bonded construction as the basis for the charts and data.

In many respects, span:depth charts for post-tensioned elements are very subjective as, for any given load and span, there is a range of legitimate depths. Indeed, in practice, many post-tensioned elements are designed to make a certain depth work. The amount of prestress assumed can be varied to make many depths work. Nonetheless, the charts and data in this publication are based on typical concretes and mid-range levels of prestress of:

- 1.5 MPa for one-way slabs.
- 2.0 MPa for flat slabs.
- 4.0 MPa for beams.

The slab charts give an indication of the range of depth for higher and lower levels of prestress. Higher levels of prestress may be appropriate in certain circumstances. For flat slabs 2.0 MPa might be considered high so the thicknesses given in the data might in some scenarios be regarded as low.

The shape of the lines for the span:depth charts for prestressed elements is the product of a number of slopes (from left to right, typically: vibration limitations, limits on the amount of prestress (P/A limit), deflection and the number of tendons allowed). For longer spans, number of tendons and limiting prestress dominate the design. At shorter spans and lower loads, it is the load that can be balanced by the catenary action of the prestressing forces in the tendons that can be critical.

It should be noted that if deeper sections than those charted are employed, less ‘normal’ reinforcement is likely to be required.

Unbonded vs bonded tendons

Whilst the charts and data assume the use bonded tendons, the charts should also be valid for use with unbonded tendons (e.g. with 15.7 mm diameter tendons, A_{ps} = 150 mm^2, f_{pk} = 1770 MPa). However, for use with unbonded tendons appropriate allowances should be made as several design assumptions made in the derivation of the charts may become invalid (e.g. cover, effective depth, long-term losses). Generally sections with unbonded tendons will need slightly fewer tendons than are indicated for sections with bonded tendons.

The arguments for unbonded and bonded tendons are outlined in Section 5.1.5. Data for unbonded construction is given under *Variations*.

Chlorides and car parks

As explained in Section 4.1.4, Table NA.4 of the UK NA to BS EN 1992–1–1[2a] requires any prestressing steel within exposure classes XD1, XD2, XD3, XS1, XS2 and XS3 to be in an area of 'decompression' under frequent load combinations. This 'decompression' requirement stipulates that all parts of the bonded tendons or duct lie at least 25 mm within concrete in compression.

Apart from their soffits, car park decks are taken to be exposure class XD1 or XD3[20]. Unbonded tendons are therefore recommended.

7.3.2 Ribbed slabs

Charts and data for ribbed slabs are based on 300 mm wide ribs, spaced at 1200 mm centres and assume a maximum of five 12.9 mm diameter tendons per rib. The weight of 'normal' (untensioned) reinforcement allows for nominal links to support the tendons, and nominal reinforcement in the topping. Where three or fewer tendons are used (and apart from 4 hours fire resistance and exposure class XD1), the sizes could be equally valid for 150 mm wide ribs at 600 centres or 225 mm wide ribs at 900 centres.

7.3.3 Flat slabs

The rules in Concrete Society Technical Report 43[22] regarding 'design hypothetical stress limits' were used to effectively control crack widths. The inclusion of untensioned bonded reinforcement was assumed (in top and bottom). Frame action using the column sizes indicated was assumed.

Punching shear can limit minimum thicknesses. The charts and data assume that column sizes will be at least equal to those given in the data (one 150 mm square hole assumed next to each internal column).

7.3.4 Beams – ratio of dead load to live load

For beams, the charts and data 'work' on ultimate applied uniformly distributed loads (uaudl). However, in multiple spans, the ratio of characteristic imposed load to characteristic dead load can alter span moments, and a ratio of 1.0 (i.e. applied variable action = applied permanent action) was assumed.

Lower ratios, with dead loads predominating, make little difference to the sizes advocated. Significantly higher ratios can induce mid-span hogging and might be dealt with by assuming that the beam depths tend to be the same as those for a single span (where ratios are of little consequence).

7.3.5 Design basis

The spreadsheets used in the preparation of the charts and data followed the method in The Concrete Society Technical Report No. 43[22], and used the load balancing method of design. Moments and shears were derived from moment distribution analysis and were modified by load balancing. Both tensioned and untensioned reinforcement were designed and allowance was made for distribution steel and reinforcement around anchorages. Designs were subject to limiting the amount of prestress and number of tendons. Generally, service moments were critical.

The charts and data assume the use of multi-strand bonded tendons, with 37 mm cover to tendon ducts and 30 or 35 mm cover to bonded reinforcement. The tendons are assumed to follow 'normal' profiles and stressing is assumed to be undertaken from one end only. The spreadsheet iterated both transfer losses and service losses until assumed losses equalled calculated losses. The effects of restraint to movement were ignored in the analysis and design.

In the production of these charts and data, the amount of prestress, P/A, was optimised to meet the following criteria:

- Tensile stresses ≥ minimum to limit cracking.
- Compressive stresses ≤ maximum to limit concrete compression.
- P/A ≤ the stated limit (1.5 MPa for one-way slabs, 2 MPa for flat slabs and 4 MPa for beams).

If these criteria or those for vibration or deflection could not be met, the section was deemed to have failed and its design was increased in depth until a practical working solution was found. In accordance with BS EN 1992–1–1[2], the maximum permissible crack width (w_k) for use with bonded tendons was set at 0.2 mm.

Deflection checks were based on calculations using the distribution coefficient, ζ, which adjusts the concrete section properties according to whether the section is cracked or not (see Eurocode 2, Cl. 7.4.3). Limiting deflections of span/250 overall and span/500 after the application of finishes were used. Vibration in use was considered using the method in the 1994 version of Concrete Society Technical Report 43[22], with square panels in the orthogonal direction. Generally, response factors of less than 4 were found (4 is acceptable for special offices, 8 for general offices and 12 is acceptable for busy offices). Further detailed vibration checks should be carried out for all final designs.

The charts for multiple spans are based on a two-span model: the three-span option was found to be non-critical for minimum depth.

The following design assumptions were used in the preparation of the charts and data:

Bonded reinforcement

f_{yk} = 500 MPa

Tendons

12.9 mm diameter tendons

A_{ps} = 100 mm^2

f_{pk} = 1860 MPa

Coefficient of friction, μ = 0.06

Wobble factor, K = 0.005

Relaxation = 2.5%

Relaxation factor = 1.5%

Young's modulus, E_{ps} = 195 GPa

Sheath thickness = 1.5 mm

Inflection of tendon at 0.1 of span

Wedge draw-in = 6 mm

Concrete

C32/40

25 kN/m^3

20 mm aggregate

Indoor exposure: (Note: Indoor exposure is more critical than exterior exposure.)

Shrinkage, ε_c, depends on effective thickness.

Creep factor, φ, depends on effective thickness and loading history.

Transfer assumed to occur at 3 days where:

$f_{ck(t)}$ at transfer = 20.8 MPa

Young's modulus, $E_{cm(t)}$ = 31.1 GPa

Quasi-permanent loading is assumed to occur at 28 days where:

Young's modulus, E_{cm} = 35.2 GPa

8 Actions

8.1 Design values of actions

According to BS EN 1990[9] the design value of an action is $\gamma_F \psi F_k$

where

F_k = characteristic value of an action

γ_F = partial factor for the action (see Sections 8.1.1 and 8.1.2 below)

ψ = a factor that converts the characteristic value of an action into a representative value

For a permanent action (dead load), $\psi = 1.0$.

For variable actions (imposed load), ψ has several possible values:

- Considering ULS, generally, $\psi = 1.0$ for a leading variable action (and $\psi = \psi_0$ for an accompanying variable action)
- Considering SLS and frequent load combinations (e.g. in assessing cracking), $\psi = \psi_1$ for a leading variable action (and $\psi = \psi_2$ for an accompanying variable action)
- Considering SLS and quasi-permanent load combinations (e.g. in assessing crack width or deformation), $\psi = \psi_2$ for the leading variable action (and for accompanying variable actions)
- Colloquially,
  - ψ_0 is known as the combination value,
  - ψ_1 the frequent value and
  - ψ_2 the quasi-permanent value.

As may be deduced from Table 8.1, the values of ψ_0, ψ_1, and ψ_2 depend on the category of use.

ψF_k may be considered as the representative action, F_{rep}, appropriate to the limit state being considered.

