ECONOMIC CONCRETE FRAME ELEMENTS

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

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REINFORCED CONCRETE

FOREWORD

This publication was commissioned by the Reinforced Concrete Council, which was set up to promote better knowledge and understanding of reinforced concrete design and building technology. The Council's members are Co-Steel Sheerness plc and Allied Steel & Wire, representing the major suppliers of reinforcing steel in the UK, and the British Cement Association, representing the major manufacturers of Portland cement in the UK. Charles Goodchild is Senior Engineer for the Reinforced Concrete Council. He was responsible for the concept and management of this publication.

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The ideas and illustrations come from many sources. The help and guidance received from many individuals are gratefully acknowledged on the inside back cover.

BS 8110 Pt 1:1997

The charts and data in this publication were prepared to BS 8110, Pt 1: 1985, up to and including Amendment No 4. During production, BS 8110 *Structural use of concrete:* Part 1:1997 *Code of practice for design and construction* was issued. This incorporated all published amendments to the 1985 version plus Draft Amendments Nos. 5 and 6. In general, the nett effect of the changes is that slightly less reinforcement is required: preliminary studies suggest 2 to 3% less in in-situ slabs and beams and as much as 10% less in columns. Readers should be aware that some of the tables in the new Code have been renumbered.

The charts and data given in this publication remain perfectly valid for pre-scheme design.

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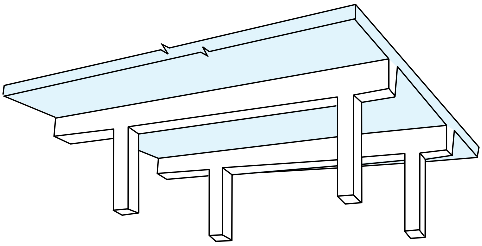

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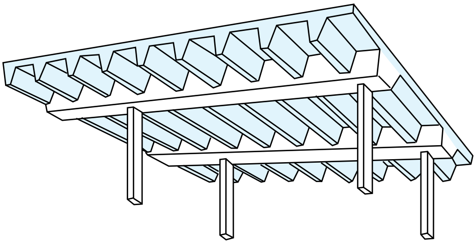

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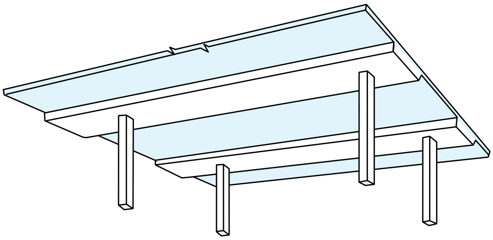

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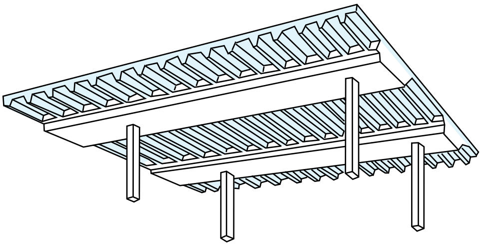

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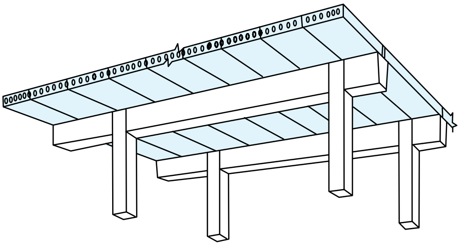

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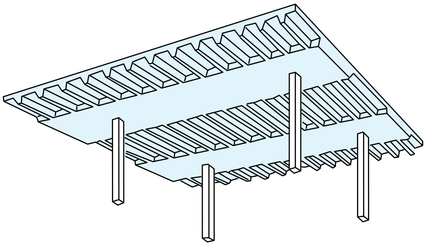

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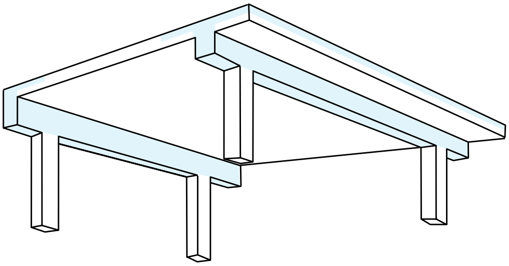

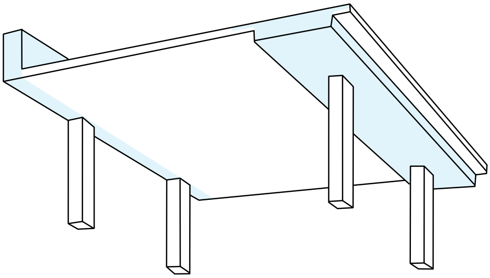

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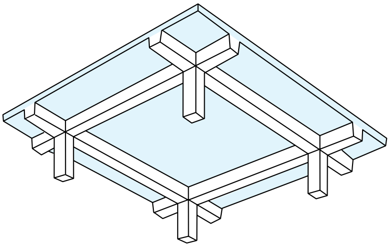