Table 8.1 Imposed loads in buildings: values of ψ factors

Action	Combination, ψ_0	Frequent, ψ_1	Quasi-permanent, ψ_2
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area – vehicle weight ≤ 30 kN	0.7	0.7	0.6
Category G: traffic area – 30 kN < vehicle weight ≤ 160 kN	0.7	0.5	0.3
Category H: roofs[a]	0.7	0.0	0.0
Snow loads where altitude ≤ 1000 m asl[a,b]	0.5	0.2	0.0
Wind loads[a]	0.5	0.2	0.0

Key
a See BS EN 1991
b Above sea level

Note
The numerical values given above are in accordance with BS EN 1990 and its UK National Annex[9, 9a]

8.1.1 Ultimate limit state

According to BS EN 1990 and its UK National Annex[9, 9a] for the ULS of strength, the designer may choose between using Expression (6.10) or the less favourable of Expressions (6.10a) and (6.10b). Applying the factors in the National Annex*, the designer effectively has the choice between using:

Expression (6.10), i.e. $1.35\ G_k + 1.5Q_k$

or the least favourable of:

Expression (6.10a), i.e. $1.35\ G_k + \psi_0 1.5Q_k$ and

Expression (6.10b), i.e. $1.25\ G_k + 1.5Q_k$.

Values for ψ_0 are given in Table 8.1. As explained in Section 7, the charts and tables are based on the use of the more onerous of Expressions (6.10a) and (6.10b) and the values for ψ_0 used in the charts and tables are detailed in Table 8.2. Generally for relatively heavy permanent actions (i.e. for concrete structures) Expression (6.10b) will usually apply. The exception is for storage loads where $\psi_0 = 1.0$ and Expression (6.10a) applies. See section 1 of *How to design concrete structures using Eurocode 2*[19] for a fuller explanation of the expressions for loading in BS EN 1990.

8.1.2 Serviceability limit state

For quasi-permanent serviceability limit states (e.g. deformation, crack widths), Table A1.4 of BS EN 1990 leads to the use of $\gamma_F = \gamma_G = 1.0$ for permanent actions and $\gamma_F = \gamma_Q = \psi_2$ for variable actions.

Appropriate values for ψ_2 are given in Table 8.1. The values used in the charts and tables for in-situ precast and post-tensioned elements are detailed in Table 8.2.

The choices made for the value for ψ_2 in the determination of the charts and data have led to some conservatism in the design of both slabs supporting office loads of 5.0 kN/m^2 and beams supporting office or residential loads where deformation is a governing criterion. On the other hand the choices are a little optimistic for beams supporting storage loads. These choices also lead to the need to assume that the imposed load constitutes the whole of the characteristic variable action on slabs, and to some peculiarities in some of the precast span:load charts.

Table 8.2 Values of ψ_0 and ψ_2 used in charts and data

Element	Loading	Value of ψ_0 used	Value of ψ_2 used	Assumed use
Slabs	IL = 2.5 kN/m^2	0.7	0.3	Residential or office
	IL = 5.0 kN/m^2	0.7	0.6	Worst case for residential, office, congregation, shopping or lightweight traffic
	IL = 7.5 kN/m^2	0.7	0.6	
	IL = 10.0 kN/m^2	1.0	0.8	Storage
Beams	All	0.7	0.6	Worst case for residential, office, congregation, shopping or lightweight traffic

Note
For precast beams values of $\psi_0 = 1.0$ and $\psi_2 = 0.8$ were used

8.2 Slabs

The slab charts and data give overall depths against span for a range of characteristic imposed loads assuming a superimposed dead load (finishes, services, etc.) of 1.5 kN/m^2. In order to use the slab charts and data as intended, it is essential that they are interrogated at the correct characteristic variable action, q_k, which should be taken as the sum of imposed load (see Section 8.2.1) and allowances for partitions (see Section 8.2.2). If necessary the load at which the chart is interrogated should be modified to account for different superimposed dead loads (see Section 8.2.4).

* The UK National Annex to BS EN 1990[9] confirms that $\gamma_F = \gamma_G = 1.35$ for permanent actions and $\gamma_F = \gamma_Q = 1.50$ for variable actions. In Expression (6.10b), γ_G is modified by a factor, ξ, which according to the National Annex = 0.925[9a]. The value for γ_G for permanent actions is intended to be constant across all spans.

8.2.1 Imposed loads, q_{ks}

The imposed load should be determined from the intended use of the building (see BS EN 1991[6] and its UK National Annex[6a]). The actual design imposed load used should be agreed with the client. However, the following characteristic imposed loads are typical of those applied to floor slabs.

Table 8.3 Imposed loads for floors

Load	Use
1.5 kN/m²	Domestic, minimum for roofs with access
2.0 kN/m²	Hotel bedrooms, hospital wards
2.5 kN/m²	General office loading, car parking
3.0 kN/m²	Classrooms, residential corridors, offices at or below ground floor level
4.0 kN/m²	High-specification office loading, shopping, museums, reading rooms, corridors
5.0 kN/m²	Office file rooms, areas of assembly, places of worship, dance halls
7.5 kN/m²	Plant rooms (NB not covered by BS EN 1991)
2.4 kN/m²/m	General storage per metre height of stored materials
4.0 kN/m²/m	Stationery stores per metre height of stored materials

The slab charts focus on:

2.5 kN/m²	General office loading, car parking
5.0 kN/m²	Office loading (‘4 + 1’, see below), file rooms, areas of assembly etc.
7.5 kN/m²	Plant room and storage loadings
10.0 kN/m²	Storage loadings (approximately = 4 x 2.4 kN/m²/m)

In the charts for slabs no reductions in imposed load have been made (as in BS EN 1991[6], Cl. 6.3.1.2 (10)) nor are provisions for concentrated loads considered.

In this publication, the term ‘imposed load’ refers to the total characteristic value of the variable action. (‘Variable load’ is that portion of the imposed loading that may be applied after the quasi-permanent portion.)

8.2.2 Partition loads

The self-weight of movable partitions may be taken into account by including a uniformly distributed load, q_k, which should be added to the imposed loads of floors as follows:

- For movable partitions with a self-weight of 1.0 kN/m wall length: q_{ks} = 0.5 kN/m²
- For movable partitions with a self-weight of 2.0 kN/m wall length: q_{ks} = 0.8 kN/m²
- For movable partitions with a self-weight of 3.0 kN/m wall length: q_{ks} = 1.2 kN/m²

For partitions imparting a line load greater than 3.0 kN/m, BS EN 1991[6] recommends calculation of an equivalent uniformly distributed load.

An allowance of 1.0 kN/m² should be considered for demountable partitions in office buildings. A common specification is ‘4 + 1’, i.e. 4.0 kN/m² imposed load plus 1.0 kN/m² for demountable partitions.

8.2.3 Superimposed dead loads (SDL), g_{ksdl}

Superimposed dead loads allow for the weight of services, finishes, etc. The charts and data make an allowance of 1.50 kN/m² for superimposed dead loading (SDL). Examples of typical build-ups are given in Table 8.4 .

Table 8.4
Superimposed dead loads for different types of building

Office floor		Residential floor		School floor	
Carpet	0.03	Carpet	0.05	Carpet/flooring	0.05
Raised floor	0.30	Floating floor	0.15	Suspended ceiling	0.15
Suspended ceiling	0.15	Suspended ceiling	0.20	Services	0.20
Services	0.30	Services	0.10		
Total	**0.78 kN/m^2**	**Total**	**0.50 kN/m^2**	**Total**	**0.40 kN/m^2**
Office core area		**Hospital floor**		**Flat roof/external terrace**	
Tiles & bedding (say)	1.00	Flooring	0.05	Paving or gravel (say)	2.20
Screed	2.20	Screed	2.20	Waterproofing	0.50
Suspended ceiling	0.15	Suspended ceiling	0.15	Insulation	0.10
Services	0.30	Services (can be greater)	0.05	Suspended ceiling	0.15
				Services	0.30
Total	**3.65 kN/m^2**	**Total**	**2.45 kN/m^2**	**Total**	**3.25 kN/m^2**

If not known precisely, allowances for dead loads on plan should be generous and not less than the following:

Floor finish (screed)	1.8 kN/m^2
Ceilings and services load	0.5 kN/m^2
Demountable partitions	1.0 kN/m^2
Blockwork partitions	2.5 kN/m^2
Raised access flooring	0.3 kN/m^2
Suspended ceilings	0.15 kN/m^2

BS EN 1991[6] also schedules the weight of some building materials. It can be used to derive the following typical characteristic loads:

Carpet	0.03 kN/m^2
Terrazzo tiles, 25 mm	0.55 kN/m^2
Screed, 1:3, 50 mm	1.10 kN/m^2
Gypsum plaster, 12.7 mm	0.21 kN/m^2
Gypsum plasterboard, 12.7 mm	0.11 kN/m^2

8.2.4 Equivalent imposed dead loads

If the actual superimposed dead load differs from the 1.50 kN/m^2 allowed, the characteristic imposed load used for interrogating the charts and data should be adjusted by adding 1.25/1.5 x (actual SDL – 1.50) kN/m^2. The equivalent characteristic imposed load may be estimated from Table 2.1, repeated here as Table 8.5.