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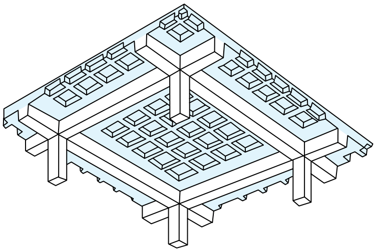

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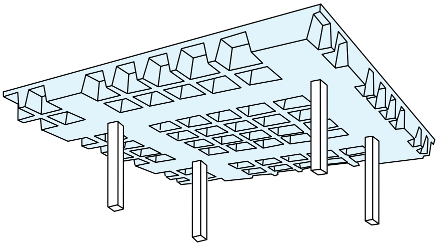

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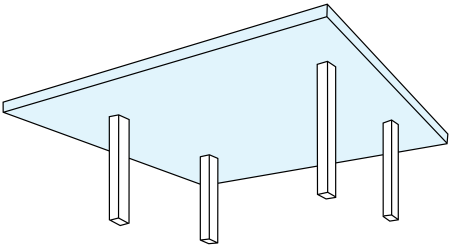

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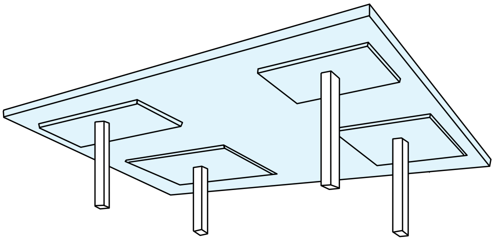

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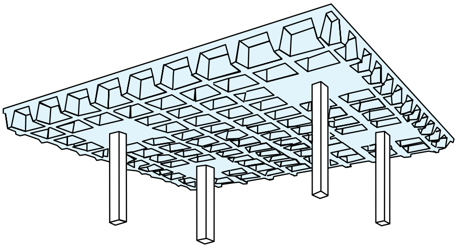

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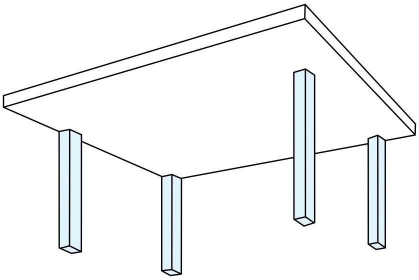

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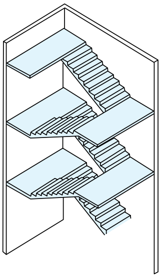

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1 INTRODUCTION

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. Its main objectives are, therefore, to:

- Present feasible, economic concrete options for consideration
- Provide preliminary sizing of concrete frame elements in multi-storey structures
- Provide first estimates of reinforcement quantities
- Outline the effects of using different types of concrete elements
- Help 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 (eg. by shear walls) and will be checked independently.

The charts and data work on loads:

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

FOR COLUMNS – Square sizes are plotted against **ultimate** axial load, and in the case of perimeter columns, according to number of storeys supported.

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

Thus a conceptual design can be built up by following load paths down the structure. This is the basis for *CONCEPT* (1), a complementary personal computer-based conceptual design program, available from the RCC.

Generally, the sizes given 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 BS 8110, *Structural use of concrete* (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 simple and idealised models (eg. for in-situ slabs and beams, they are based on moment and shear factors given in BS 8110). Engineers must assess the data in the light of their own experience, methods and concerns (4) and the particular 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 before an engineer sets forth with calculator or computer.

2 USING THE CHARTS AND DATA

2.1 General

The charts and data are intended to be used as follows.

		Refer
DETERMINE GENERAL DESIGN CRITERIA		
	● **Establish** layout, spans, loads, intended use, stability, aesthetics, service integration, programme, etc. **Identify worst case(s) of span and load.**	2.2, 2.3
SHORT-LIST FEASIBLE OPTIONS		
	● 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 2 and chart, p 8). Identify preferred structural solutions.	2.4
FOR EACH SHORT-LISTED OPTION:		
DETERMINE SLAB THICKNESS	● Interpolate from the appropriate chart or data, using the maximum slab span and the relevant **characteristic** imposed load, ie. interpolate between IL = 2.5, 5.0, 7.5 and 10.0 kN/m^2.	2.5, 2.11, 8.1
	● Make note of **ultimate** line loads to supporting beams (ie. characteristic line loads x load factors) or, in the case of flat slabs, troughed slabs, etc. **ultimate** axial loads to columns.	8.2
DETERMINE BEAM SIZES	● Estimate **ultimate** applied uniformly distributed load (uaudl) to beams by summing **ultimate** loads from: – slab(s), – cladding, – other line loads.	2.6, 2.11, 8.2
	● Choose the chart(s) 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. Adjust, if required, to account for elastic reaction factors.	8.3
DETERMINE COLUMN SIZES	● Estimate total **ultimate axial** load at lowest level, eg. multiply ultimate load per floor by the number of storeys.	2.7, 2.11,
	● Interpolate square size of column from the appropriate chart and/or data using the estimated total **ultimate axial** load, and in the case of perimeter columns, number of storeys.	8.3
IDENTIFY BEST VALUE OPTION(S)		
	● **Using engineering judgement, compare** and select the option(s) which appear(s) to be the best balance between structural and aesthetic requirements, buildability and economic constraints.	2.8
	● For cost comparisons, concentrate on floor plates. 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) and time.	
	● Visualize the construction process as a whole and the resultant impact on programme and cost.	2.9
PREPARE SCHEME DESIGN(S)		
	● Refine the design by designing critical elements using usual design procedures, making due allowance for unknowns.	2.10
	● Distribute copies of the scheme design(s) to all remaining design team members, and, whenever appropriate, members of the construction team.	