Table 8.5
Equivalent imposed loads, kN/m^2

Imposed load, kN/m^2	Superimposed dead load, SDL, kN/m^2					
	0.0	1.0	2.0	3.0	4.0	5.0
2.5	1.25	2.08	2.92	3.75	4.58	5.42
5.0	3.75	4.58	5.42	6.25	7.08	7.92
7.5	6.25	7.08	7.92	8.75	9.58	10.40
10.0	8.75	9.58	10.40	11.30	12.10	n/a

Note
The values in this table have been derived from 1.25(SDL –1.5)/1.5 + IL

8.2.5 Self-weights of slabs, g_{ks}

The self-weights of slabs are given in Table 8.6 and are indicative. Values for ribbed and waffle slabs may differ from those given depending upon the mould manufacturer. Values for precast slabs also may vary between manufacturers.

Table 8.6
Characteristic self-weight of slabs, g_{ks}, kN/m^2

Type of slab	Slab thickness, mm					
	100	**200**	**300**	**400**	**500**	**600**
Solid slabs[a]	2.5	5.0	7.5	10.0	12.5	15.0
Ribbed slabs[b] : 100% ribbed	–	–	3.6	4.3	5.0	5.8
75% ribbed, 25% solid	–	–	4.6	5.7	6.9	8.1
Waffle slabs[c] : 100% waffle	–	–	4.2	5.2	6.3	7.4
75% waffle, 25% solid	–	–	5.0	6.4	7.8	9.3
	150	**200**	**250**	**300**	**350**	**400**
Hollowcore slabs without topping	2.8	3.4	3.9	4.5	5.1	5.7
	200	**250**	**300**	**350**	**400**	**450**
Hollowcore slabs with 50 mm topping[d]	4.1	4.7	5.2	5.8	6.4	7.0
	300	**400**	**500**	**600**	**700**	**800**
Double-tees without topping[e]	3.1	3.4	3.8	4.2	4.6	4.9
	375	**475**	**575**	**675**	**775**	**875**
Double-tees with 75 mm topping[f]	4.9	5.3	5.7	6.0	6.4	6.8

Key
a Including in-situ, precast and composite solid slabs
b Bespoke moulds, 150 mm ribs at 750 mm cc, 100 mm topping
c Bespoke moulds, 150 mm ribs at 900 mm cc, 100 mm topping
d For slabs with 40 mm topping, deduct 0.25 kN/m^2
e Some double-tees for use without topping are produced at thicknesses of 325, 425 and 525 mm. In these instances add 0.4 kN/m^2 to the value given for 300, 400 and 500 mm
f For slabs with 100 mm topping, add 0.6 kN/m^2

In order to use the beam and column charts and data as intended, it may be necessary to calculate beam and column loads from first principles, or, in other cases, it may be necessary to know the proportion of dead load to imposed load.

In accordance with BS EN 1991, all slab charts and data include allowances for self-weight of reinforced concrete at a density of 25 kN/m^2.

8.2.6 Ultimate slab load, n_s

Ultimate slab load is the summation of characteristic permanent and variable actions multiplied by appropriate partial load factors:

$$n_s = g_{ks}\gamma_G + g_{ksdl}\gamma_G + q_{ks}\gamma_Q$$

where

$g_{ks}\gamma_G$ = ultimate self-weight of slab

$g_{ksdl}\gamma_G$ = ultimate superimposed dead loads

$q_{ks}\gamma_Q$ = ultimate imposed load

where

g_{ks}, g_{ksdl} and q_{ks} are as explained in Sections 8.2.5, 8.2.3 and 8.2.1 above and measured in kN/m^2.

γ_G = load factor for dead loads = 1.25 in most cases (see Section 8.1.2)

γ_Q = load factor for imposed loads = 1.5 in almost all cases (see Section 8.1.2)

Example: Calculate the ultimate load of a 300 mm solid slab supporting 1.5 kN/m^2 superimposed dead load and 5.0 kN/m^2 imposed load.

From Table 8.6,
g_{ks} for 300 mm solid slab = 7.5 kN/m^2. Therefore, assuming use of Expression (6.10b)
n_s = 7.5 x 1.25 + 1.5 x 1.25 + 5.0 x 1.5
= 17.75 kN/m^2

8.3 Beams

8.3.1 Ultimate applied uniformly distributed loads to beams (uaudl)

The beam charts give overall depths against span for a range of ultimate applied uniformly distributed loads (uaudls) and web widths, assuming single spans or the end span of three spans. Ultimate applied udl, uaudl, to a beam may be determined as follows:

uaudl $= n_s \, l_s \, \text{erf} + n_{ll}$

where

n_{ll} = ultimate line load. See Section 8.3.3.

$n_s \, l_s \, \text{erf}$ = ultimate applied load from slabs.

where

n_s = ultimate slab load, kN/m^2, as described above

l_s = slab span perpendicular to the beam, in metres. In the case of multiple-span slabs, take the average of the two spans perpendicular to the beam

erf = elastic reaction factor, taken as

- 0.46 for end support of continuous slabs (0.45 for beams)
- 0.5 for end support of simply supported slabs (or beams)
- 1.0 for other interior supports of multiple-span continuous slabs (e.g. in-situ slabs) or for all interior supports of discontinuous slabs (e.g. precast slabs)
- 1.1 for the first interior supports of continuous slabs of three or more spans
- 1.2 for the internal support of a two-span continuous slab.

$n_s l_s \text{erf}$ may be calculated from first principles.

Alternatively $n_s l_s \text{erf}$ may be estimated from ultimate load to supporting beams data for the appropriate slab in tables in Sections 3.1, 4.1, 5.2.1 or 5.2.2. See also Section 8.3.2.

8.3.2 Elastic reaction factors

The data for slabs assume elastic reaction factors of 0.5 at end supports and 1.0 at internal supports. These figures may need to be adjusted to account for actual conditions. For instance, for beams providing internal support to three spans of slabs, ultimate load to supporting beams should be increased by 1.1/1.0, i.e. by 10%. Where a beam provides the internal support for an in-situ slab of two spans, consider increasing loads to beams by 20%.

Precast construction is generally assumed to be simply supported and an internal elastic reaction factor of 1.0 is usually appropriate (see Section 5.1.2). Reactions for multiple span post-tensioned slabs and beams should be verified from analysis and design. $P\Delta$ effects caused by stressed tendons alter the distribution of load, and so elastic reaction factors greater than 1.1 may be appropriate for internal supports.

As an indication, tabulated loads to end supports (or edge columns) may be reduced by an average of:

- One-way slabs – 6% of ultimate load to supporting beam (end)
- Ribbed slabs – 3% of ultimate load to supporting beam (end)
- Flat slabs – 15% of ultimate load to edge supporting column

- 1000 mm beams – 18% of ultimate load to end support per metre width
- 2400 mm T-beams – 21% of ultimate load to end support

Please note that the values given above for flat slabs and beams may vary by ±5% and that the loads to 1st internal supports (or columns) should be increased by a similar amount. Detailed checks and scheme design should be undertaken to verify reactions to supports.

8.3.3 Ultimate line load, n_{ll}

The permanent actions arising from cladding, other line loads such as heavy partitions and line loads, and slabs other than the 200 mm thickness assumed in the derivation of beams, need to be allowed for. Conveniently this may be done at the ultimate limit state by allowing for an ultimate line load, n_{ll}, where

$n_{ll} = g_{kc}\gamma_{fgk}h + g_{ko}\gamma_{fgk}h + g_{kbm}\gamma_{fgk}$

where

$g_{kc}\gamma_{fgk}h$ = ultimate cladding loads. See Table 8.7

where

g_{kc} = characteristic dead load of cladding, kN/m^2. See Table 8.8 for typical characteristics loads of cladding and components of cladding

γ_{fgk} = partial safety factor for permanent action, normally 1.25

h = supported height of cladding (e.g. floor to floor), m.