2.2 Limitations

2.2.1 GENERAL

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

2.2.2 ACCURACY

The charts and data have been prepared using spreadsheets which optimised on theoretical overall costs (see Section 7.1.1). Increments of 2 mm depth were used to obtain smooth curves for the charts (nonetheless some manual smoothing was necessary). The use of 2 mm increments is not intended to instill 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 282 mm thick slab, it is intended that the user would actually choose a 285, 290 or 300 mm thick slab, confident in the knowledge that a 282 mm slab would work. Going up to, say, a 325 mm thick slab might add 5% to the overall cost of structure and cladding but might be warranted in certain circumstances.

2.2.3 SENSITIVITY

At pre-scheme design, it is unlikely that architectural layouts, finishes, services, etc. will have been finalized. 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 (including tendons) densities. These are included for very preliminary estimates and 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 actual reinforcement quantities on specific projects. These include non-rectangular layouts, large holes, actual covers used, detailing preferences (curtailment, laps, wastage), and the unforseen complications that inevitably occur. Different methods of analysis alone can account for 15% of reinforcement weight. Choosing to use a 300 mm deep slab rather than the 282 mm depth described above could alter reinforcement tonnages by 10%.

The densities given in the data are derived from simple rectangular layouts, the RCC's interpretation of BS 8110, the spreadsheets (as described in Section 7), with allowances for curtailment (as described in BS 8110), and, generally, a 10% allowance for wastage and laps.

Additionally, in order to obtain smooth curves for the charts for narrow beams, ribbed slabs, troughed and waffle slabs, it proved necessary to use and quote densities based on $A_{s\ \mathrm{required}}$ rather than $A_{s\ \mathrm{provided}}$. It may be appreciated that the difference between these figures can be quite substantial for individual spans and loads.

2.2.5 COLUMNS

The design of columns depends on many criteria. In this publication, only axial loads and, to an extent, 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 (eg, by shear walls).**

2.3 General design criteria

2.3.1 SPANS AND LAYOUT

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. Pinned supports are assumed.

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.

2.3.2 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 use of moment and shear factors from BS 8110, Pt 1(2) is restricted to spans which do not differ by more than 15% of the longest span. The charts and data are likewise restricted. 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 BS 8110, Pt 2(3), Cl 3.7.2 assuming equal deflections in all spans, equal EI and $1/r_b \alpha M$)).

2.3.3 LOADS

Client requirements and, via BS 6399(5), occupancy or intended use usually dictate the imposed loads to be applied to floor slabs. Finishes, services, cladding and layout of permanent partitions should be discussed with the other members of the design team in order that allowances (eg superimposed dead loads for slabs) can be determined. See Section 8.

2.3.4 INTENDED USE

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

2.3.5 STABILITY

Means of achieving lateral stability (eg. using core or shear walls or frame action) and robustness (eg. 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.**

2.3.6 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. 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 BS 8110 or, for precast elements, to manufacturers' recommendations.

Exposure is defined in BS 8110, Pt 1(2) as follows:

Mild – *concrete surfaces protected against weather or aggressive conditions.*

Moderate – *concrete sheltered from driving rain; concrete sheltered from freezing while wet; concrete subject to condensation; concrete continuously under water and/or concrete in contact with non-aggressive soils.*

Severe – *concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing, or severe condensation.*

2.3.7 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 'T's; or by using combinations of precast and in-situ concrete. Each method has implications on the standard of finish and cost.

2.3.8 SERVICE INTEGRATION

Services and structural design must be co-ordinated.

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, etc. These holes may significantly affect the design of slabs, eg. 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

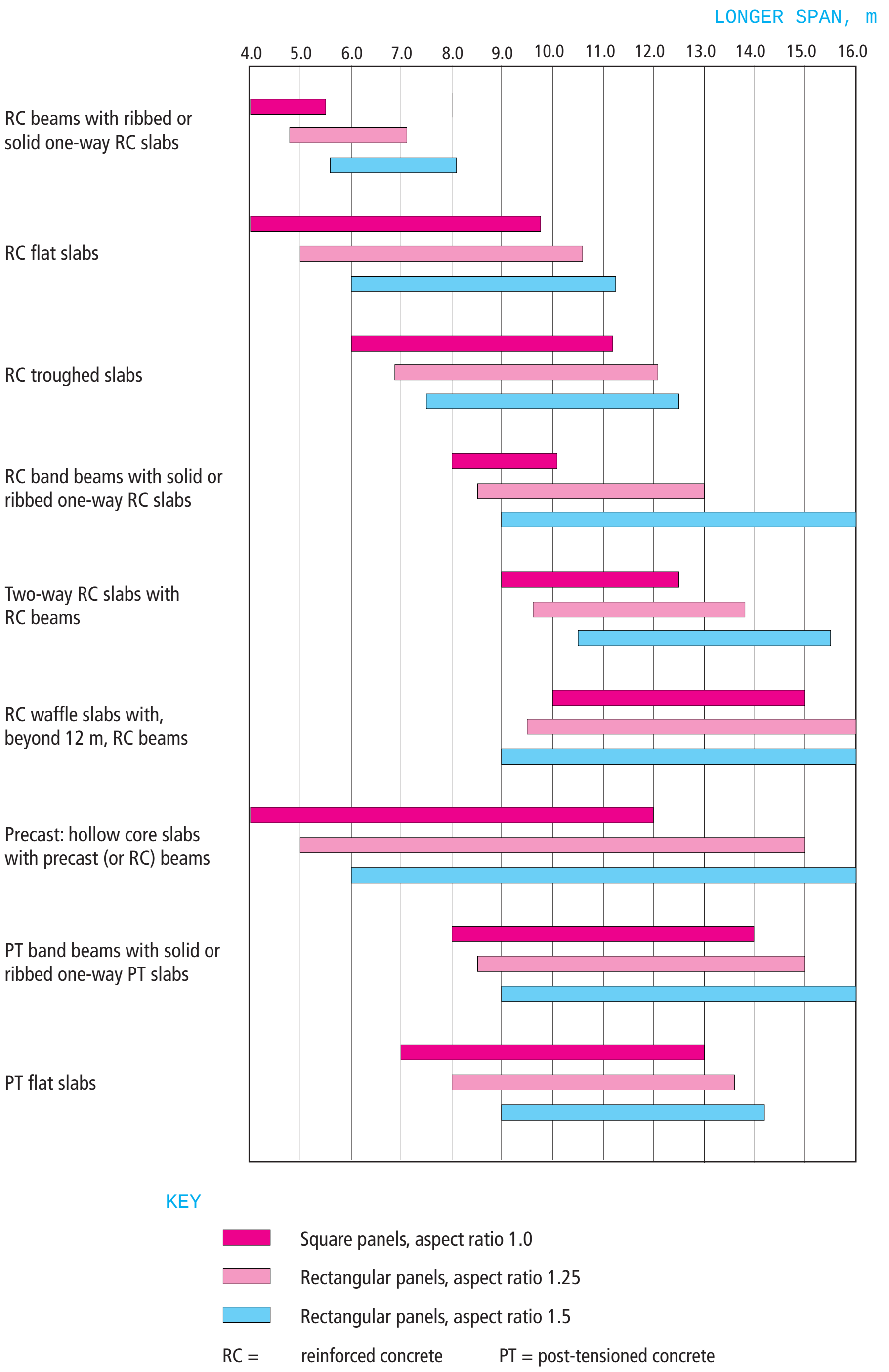
2.4.1 GENERAL PRINCIPLES

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 on page 8 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 THE CHOICE

Concrete floor slabs: typical economic span ranges



Note: All subject to market conditions and project specific requirements

Briefly, the main differences between types of construction may be summarised as follows:

One-way slabs (solid or ribbed)
Economic over wide range 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 - 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 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 slabs and beams
Extend the economic span range of in-situ slabs and beams, especially useful where depth is critical.

2.4.3 HYBRIDS

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[(7)] 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.

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 charts illustrate thicknesses given in the data. 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 6399, *Design loadings for buildings*, Pt 1(5), 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

The charts and data assume 1.50 kN/m^2 for superimposed dead loading (SDL). If the actual 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 the following table. See Section 8.1.

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	1.2	2.1	2.9	3.8	4.7	5.6
5.0	3.7	4.6	5.4	6.3	7.2	8.1
7.5	6.2	7.1	7.9	8.8	9.7	10.6
10.0	8.7	9.6	10.4	11.3	12.2	n/a

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 and flat slabs with rectangular panels. The user may, with caution, interpolate from this information.

2.6 Determine beam sizes

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

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. This load includes the ultimate reaction from slabs and ultimate applied line loads such as cladding or partitions which are to be carried by the beam. Self-weight of beams is allowed for within the beam charts and data. See Section 8.2.

For internal beams, this 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 (eg. two-span rather than three-span assumed – see Section 8.2.2).

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 8.2.3.

The user can interpolate between values given in the charts 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. See Section 8.2.4.

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, kN/m.

2.6.1 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: eg. isolated or upstand beams, beams with no flange, beams not homogeneous with supported slabs

Inverted 'L' beams: eg. perimeter beams with top flange one side of the web

'T' beams: eg. 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 derated from a 'T' to an 'L' or even to a rectangular beam.

2.6.2 PRECAST BEAMS

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

Rectangular beams: ie. isolated or upstand beams

'L' beams: eg. perimeter beams supporting hollow core floor units

(Inverted) 'T' beams: eg. internal beams supporting hollow core floor units

The charts assume that the beams are simply supported and non-composite, ie. no flange action or benefit from temporary propping is assumed. The user must determine which form of beam is appropriate.