$g_{ko}\gamma_{fgk}h$ = other ultimate line loads. The ultimate applied load from partitions may be determined by using Table 8.8 for characteristic loads and interpolating between values in Table 8.7 for the appropriate load and supported height. Alternatively it may be calculated as illustrated in the following examples.

where

g_{ko} = characteristic dead load from other sources (e.g. internal partitions), kN/m^2

γ_{fgk} = as before

h = as before

$g_{kbm}\gamma_{fgk}$ = adjustment for ultimate beam self-weight. The beam charts assume that in-situ slab loads are imparted by a 200 mm deep solid slab. Where the slab is not 200 mm deep some adjustment may be made to n_{ll} as indicated by Table 8.9.

where

g_{kbm} = adjustment for slab thickness, kN/m

γ_{fgk} = as before

Table 8.7
Ultimate cladding loads, $g_{kc}\gamma_{fgk}h$, kN/m

Characteristic cladding load, g_{kc}, kN/m^2	Supported height of cladding, h, m								
	2.4	2.6	2.8	3.0	3.2	3.4	3.6	3.8	4.0
0.5	2	2	2	2	2	3	3	3	3
1.0	3	4	4	4	4	5	5	5	5
1.5	5	5	6	6	6	7	7	8	8
2.0	6	7	7	8	8	9	9	10	10
2.5	8	9	9	10	10	11	12	12	13
3.0	9	10	11	12	12	13	14	15	15
3.5	11	12	13	14	14	15	16	17	18
4.0	12	13	14	15	16	17	18	19	20
4.5	14	15	16	17	18	20	21	22	23
5.0	15	17	18	19	20	22	23	24	25

Table 8.8
Typical characteristic cladding loads, g_{kc} (and partition loads)

Element	g_{kc}, kN/m^2
Brickwork, 102.5 mm	
Solid high-density clay	2.34
Solid medium-density clay	2.17
High-density clay with 15% voids	1.95
Concrete	2.30
Blockwork, solid, 150 mm	
Stone aggregate	3.20
Lightweight aggregate	1.90
Aerated (560 kg/m^3)	0.85
Aerated (800 kg/m^3)	1.13
Blockwork, cellular, 150 mm	
Stone aggregate	2.35
Lightweight aggregate	1.67
Plaster, 12 mm	
Gypsum, 2-coat	0.21
Lightweight, 2-coat	0.11
Other cladding loads	
Double glazing, 2 no. x 6 mm, c/w aluminium framing	0.35
Curtain wall glazing, 2 no. x 8 mm, c/w aluminium framing	0.50
Precast concrete cladding, average 100 mm thick	2.50
Profiled metal cladding	0.15
20 mm dry lining on studwork	0.15
50 mm insulation	0.02

Table 8.9
Adjustment to uaudl per metre width of beam web, $g_{kbm}\gamma_{fgk}$, kN/m

Depth of slab, mm	Internal T-beams	Perimeter L-beams
100	4	2
200	0	0
300	–3	–1
400	–6	–3
500	–9	–4

Example: Calculate the uaudl for perimeter beam with cladding and supporting a 250 mm deep slab.

Determine the ultimate applied uniformly distributed load to a 300 mm wide perimeter beam supporting a 250 mm one-way solid slab, IL 5.0 kN/m^2, SDL 1.5 kN/m^2, spanning 6.0 m, with 3.5 m of cladding, average IL 3.0 kN/m^2.

Ultimate slab load,

$n_s = (6.25 + 1.5) \times 1.25 + 5.0 \times 1.5 \quad = 17.2$ kN/m^2

Ultimate applied load from slabs,

$n_s l_s erf = 17.2 \times 6.0 \times 0.5 \quad = 51.6$ kN/m

Ultimate line load from cladding,

$n_{ll} = 3.5 \times 3.0 \times 1.25 \quad = 13.1$ kN/m

Adjustment for self-weight of beam,

$n_{ll} = (0.25 - 0.20) \times 0.30/2 \times 25 \times 1.25 = -0.2$ kN/m

Total $\quad = 64.5$ kN/m

Therefore total ultimate applied uniformly distributed load (uaudl) to beam = 64.5 kN/m

Example: Calculate allowance for cladding.

Determine typical line loads from traditional brick-and-block cavity wall cladding onto a perimeter beam.

Determine load/m^2	
102.5 mm brickwork, solid high-density clay	= 2.34 kN/m^2
50 mm insulation	= 0.02 kN/m^2
150 mm lightweight (800 kg/m^3) blockwork	= 1.13 kN/m^2
12.7 mm gypsum plaster	= 0.21 kN/m^2
Subtotal	= 3.70 kN/m^2
2 no. x 6 mm double glazing c/w framing	= 0.35 kN/m^2
Determine average load/m^2, g_{kc}	
Assuming minimum 25% glazing, average load/m^2,	
g_{kc} = 75% x 3.70 + 25% x 0.35	= 2.86 kN/m^2
Determine load/m	
Assuming the height of cladding to be supported is 3.5 m, characteristic load per metre run	
g_{kc} = 2.86 x 3.5	= 10 kN/m^2
Ultimate load per metre run	
$n_{ll} = g_{kc} g_{fgk} h$ = 10.0 x 1.25	= 12.5 kN/m
	Allow 12.5 kN/m for cladding at ULS

Example: Calculate allowance for line loads from other sources. For example, allowing for a solid 150 mm blockwork partition on an internal beam:

Characteristic loads		
150 mm blockwork, solid, stone aggregate	=	2.35 kN/m^2
2 no. x 12 mm plaster, gypsum, two coat	=	0.42 kN/m^2
Total load g_{ko}	=	2.77 kN/m^2
If the height of partition to be supported is 3.0 m, ultimate cladding load,		
$n_{ll} = g_{ko} g_{fgk} h$ = 2.77 x 3.0 x 1.25	=	10.4 kN/m
Allow 10.4 kN/m for 3 m high 150 mm blockwork partition		

8.3.4 Beams supporting two-way slabs

The loads outlined in the two-way slab data are derived in accordance with Eurocode 2. They assume square corner panels and that the ultimate loads to supporting beams are treated as uniformly distributed loads over 75% of the supporting beams' spans. Treating the loads as though they were applied to 100% of the supporting beams' spans overestimates the moment but may be regarded as making little practical difference for the purposes of sizing beams.

However, where more accuracy is required, the ultimate loads to supporting beams given in Section 3.1.9 should be regarded as maxima and should be multiplied by the appropriate factor from Table 8.10, to obtain the equivalent $n_s l_s$erf (See 8.3.1 above) over 75% of the beam span. This equivalent $n_s l_s$erf may be used, together with allowances for cladding etc. as the basis for sizing supporting beams more accurately.

Reactions to supporting columns should be calculated.

Table 8.10
Factors to be applied to ultimate loads to supporting beams given in Section 3.1.9 to give equivalent $n_s l_s$erf over 75% of the supporting beam span

Beam on	Panel aspect ratio, l_z/l_y					
	1.0	1.1	1.2	1.3	1.4	1.5
Long edge	0.67	0.73	0.78	0.82	0.86	0.89
Short edge	0.67	0.67	0.67	0.67	0.67	0.67

Example: Calculate the uaudl for the internal beams supporting a two-way slab.

The multiple-span two-way slab supports loads of SDL = 1.5 kN/m^2 and IL = 5.0 kN/m^2. It is supported by beams on a 7.0 m by 9.0 m grid.

As the slab has rectangular panels of 7.0 m by 9.0 m, from Table 3.6c, the equivalent span is 7.7 m.

For the 7.7 m span, from Table 3.6b (multiple span, 7.7 m span, 5.0 kN/m^2) a 175 mm slab would be required with a peak load to internal beam = say, 115 kN/m.

Panel aspect ratio = 9.0/7.0 = 1.29

Therefore, from Table 8.10 long edge factor = say, 0.81.

For the 9.0 m span beam, equivalent load to $n_s l_s$erf on internal beam
= 115 x 0.81 = 93 kN/m

And for the shorter 7.0 m span beam, from Table 8.10 factor = 0.67.