2.6.3 POST-TENSIONED BEAMS

The first set of charts for post-tensioned beams assumes 1000 mm wide rectangular beams with no flange action. Other post-tensioned beam widths can be investigated on a pro-rata basis, ie. ultimate load per metre width of web (see Section 8.2.5). Additionally data are presented for 2400 mm wide 'T' beams assuming full flange action.

2.7 Determine column sizes

The charts are divided into internal, edge and (external) corner columns at different percentages of reinforcement contents. The square size of column required can be interpolated from the appropriate chart(s) using the total **ultimate axial** load at the lowest level and, in the case of perimeter columns, number of storeys supported.

The total **ultimate axial** load, N, is the summation of beam (or two-way floor system) reactions 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 8.3). In accordance with BS 6399, table 2, live loads might be reduced. However, to do so is generally unwarranted in pre-scheme design of low-rise structures. Sufficient accuracy can be obtained by approximating the load to be as follows:

N = {(ult. load from beams per level or ult.
load from two-way slab system per level)
+ ultimate self-weight of column per level}
x no. of floors

For schemes using beams
Beams reactions can be read or interpolated from the data for beams. Reactions in two orthogonal directions should be considered, eg. perimeter columns may provide end support for an internal beam and internal support for a perimeter beam. Usually the weight of cladding will have been allowed for in the loads on perimeter beams (see Section 8.2). If not, or if other loads are envisaged, due allowance must be made.

For schemes using two-way floor systems
Two-way floor systems (ie. flat slabs, troughed slabs and waffle slabs designed as two-way slabs with integral beams and level soffits) either do not require beams or else include prescribed beams. Their data include ultimate loads or reactions to supporting columns. These loads assume a cladding load of 10 kN/m (ie. 14 kN/m ultimate). NB: some reactions are expressed as mega-newtons (MN, ie.1000 kN).

Roofs
Other than in areas of mechanical plant, roof loadings seldom exceed floor loadings. For the purposes of estimating column loads, loads from concrete roofs may be equated to those from a normal floor, and 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 Identify best value option(s)

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 time scales, cladding and foundation costs should be included when determining best value and the most appropriate option(s) for further study.

2.9 Visualize the construction process

Imagine how the structure will be constructed. Consider buildability and the principles of value engineering. Consider time-scales, 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.

2.10 Prepare scheme design(s)

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, eg. the architect for co-ordination 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 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/m², and superimposed dead load of 3.2 kN/m²

From Section 2.5 or 8.1, equivalent imposed load is estimated to be 4.0 kN/m². From chart (p 16), depth required is estimated to be 220 mm.

Alternatively, interpolating from one-way solid slab data (p 17), multiple span, at 4 kN/m², between 2.5 (208 mm) and 5 kN/m² (226 mm), then:

thickness = 208 + (226 - 208) x (4.0 - 2.5)/(5.0 - 2.5)
= 208 + 18 x 0.6
= 219 mm, say, 220 mm

Answer: 220 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.

Interpolating from one-way solid slab data (p 17), multiple span, at 4 kN/m², between 2.5 (101 kN/m) and 5 kN/m² (136 kN/m), then:

load = 101 + (4.0 - 2.5) x (136 - 101)/(5.0 - 2.5)
= 122 kN/m

This value assumes an elastic reaction factor of 1.1 is appropriate (see Section 8.2.2). Interpolating from the chart for, say, a 'T' beam web 900 mm wide multiple span (p 68) at 8.0 m span and between loads of 100 kN/m (408 mm) and 200 kN/m (586 mm, singly reinforced), then:

depth = 408 + (586 - 408) x (122 - 100)/(200 - 100)
= 408 + 39
= 447 mm

Answer: say, 900 mm wide by 450 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 weighs 3.0 kN/m (characteristic) per storey.

For perimeter beam perpendicular to slab span. Interpolating end support reaction from one-way solid slab chart and data (p 17), multiple span, at 4 kN/m², between 2.5 (46 kN/m) and 5 kN/m² (62 kN/m), then:

load from slab = 46 + (4.0 - 2.5) x (62 - 46)/(5.0-2.5)
= 56 kN/m

load from cladding = 3 x 1.4
= 4.2 kN/m

Total load = 56 + 4.2
= 60.2, say, 60 kN/m

Beam size: interpolating from 'L' beam chart and data, multiple span, say, 450 mm web width (p57), at 60 kN/m over 8 m. At 50 kN/m suggested depth is 404 mm; at 100 kN/m (662 mm), then:

depth required = 404 + 20% x (662 - 404)
= 456 mm

For perimeter beams parallel to slab span.
Allow, say, 1.0 m of slab, then:

load from slab = (0.22 x 24 + 3.2) x 1.4 + 2.5 x 1.6
= 15.9 kN/m

load from cladding = 4.2 kN/m

Total load = 20.1 kN/m

Beam size: reading from 'L' beam chart and data, multiple span, say, 225 mm web width, at 25 kN/m over 7.0 m, suggested depth is 360 mm.

Answer: for edges perpendicular to slab span, use 450 x 460 mm deep edge beams; for edges parallel to slab span, 225 x 360 mm deep edge beams can be used. For simplicity, use 450 x 460 mm deep, say, 450 x 450 mm deep edge beams all round.

Commentary: for buildability, a wider shallower beam might be more appropriate.

2.11.4 COLUMNS

Estimate the column sizes for the above examples assuming a three-storey structure and floor-to-floor height of 3.5 m.

Loads

Beam reactions by interpolating data (pp 68 and 60)

	Internal support reaction	End support reaction
Internal beams 900 x 450 mm deep 122 kN/m, 8.0m span	1035 kN#	518 kN
Perimeter, perpendicular to slab span 450 x 450 mm deep 60kN/m, 8.0 m span	523kN	261kN
Perimeter, parallel to slab span 450 x 450 mm deep Self weight and cladding 11 kN/m, 7.0 m span	say 77 kN	say 40 kN

Note:
\# Figure interpolated from data and no adjustment made for elastic reactions (see Section 8.3.2). Alternatively, this load may be calculated:

span x uaudl (see 2.11.2) = 8 x 122 = 976 kN
self-weight
= 0.9 x (0.45-0.22) x 8 x 24 x 1.4 = <u>56 kN</u>
Total = 1032 kN

Self-weight of column
Assume 450 mm square columns and 3.5 m storey height, from table in Section 8.3.3, allow 25 kN or calculate:
0.45 x 0.45 x 3.5 x 24 x 1.4 = 23.8kN, say, 25 kN/floor

Total ultimate axial loads in the columns:
Internal
(1035 + 0 + 25) kN x 3 storeys = 3180 kN, say, 3200 kN.
Edge L'r to slab span
(523 + 0 + 25) x 3 = 1644 kN, say, 1650 kN.
Edge II to slab span
(77 + 518 + 25) x 3 = 1860 kN, say, 1900 kN.
Corner
(261 + 40 + 25) x 3 = 978 kN, say, 1000 kN.

Estimating column sizes from charts

Internal columns, p 74, for 3200 kN
A 440 mm square column would require approximately 1% reinforcement. A 395 mm square column would require approximately 2% reinforcement. Try 400 mm square with 2% reinforcement provided by (from p 75) 8T25s, approximately 285 kg/m^3.

Edge columns, pp 76 and 77, for 1900 kN over 3 storeys
Estimated sizes: 535 mm square @ 2% or 385 mm square @ 3%. Try 450 mm square with 2.6% reinforcement provided by (from p 80) 12T32s, approximately 536 kg/m^3.

Corner columns, pp 78 and 79, for 1000 kN over 3 storeys
Estimated sizes: 530 mm square @ 2% or 435 mm square @ 3%. Try 450 mm square @ 2.8% reinforcement, 12T32s as above.

Answer: suggested column sizes:
internal 400 mm square
perimeter 450 mm square

Commentary: the perimeter columns are critical to this scheme option. 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 rationalized to, say, 450 mm square, without the need for undue amounts of reinforcement.

Perimeter beams would be rationalized at 450 wide, to match perimeter columns, by 450 mm deep. Internal beams would be 900 mm wide and 450 mm deep.

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 1.5 kN/m^2. Curtain wall glazing is envisaged. Approximately how much reinforcement would there be in such a superstructure?

Slab
Interpolating from the solid flat slab chart and data, p 37, at 5.0 kN/m^2 and 7.5 m, the slab should be 282, say,

285 mm thick with approximately 109 kg/m^3 of reinforcement.

Columns

The minimum square sizes of columns should be 400 mm (from p 37, at 5.0 kN/m^2, average of 370 mm at 7 m and 430 mm at 8 m) internally and 355 mm (from p 37, average of 330 mm at 7 m and 380 mm at 8 m) around the perimeter to avoid punching shear problems.

From the flat slab data, ultimate load to **internal** column is 1.1 MN, ie. 1100 kN per floor. Allowing 25 kN/floor for ultimate self-weight of column, total axial load = (1100 + 25) x 7 = 7875 kN. From internal column chart, p 74, at 8000 kN, the internal columns could be 600 mm square, ie. greater than required to avoid punching shear problems. They would require approximately 2.5% reinforcement, ie. from p 75, 12T32s, about 318 kg/m^3, including links.

From the flat slab data, ultimate load to **edge** columns is 0.7 MN, ie. 700 kN per floor. This includes a cladding load of 10 kN/m whereas 2.0 kN/m might be more appropriate. Therefore deduct (10.0 - 2.0) x 7.5 x 1.4 = 84 kN ultimate per floor. Allowing 25 kN/floor for ultimate self-weight of column, total axial load = (700 + 25 - 84) x 7 = 4487 kN. Interpolating from edge column charts, pp 76 and 77, at 4500 kN and at seven stories, the edge columns could be 565 mm square at 2% reinforcement or 475 mm square at 3%.

Checking **corner** columns: load per floor will be approximately:

Floor less cladding
= (700 -10 x 7.5 x 1.4)/2 = 298 kN/floor
Cladding = 2 x 7.5 x 1.4 = 21 kN/floor
Self-weight, say, = 25 kN/floor
344 kN/floor
Total load = 344 x 7 = 2408 kN

From corner column charts at 2400 kN, pp 78 and 79, these columns could be 555 mm square at 2% reinforcement or 460 mm at 3%.