So equivalent load to $n_s l_s$erf on internal beam = 115 x 0.67 = 77 kN/m

8.3.5 Post-tensioned beams

The first set of charts for post-tensioned beams assume 1000 mm wide rectangular beams. Other rectangular post-tensioned beam widths can be investigated on a pro-rata basis, i.e. by determining the ultimate applied uniformly distributed load (uaudl) per metre of web. The equivalent uaudl per metre width for a beam that is not 1000 mm wide may be estimated from Table 8.11. For the proposed beam width read down Table 8.11 to the appropriate uaudl for the beam, then read across to estimate the equivalent uaudl per metre width, interpolating as necessary.

Table 8.11
Equivalent uaudl per metre width of web

Proposed beam width, mm							Equivalent uaudl per metre width, kN/m
300	450	600	900	1200	1800	2400	
uaudl, kN/m							
8	11	15	23	30	45	60	**25**
15	23	30	45	60	90	120	**50**
23	34	45	68	90	135	180	**75**
30	45	60	90	120	180	240	**100**
45	68	90	135	180	270	360	**150**
60	90	120	180	240	360	480	**200**
75	113	150	225	300	450	600	**250**
90	135	180	270	360	540	720	**300**

8.4 Columns

8.4.1 Calculating ultimate axial load, N_{Ed}

In design calculations, it is usual to determine the characteristic loads on a column on a floor-by-floor basis, keeping dead and imposed loads separate. Elastic reaction factors, erf, and load factors, γ_f, are applied to the summation of these loads to obtain ultimate loads used in the design. BS EN 1991[6] allows some reduction in imposed load depending on building usage, area supported and number of storeys.

Hence, the ultimate axial load can be expressed as and calculated from:

$$N_{Ed} = \Sigma\{g_{ks}l_y l_z \mathrm{erf} + g_{kby}l_y \mathrm{erf} + g_{kbz}l_z \mathrm{erf} + G_{kc}\}\ \gamma_{fgk} + \Sigma\{q_{ks}l_y l_z\}\ \mathrm{erf}\alpha_n\gamma_{fqky}$$

where

$\Sigma\{....\}$ = summation from highest to lowest level

g_{ks} = characteristic slab self-weight and superimposed dead loads

l_y = supported span in the y direction, taken to be half of the sum of the two adjacent spans (but see Section 8.4.5, *Elastic reaction factors*, below)

l_z = supported span in the z direction, taken to be half of the sum of the two adjacent spans (but see Section 8.4.5, *Elastic reaction factors*, below)

erf = elastic reaction factor, see Section 8.4.5 below

g_{kby} = characteristic line loads from permanent actions (dead loads) parallel to the y direction e.g. cladding loads, partition loads, extra over beam loads

g_{kbz} = characteristic line loads from permanent actions (dead loads) parallel to the z direction

G_{kc} = characteristic self-weight of column

q_{ks} = characteristic imposed load for the slab

γ_{fgk} = partial factor for dead load: 1.35 suggested but see Section 8.4.3 below

γ_{fqky} = partial factor for imposed load, 1.5 suggested but see Section 8.4.3 below

α_n = imposed load reduction factor, see Section 8.4.4 below

8.4.2 Estimating ultimate axial load, N_{Ed}

N_{Ed} may be estimated per level from the data given under either:

- *Ultimate load to support/columns* data for the appropriate beams in tables in Sections 3.2, 4.2, or 5.3 or
- *Ultimate load to supporting columns* data for the appropriate troughed, flat or waffle slabs in Sections 3.1.8, 3.1.10, 3.1.11, 3.1.12, 4.1.12 or 5.2.3.

Due allowance should be made for:

- Elastic reaction factors. Please note that the data for beams and flat slabs assume elastic reaction factors of 0.5 at end supports and 1.0 at internal supports. See Section 8.4.5.
- Line loads at each level from cladding, partitions, etc, as described for n_{ll} in Section 8.3.3.
- Partial factors: see Section 8.4.3.
- Ultimate self-weight of columns, which can be estimated from Table 8.12.

Examples of this method is given in Sections 2.11.4 and 2.11.5.

Table 8.12
Ultimate self-weight of columns, $\gamma_G G_{kc}$, per storey, kN

Size mm sq.	Ultimate self-weight at height (e.g. floor-to-soffit), m								
	2.4	2.6	2.8	3.0	3.2	3.4	3.6	3.8	4.0
250	5	5	5	6	6	7	7	7	8
300	7	7	8	8	9	10	10	11	11
400	12	13	14	15	16	17	18	19	20
500	19	20	22	23	25	27	28	30	31
600	27	29	32	34	36	38	41	43	45
700	37	40	43	46	49	52	55	58	61
800	48	52	56	60	64	68	72	76	80

Note
Table assumes $\gamma_G = 1.25$

8.4.3 Partial factors for load

As explained in Section 8.1.1, based on BS EN 1990 and its UK National Annex[9, 9a] for the ULS of strength, the designer may choose between using Expression (6.10) or the less favourable of Expressions (6.10a) and (6.10b). Applying the factors in the National Annex , the designer effectively has the choice between using:

Expression (6.10) i.e. $1.35\ G_k + 1.5Q_k$

or the least favourable of:

Expression (6.10a) i.e. $1.35\ G_k + \psi_0 1.5Q_k$
and
Expression (6.10b) i.e. $1.25\ G_k + 1.5Q_k$.

Values for ψ_0 are given in Table 8.1 and it may be seen that generally for relatively heavy permanent actions (i.e. for concrete structures) Expression (6.10b) will usually apply. The exception is for storage loads where $\psi_0 = 1.0$, in which case Expression (6.10a) applies.

For column scheme design it is suggested that conservatively Expression (6.10) i.e. $1.35\ G_k + 1.5Q_k$ is used. Where column loads have been derived using Expression (6.10b) i.e. $1.25\ G_k + 1.5Q_k$ and $G_k \doteqdot 2Q_k$, a 5% increase in axial load is recommended.

8.4.4 Imposed load reduction factors

In accordance with BS EN 1991[6], Clause 6.3.1.2(11), imposed loads (apart from those on roofs) may be reduced in accordance with the area supported, A (m^2), or the number of floors, excluding roof, being supported, n, as shown below:

$\alpha_A = 1.0 - A/1000 \geq 0.75$

$\alpha_n = 1.1 - n/10$ for $1 \leq n \leq 5$

$= 0.6$ for $5 \leq n \leq 10$ and

$= 0.5$ for $n > 10$

Generally, live load reduction is unwarranted in the pre-scheme design of medium-rise structures, where a factor of 1.00 should be used.

8.4.5 Elastic reaction factors for column loads

The *Ultimate loads to supports/columns* quoted in the beam data assume an elastic reaction factor of 0.5 to end supports and 1.0 to internal supports. Elastic reaction factors should reflect circumstances and generally for normally reinforced elements, the elastic reaction factors, erf, are taken as:

- 0.46 for end support of continuous slabs (0.45 for beams)
- 0.5 for end support of simply supported slabs (or beams)
- 1.0 for other interior supports of multiple-span continuous slabs (e.g. in-situ slabs) or for all interior supports of discontinuous slabs (e.g. precast slabs)
- 1.1 for the first interior supports of continuous slabs of three or more spans
- 1.2 for the internal support of a two-span continuous slab.

Often, to allow for continuity of in-situ construction, an increase of 10% (1.1/1.0) is used for penultimate columns supporting a beam of three or more equal spans. In the case of two-span beams an increase of 20% might be warranted.

For columns, using elastic reaction factors of 0.5 to end supports and 1.0 to internal supports reflects Clause 3.8.2.3 of BS 8110[5]. This clause states that the load transmitted from a floor to a column in a column and beam construction or in a monolithic braced frame "may be calculated on the assumption that members framing into the column are simply supported". However, to avoid anomalies with more rigorous analysis or to reflect serviceability foundation loads more accurately, the effects of continuity should be considered in the determination of loads to columns from in-situ beams or slabs. Elastic reaction factors derived from continuous beam analysis are often used for columns and indeed this is the basis for in the figures given for erf above. But using these elastic reaction factors for columns can be shown to be onerous for internal columns and unconservative for perimeter columns.

Elastic analysis of a four-equal-span continuous beam shows that the factors approach 0.4 at end supports and 1.13 at internal end supports. By analysing this beam with columns of equal stiffness to the beam top and bottom, the factors become approximately 0.46 and 1.04 (A. W. Beeby, personal communication, 2003). Assuming full frame analysis with the same beam on three levels and columns of equal stiffness, the factors become 0.47 and 1.03. Using Finite Element analysis on a flat slab where the stiffness of the slab was approximately equal to the stiffness of the column, factors of 0.48 and 1.07 are indicated (A. W. Beeby, personal communication, 2003). It should be recognised that any method of analysis is approximate but will give answers that are in equilibrium. It is argued that there is sufficient ductility in the system near failure to allow redistribution of the forces to give a safe result: this is the argument used to justify BS 8110 Cl. 3.8.2.3. Axial shortening and sinking support will also play their part.

Nonetheless, some conservatism is recommended in total load be supported. Therefore, for the purpose of deriving column loads in in-situ structures, where rigorous methods are unavailable, and the structure is not unusual, it is recommended that in scheme design, an elastic reaction factor of 0.5 is applied to loads to perimeter columns and 1.10 to loads to internal columns.

In the case of post-tensioned construction, $P\Delta$ effects can cause considerable redistribution of loads. As indicated in Sections 5.12 and 8.3.2, reactions should be verified from analysis and design.

9 Concrete benefits

Figure 9.1
BDP offices, Manchester. Certified as a carbon neutral development, this six-storey 3000 m^2 building has an in-situ frame that provides substantial thermal mass.
Photo courtesy of BDP

9.1 Main design considerations

In the early stages of design, the four most important issues influencing the choice of frame type are:

- Cost
- Programme
- Performance in use
- Architecture

Although a concrete frame contributes typically to only around 10% of the cost of construction, choosing concrete can have a significant flow-on effect on the issues listed above and other areas of construction.

Sustainabilty is also an increasingly important issue in the choice of material.

9.2 Cost

Concrete frames can be constructed quickly and safely, and are competitive in most situations[25,26]. There are many aspects of cost to consider:

Initial costs

Driven by market forces, concrete frames are usually competitive. Recent studies[25,26] confirm that using concrete frames leads to marginally more economic buildings than those constructed with competing materials. Concrete frames also provide the inherent benefits of fire resistance, excellent acoustic and vibration performance, thermal mass and robustness – all at no extra cost.

Specialist concrete frame contractors have expertise that can reduce costs and maximise value when their input is harnessed early in the design process. Whenever possible, consider early (specialist) contractor involvement (ECI).

Foundation costs

As concrete is a heavy material, foundations to concrete framed buildings tend to be marginally more expensive than for those constructed of steel. However, this is more than offset by savings in other areas such as cladding, as illustrated below.

Cladding costs
Cladding can represent up to 25% of the construction cost, so the shallower floor and services zone of concrete solutions leads to lower floor-to-floor height and hence lower cladding costs.

Partition costs
Sealing and fire stopping at partition heads is simplest when using flat soffits, saving up to 10% of the partitions package compared with that for options with downstand beams. Even when rectangular concrete downstand beams are used, there are still savings over profiled downstand steel beams.

In service cores, structural concrete walls often take the place of what would otherwise be non-loadbearing stud partitions. However, the costs then show in the structural frame budget and savings in the partitions budget.

Services integration
Services distribution below a profiled slab costs more and takes longer than below the flat soffit of a concrete flat slab: a premium of 2% on M&E costs has been reported[26].

Finance costs
All other things being equal, in-situ concrete construction's 'pay as you pour' principle saves on finance costs – up to 0.3% of overall construction cost compared with steel-framed buildings[26].

Operating costs
Fabric energy storage means that concrete buildings that use their inherent thermal mass will have no or minimal air-handling plant. This reduces plant operating costs and maintenance requirements.

9.3 Programme

In overall terms, in-situ concrete-framed buildings are as fast to construct as steel-framed buildings: indeed, in some situations, they can be faster[25,26]. Sound planning will ensure that follow-on trades do not lag behind the structure. The following issues have an influence on programme times:

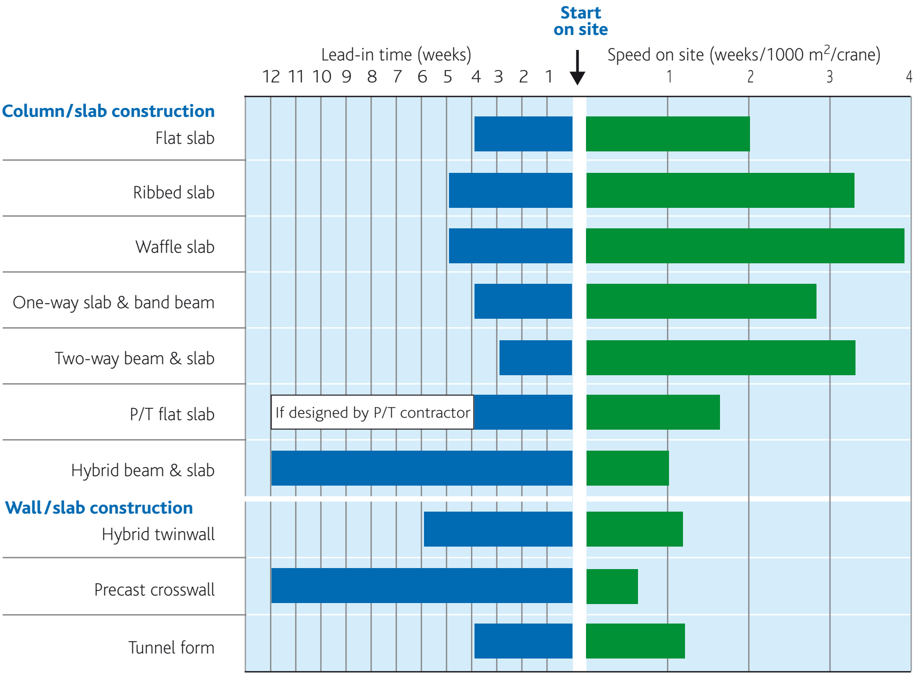
Speed of construction
As may be deduced from Figure 9.2, it is common to install 500 m^2 per crane per week, on reasonably large concrete flat slab projects. Even faster on-site programmes can be achieved by:

- Using greater resources.
- Post-tensioning of in-situ elements.
- Using precast elements or combinations of precast and in-situ (known as hybrid concrete construction).
- Rationalising reinforcement.
- Prefabricating reinforcement.
- Using proprietary reinforcement such as shear stud rails.

The prerequisite for fast construction in any material is buildability. This includes having a design discipline that provides simplification, standardisation, repetition and integration of design details.

Lead-in times
Generally, in-situ projects require very short lead-in times. The use of precast elements requires longer lead-in periods to accommodate design development, coordination and, where necessary, precasting. Contractor-led designs will generally lead to shorter overall construction times but the contractor will need additional lead-in time to mobilise, consider options, develop designs and co-ordinate with designers and subcontractors. Figure 9.2 shows these effects and also shows the possible effect of using a specialist post-tensioning (P/T) contractor for specialist design.

Figure 9.2
Typical speed of construction and lead-in times[32]



Note

Times and speeds shown here are typical for large projects and will vary, depending on size of project, availability of contractors and materials, and site constraints.

Liaison with specialist contractors

The use of enlightened specifications and, where appropriate, a willingness to adopt specialist contractors' methods, can have a significant effect on concrete construction programmes. Many contractors appreciate the opportunity to discuss buildability and influence designs for easier construction.

Managing progress

Improved speed of construction can be achieved by increasing resources. Whilst this option comes at a price, managing speed in this way is an attribute of concrete construction valued by many contractors.

Services integration

Flat soffits allow maximum off-site fabrication of services, higher quality of work and quicker installation. Openings in concrete slabs for service risers can be simply accommodated during design. Small openings can usually be accommodated during construction.

Accuracy

The overall accuracy of concrete framed buildings is not markedly different from other forms of construction. BS 5606[33] gives 95% confidence limits as follows:

Variation in plane for beams: concrete ± 22 mm, steel ± 20 mm
Position in plan: concrete ± 12 mm, steel ± 10 mm.

Late changes

The use of in-situ concrete allows alteration at a very late stage. However, this attribute should not be abused or productivity will suffer.

Striking times and propping

Allowances for striking times and propping are a part of traditional in-situ concrete construction. When critical to programme, specialist contractors, with the co-operation of designers, can mitigate their effects.

Safety

New methods, such as climbing panel protection systems that enclose two or three floors of work areas, provide safe and secure working environments at height. Panel formwork systems, which can be assembled from below, dramatically reduce the risk of falls. Concrete structures provide a safe working platform and semi-enclosed conditions suitable for follow-on trades.