For the sake of buildability, make all perimeter columns the same size as internal columns, ie. 600 mm square. This size avoids punching shear problems, and would require approximately 1.8% (effective) reinforcement. From the chart on p 80, allow for 12T32s, at a density of 318 kg/m$^{3.}$

Walls

From p 112 assuming 200 mm thick walls, reinforcement density is approximately 80 kg/m^3.

Stairs

From p 113 say 5 m span and 4.0 kN/m^2 imposed load, reinforcement density is approximately 30 kg/m^2 (assume landings included with floor slab estimate).

Reinforcement

Slabs =
(7.5 x 5 + 0.6)^2 x 7 x 285/1000 x 109/1000 = 316 t
Columns =
0.6 x 0.6 x 3.3 x 6 x 6 x 7 x 318/1000 = 95 t
Walls, say, =
41 x 3.3 x 0.2 x 7 x 80 /1000 = 15 t
Stairs, say, =
30 flights x 5 x 1.5 x 30 / 1000 = 8 t
Plant roof, say, =
7.5 x 7.5 x 3 x 1 x 0.282 x 109/1000 = 5 t
Plant room columns, say, =
0.6 x 0.6 x 3.3 x 8 x 318/1000 = 3 t
Total, approximately = 442 t

Answer: use 285 mm flat slabs and 600 mm square columns throughout. Reinforcement quantities for the superstructure would be in the order of 445 tonnes.

Commentary: this example is based on the M4C7 building in the RCC's Cost Model Study(6) which used 300 mm thick flat slabs and 700 mm square columns. The estimated tonnage of of reinforcement in the superstructure was 452 tonnes. Further work on the Cost Model Study indicated that a 285 mm slab gives the least-cost solution (albeit with little scope for further design development).

More detailed analysis (including live load reduction) revealed that internal columns could be 500 mm square at 3.4% reinforcement (12T32s) and perimeter columns 450 mm at 2.1% (8T32s)

Combined Operations Centre, Heathrow, under construction

3.1 Slabs

3.1.1 USING IN-SITU SLABS

In-situ slabs offer economy, versatility, mouldability, fire resistance, sound attenuation, thermal capacity and 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 USING 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 the slabs have line support (ie. 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 (ie. flat slabs, troughed slabs and waffle slabs designed as two-way slabs with integral beams) do not, generally, need separate consideration of beams. In these cases, the ultimate load to supporting columns is given. An allowance of 10 kN/m characteristic load has been made 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 8.2.3.

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 moment and shear factors in BS 8110, Pt 1(2) tables 3.6 and 3.13 assuming end spans are critical.

In order to satisfy defection criteria, service stress, f_s, is, in very many cases, reduced (to as low as 200 N/mm^2) by increasing steel contents.

Reinforcement

Main reinforcement, $f_y = 460$ N/mm^2. Links, $f_y = 250$ N/mm^2.

For reinforcement quantities, see Section 2.2.4.

Concrete

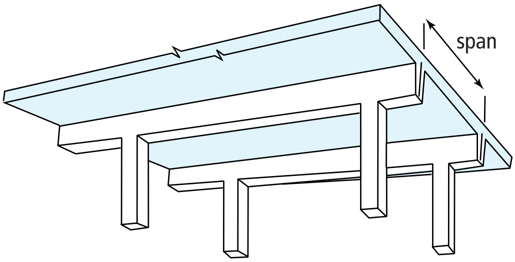
C35, 24 kN/m^3, 20 mm aggregate.

Fire and durability

Fire resistance 1 hour; mild exposure.

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.*

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 office buildings, retail developments, warehouses, stores, etc. Can be economical for spans from 4 to 8 m.