Inclement weather

Modern methods of concrete construction can overcome the effects of wind, rain, snow, and hot or cold weather. Such events just need some planning and preparation.

Quality

Quality requires proper planning and committed management from the outset. Success depends on the use of quality materials and skilled and motivated personnel. Systems can be formally overseen by using Quality Assurance schemes such as SPECC[34]. It should be borne in mind that over-specification is both costly and wasteful.

9.4 Performance in use

Concrete frames provide performance benefits in the following areas:

Acoustics

When meeting the stringent amendments[35,36] to Part E of the Building Regulations[37], the inherent mass of concrete means the requirement for additional finishing to combat sound is minimised or even eliminated. This is illustrated by the results of independent testing which are given in Table 9.1. It is worth remembering that acoustic sealing of partition heads is most easily achieved with flat soffits.

Table 9.1
Acoustic tests summary[a]

Element	Structure	Finishes	Airborne sound insulation (min. 45 dB)[b]	Impact sound insulation (max. 62 dB)[b]
Floor	150 mm beam and block (300 kg/m^3)	Varying screeds, resilient layers and suspended ceilings	Pass	Pass
Floor	175 mm in-situ concrete	Specialist suspended ceiling	52 dB – Pass	60 dB – Pass
Floor	200 mm precast hollowcore concrete	65 mm screed on resilient layer. Ceiling 12.5 mm plasterboard on channel support	50 dB – Pass	
Floor	225 mm in-situ concrete	Bonded 5 mm carpet. Ceiling 15 mm polystyrene on aluminium exposed grids	59 dB – Pass	42 dB – Pass
Floor	250 mm in-situ concrete	Bonded 6 mm carpet on 50 mm screed. Painted ceiling	57 dB – Pass	39 dB – Pass
Wall	150 mm precast concrete	One side - 2 sheets of 12.5 mm plasterboard supported by channel system Other side - 1 sheet of 12.5 mm plasterboard supported by timber battens	51 dB – Pass	n/a
Wall	180 mm in-situ concrete		48 dB - Pass	n/a

Key
a www.concretecentre.com/main.asp?page=1405 (Dec. 2008) or search for acoustic tests summary
b From table 1c, Section 0, Approved Document E[36]

Adaptability

Markets and working practices are constantly changing, resulting in the need to adapt buildings. Flat soffits allow greater future modification of services and partition layouts. Concrete frames can easily be adjusted for other uses, and new service holes can be cut through slabs and walls relatively simply. If required, there are methods available to strengthen the frame if holes are required to be cut later.

Aesthetics

Fair-faced concrete can be aesthetically pleasing and durable, requiring little maintenance. However, special finishes do need careful attention in design, specification and construction to attain the desired result.

Airtightness

Part L of the Building Regulations requires pre-completion pressure testing. Concrete edge details are typically simple to seal to provide good airtightness; some projects have been switched to concrete frames on this criterion alone.

Corrosion

Corrosion of reinforcement is a potential problem only in concrete used in external or damp environments. Provided that the prescribed covers to reinforcement are achieved, and the concrete is of an appropriate quality, concrete structures should experience no corrosion (or durability) problems within the design life of the structure.

Deflections

Limiting deflections are generally given as span/250 for total deflection and span/500 for deflection after installation of non-structural items[27]. Codes do not give definitive limits, but the span/250 limit is implicit within Eurocode 2. Interaction with cladding may require the designer to assess deflection and to take appropriate measures.

Fire protection

Concrete provides inherent fire resistance[38]. It requires no additional fire protective coverings, chemical preservatives or paint systems that may release volatile organic compounds (VOCs), affecting internal air quality.

Long spans

Prestressing or post-tensioning becomes economic for spans over about 7.5 m, particularly if construction depth is critical.

Net lettable area

Net/gross area ratios are generally higher with concrete frames. Concrete structures tend to have shallower floor-to-floor heights, hence fewer steps between floors using less plan area. Also RC shear walls tend to be narrower than walls or partitions covering bracing in steel frames. In tall buildings, this compensates for generally larger concrete columns than those used in steel framed buildings. Using concrete's thermal mass can result in a reduction in HAV plant, which can free up plant space that can then become usable space. Overall, an increase of 1.5% in net area has been reported when using concrete frames[25]. Concrete construction permits shallow ceiling-to-finished-floor zones, particularly when using post-tensioned flat slabs. This attribute allows more storeys to be provided within overall height restrictions.

Robustness and vandal resistance

Reinforced concrete is very robust; it stands up to hard use, day after day. It is capable of withstanding both accidental knocks and vandalism, and has performed well in explosions. It is flood resistant and if inundated, it can be reinstated relatively quickly.

Thermal mass
Concrete frames offer a high degree of thermal mass that can be utilised to reduce heating and/ or air conditioning equipment and energy consumption.

Vibration control
The inherent mass and stiffness of concrete means that concrete floors generally meet vibration criteria without any change to the normal design. For some uses, such laboratories or hospitals with long spans, additional measures may need to be taken, but these are significantly less than those required for other materials[39].

9.5 Architecture

In addition to its cost, programme and performance attributes, concrete is an architectural material that provides for both form and function. It enables architectural vision to be realised efficiently and effectively. It can be engineered to be responsive to form, function and aesthetic to make the building work as a successful and coherent whole.

Concrete can have visual impact. It has the ability to appear massive and monolithic yet can be aesthetically refined. There is often a desire to express concrete's many visual qualities by using exposed concrete finishes – not only in the structure but also in the envelope, internal walls, stairs, ancillary areas and hard landscaping. There are very many possible finishes available, but to achieve the desired effect, visual concrete needs careful specification and care and attention in execution[40].

Spans between columns (or walls) usually dictate the most economic form of concrete construction. In-situ flat slabs are currently most popular for 'usual' spans and layouts. Other forms of construction such as post-tensioned flat slabs, troughed slabs, beam and slabs, precast beams and slabs, may suit longer spans, irregular layouts, greater speed or other key drivers for a specific project. Although costly, waffle slabs may be used for the visual appeal of the soffit. High quality plain soffits can be achieved using precast units, and attractive sculpted soffits can be created with bespoke precast concrete coffered floor units. Capable of being moulded into any size and shape, concrete's use in architecture is limited only by the imagination.

However, structures must have lateral stability to resist horizontal loads, including wind loads. Lateral stability is most easily provided by the inclusion of shear walls, which are usually arranged to be within core areas, for instance as lift shafts or as walls in stair wells. Taller structures may require more sophisticated solutions.

As Section 9.6 describes, concrete has many sustainability credentials. Concrete framed buildings provide quiet, durable and robust environments with long-term performance.

9.6 Sustainability

Concrete frames can withstand the impacts of climate change. They can be easily adapted to meet changing future requirements and they require minimal maintenance. They can withstand the impacts of climate change. At the end of their useful lives they can be demolished and recycled. Sustainability is a complex area encompassing economic, social and environmental aspects – the triple bottom line. Each aspect should be considered equally to ensure that a holistic approach is achieved.

9.6.1 Economy

Locally produced
The UK is self-sufficient in concrete and the materials needed to produce it. Indeed, the UK is a net exporter of concrete and concrete products[41]. Locally produced concrete provides local employment supporting local economies.

Competitive
When used in structures, concrete is a competitive construction material.

Thermal mass
Through fabric energy storage (FES), concrete's thermal mass can be used to regulate temperature swings. This can reduce initial plant expenditure and ongoing operational costs. Also it can free up plant space which can then be used as lettable space.

Maintenance
Except in exposed environments, concrete's maintenance requirements are minimal.

9.6.2 Society

Social contribution
Concrete contributes to the neighbourhood with its high sound insulation, thermal mass, fire resistance, robustness, durability and security, and the provision of local employment and leisure facilities.

Local environment
Local employment in a safe and healthy working environment supports local communities. Worked out quarries and pits are used for leisure and wildlife reserves.

Individual comfort
Many high thermal mass concrete buildings feature natural ventilation where increased airflow rates result in good air quality, which usually allows occupants control over their internal environment. This has been shown to improve productivity. Concrete is essentially inert and inherently fire resistant. It does not require toxic chemical treatments. As a high mass material, concrete is often the sole provider of sound insulation.

Longevity
As long as appropriate covers and concrete qualities are used, concrete offers intended working lives of 50 or 100 years. During this time-span, concrete structures can often be economically refurbished or reused.

Safety and security
Reinforced concrete can easily be made to comply with the normal robustness requirements in codes to resist accidental situations such as explosion. Concrete walls are acknowledged as being robust and secure against unlawful access.

9.6.3 Environment

There are many environmental indicators. When indicators such as emissions to air and use of land, energy and water are combined, concrete's overall environmental impact stood at just 2.1% of the UK total environmental impact for 2001[41].

Carbon dioxide
CO_2 emissions are of key concern. In the UK, construction of the built environment accounts for 7% of CO_2 emissions. Of this 2.6% results from the manufacture and delivery of concrete. This 2.6% figure should be compared with 47% emanating from use of the built environment and 24% from transport. It should also be considered in the light of the widespread and fundamental role that concrete plays in delivering the infrastructure and buildings that our society depends upon[42].

Energy use
For buildings, about 90% of the CO_2 environmental impact is from heating, cooling and lighting, and only about 10% is from the embodied energy used to produce the fabric of buildings (taken over a 60 year life-cycle). The 90% is being addressed through more energy efficient buildings

but will remain the bulk of a building's CO_2 impact. Furthermore, in efforts to help reduce the heating and cooling, concrete is seen as part of the solution: active fabric energy storage (FES) can reduce carbon dioxide emissions by up to 50% and can offset the additional embodied energy in heavyweight concrete structures in six years or less[41].

Cement

Cement making is an energy-intensive business, but the industry is committed to reducing greenhouse gas emissions. In recent years, there have been significant reductions in energy consumption and emissions of CO_2, nitrogen oxides, sulfur dioxide, particulate matter and dust. Every year the cement industry consumes over 1 MT of waste materials such as used tyres, household waste and waste solvents. Ready-mixed and precast plants are covered by strict environmental legislation which minimises the effects of manufacturing processes and factories on the environment.

Cement replacements

All construction materials have an environmental impact but that associated with concrete can be reduced by using ggbs (ground granular blastfurnace slag) or fly ash in combination with cement. These by-products from industrial processes reduce CO_2 embodied in the concrete. For example, cement combinations incorporating 50% ggbs will reduce embodied CO_2 of the concrete by some 40% compared with that when using a CEM I cement alone (see Table 9.2). In some exposure conditions cement combinations may be more appropriate than cement on its own. Indeed, the lower rate strength gain (and heat production) can be of benefit. However, in multi-storey structures, using combinations with more than 30% ggbs or fly ash may impact on the ability to strike formwork early.

Table 9.2
Embodied CO_2 (ECO_2) in typical concrete mixes

Concrete mix	ECO_2, kg CO_2/m^3			ECO_2, kg CO_2/tonne		
	CEM 1 concrete	30% fly ash concrete	50% ggbs concrete	CEM 1 concrete	30% fly ash concrete	50% ggbs concrete
GEN 1	173	124	98	75	54	43
RC 30	318	266	201	132	110	84
RC 35	315	261	187	133	110	79
RC 40	372	317	236	153	131	97
RC 50	436	356	275	176	145	112

Note
The above information was compiled in June 2007. The information is updated frequently as the industry continues to improve its processes; specifiers should refer to www.sustainableconcrete.org.uk for the latest information.

Aggregates

Concrete is 100% recyclable and so can be crushed for use as aggregate for new construction. The use of recycled concrete aggregate (RCA) in concrete is covered in BS 8500–2[4]. However, provenance, economic volumes and angularity (which affects the flow characteristics of concrete) often restrict its viability in structural grades of concrete. Recycled aggregates (RA) can be used in GEN prescribed mixes[4] and in small amounts in some structural grades. For guidance on the use of recycled aggregates in concrete please refer to Section 9.6.4. Government research[43] has found little evidence of hard demolition waste being land-filled – it is all being used.

Reinforcement

Reinforcement produced in the UK comes entirely from recycled UK scrap steel. The energy used producing 1 tonne of reinforcement is about half that used for 1 tonne of steel from ore. The majority of reinforcement used in the UK is made in the UK.

Concrete

Concrete is a local material. On average, there is an off-site ready-mixed concrete plant within ten miles of every UK construction site. Consequently, the energy and CO_2 emissions associated with transportation are relatively low.

Formwork

The timber used for formwork comes from renewable sources and as far as designs allow, formwork is used many times over. Steel formwork may be used hundreds of times.

9.6.4 Use of recycled aggregates

Many recycled and secondary aggregates (RSA) can be used as the constituents of concrete. In practice recycled aggregates (RA) and recycled concrete aggregates (RCA) are more commonly available and can form all or part of the coarse aggregate. However, as explained below, there are restrictions on structural use, but fewer restrictions exist when RCA has a known history.

The following definitions are given in BS 8500: *Concrete*[4]

- Recycled aggregate (RA) is aggregate resulting from the reprocessing of inorganic material previously used in construction.
- Recycled concrete aggregate (RCA) is recycled aggregate principally comprising crushed concrete.

Designated concrete

BS 8500 permits the use of coarse RCA in designated concrete as shown in Table 9.3, subject to the limits on exposure class given in Table 9.4.

Coarse RA may also be used, provided it can be shown that the material is suitable for the intended use. However, its use is not generally encouraged because the composition of RA is very variable and it is therefore difficult to adequately specify or test.

Table 9.3
Use of RA and RCA in BS 8500 for designated concrete

Designated concrete	Percentage of coarse aggregate in RA or RCA
GEN 0 to GEN 3	100%
RC20/25 to RC40/50	20%*
RC40/50XF	0%
PAV1 & PAV2	0%
FND2 to FND4	0%
Key * A higher proportion may be used if permitted in the (project) specification	

Table 9.4
Use of RCA in BS 8500 for designated concrete of different Exposure Classes

Exposure Class	Use of RCA permitted?
XO	Yes
XC1, XC2 & XC3/4	Yes
XD1, XD2 & XD3	Possibly*
XS1, XS2 & XS3	Possibly*
XF1	Yes
XF2, XF3 & XFa	Possibly*
DC–1	Yes
DC–2, DC–3 & DC–4	Possibly*
Key * RCA may be used if it can be demonstrated that it is suitable for the exposure condition	

Designed concrete

Coarse RCA and RA may also be specified for designed concrete. The specifier is responsible for ensuring that it is suitable for the intended use. BS 8500 allows fine RCA and fine RA to be used but again it is discouraged because of the difficulty in specifying and testing the requirements for such variable materials. The requirements for coarse RCA and RA are given in BS 8500–2.

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A handbook for the design of in-situ concrete buildings to Eurocode 2 and its UK National Annex

How to design concrete structures using Eurocode 2
CCIP-004, The Concrete Centre, 2006
Guidance for the design and detailing of a broad range of concrete elements to Eurocode 2

Eurocode 2: Worked examples Volumes 1 & 2
CCIP-041 & 042, The Concrete Centre, 2009 & 2010
Worked examples for the design of concrete buildings to Eurocode 2 and its National Annex

Precast Eurocode 2: Design manual
CCIP-014, British Precast Concrete Federation, 2008
A handbook for the design of precast concrete building structures to Eurocode 2 and its National Annex

Precast Eurocode 2: Worked examples
CCIP-034, British Precast Concrete Federation, 2008
Worked examples for the design of precast concrete buildings to Eurocode 2 and its National Annex

Properties of concrete for use in Eurocode 2
CCIP-029, The Concrete Centre, 2008
How to optimize the engineering properties of concrete in design to Eurocode 2

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Institution of Structural Engineers/ The Concrete Society, 2006
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The Concrete Centre – www.concretecentre.com

Institution of Structural Engineers – www.istructe.org

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Economic Concrete Frame Elements to Eurocode 2

This publication acts as a pre-scheme design handbook for the rapid sizing and selection of reinforced concrete frame elements in multi-storey buildings designed to Eurocode 2

Compared with frame designs to BS 8110, Eurocode 2 brings economies to most concrete frame elements. In order that these economies may be realised, this handbook is intended to give designers safe, robust and useful charts and data on which to base their scheme designs. The methodology behind the new charts and data is fully explained.

Charles Goodchild is Principal Structural Engineer for The Concrete Centre where he promotes efficient concrete design and construction. Besides project managing and co-authoring this publication he has undertaken many projects to help with the introduction of Eurocode 2 to the UK.

Rod Webster of Concrete Innovation and Design is the main author of the data in this publication and the spreadsheets on which they are based. Rod has been writing spreadsheets since 1984 and is expert in the design of tall buildings and advanced analytical methods.

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