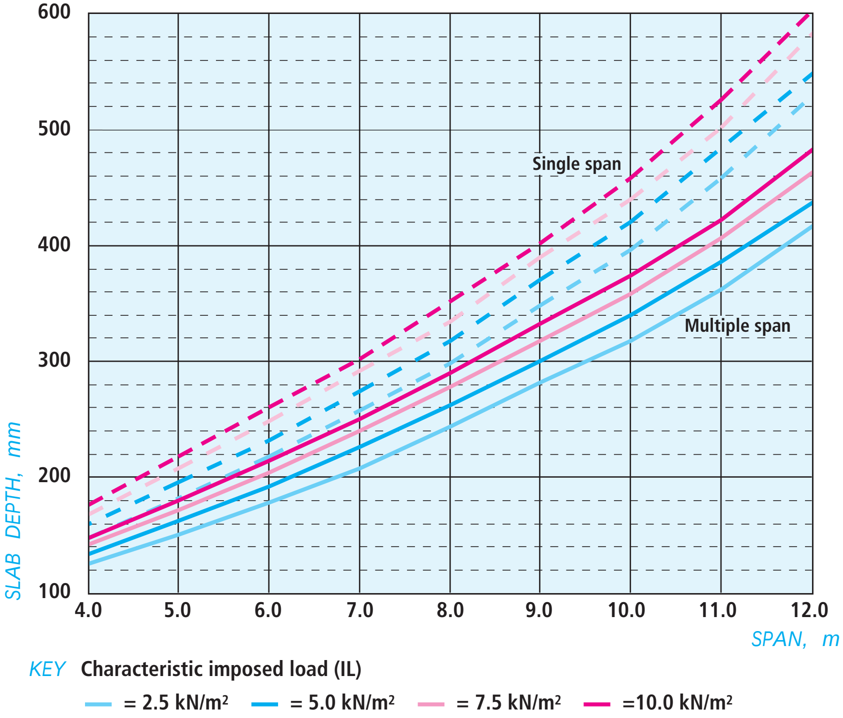
ADVANTAGES

- Simple
- Holes cause few structural problems

DISADVANTAGES

- Associated downstand beams may require greater storey height, deter fast formwork cycles and compromise flexibility of partition location and horizontal service distribution

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	BEAMS. Refer to beam charts and data to estimate sizes. End supports min 300 mm wide.
REINFORCEMENT	<6.5 m:T16T&B, >6.5 m: T20T&B uno. T10 @ 300 distribution. 10% allowed for wastage and laps. To comply with deflection criteria, service stress, f_s, may have been reduced. No A_sT in midspan.
LOADS	A superimposed dead load (SDL) of 1.50 kN/m² (for finishes, services, etc.) is included. Ultimate loads assume elastic reaction factors of 0.5 to supports of single spans, 1.1 to internal supports and 0.46 to end supports of multiple span continuous slabs.
CONCRETE	C35, 24 kN/m³, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.

SINGLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m²	148	182	218	258	298	348	396	458	528
IL = 5.0 kN/m²	160	196	232	274	318	370	420	484	548
IL = 7.5 kN/m²	168	208	248	292	334	390	440	502	582
IL = 10.0 kN/m²	176	218	260	302	352	402	458	526	602
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m²	n/a (22)	n/a (31)	n/a (40)	n/a (52)	n/a (64)	n/a (80)	n/a (96)	n/a (118)	n/a (143)
IL = 5.0 kN/m²	n/a (31)	n/a (42)	n/a (54)	n/a (68)	n/a (83)	n/a (102)	n/a (120)	n/a (145)	n/a (171)
IL = 7.5 kN/m²	n/a (39)	n/a (53)	n/a (67)	n/a (84)	n/a (101)	n/a (122)	n/a (143)	n/a (170)	n/a (202)
IL = 10.0 kN/m²	n/a (48)	n/a (64)	n/a (81)	n/a (99)	n/a (120)	n/a (142)	n/a (167)	n/a (197)	n/a (230)
REINFORCEMENT, kg/m² (kg/m³)									
IL = 2.5 kN/m²	14 (95)	16 (90)	19 (89)	23 (89)	26 (89)	30 (86)	34 (88)	39 (85)	45 (85)
IL = 5.0 kN/m²	15 (96)	18 (94)	23 (99)	27 (98)	29 (92)	33 (89)	38 (90)	43 (88)	51 (93)
IL = 7.5 kN/m²	18 (106)	20 (95)	24 (96)	28 (96)	32 (97)	36 (93)	43 (99)	47 (93)	51 (88)
IL = 10.0 kN/m²	19 (108)	21 (98)	25 (98)	31 (104)	33 (95)	40 (100)	43 (95)	48 (92)	54 (90)

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

Fire resistance	2 hours	+20 mm	4 hours	+40 mm
Exposure	Moderate	+15 mm	Severe, C40 concrete	+25 mm

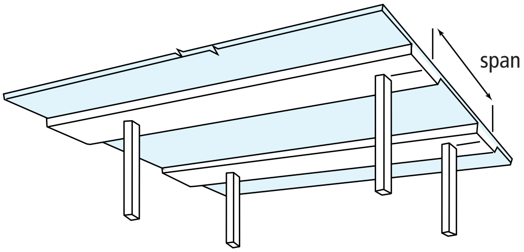
MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m²	125	150	178	208	244	282	318	362	416
IL = 5.0 kN/m²	134	162	192	226	262	300	340	386	438
IL = 7.5 kN/m²	142	172	204	240	278	318	358	406	462
IL = 10.0 kN/m²	148	180	214	250	290	332	374	422	482
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m²	45 (21)	61 (28)	80 (36)	101 (46)	125 (57)	154 (70)	183 (83)	221 (100)	265 (120)
IL = 5.0 kN/m²	64 (29)	85 (39)	109 (50)	136 (62)	165 (75)	200 (91)	235 (107)	279 (127)	324 (147)
IL = 7.5 kN/m²	83 (38)	109 (50)	138 (63)	171 (78)	205 (93)	245 (112)	285 (130)	334 (152)	391 (178)
IL = 10.0 kN/m²	102 (46)	133 (60)	167 (76)	204 (93)	244 (111)	290 (132)	335 (152)	391 (178)	453 (206)
REINFORCEMENT, kg/m² (kg/m³)									
IL = 2.5 kN/m²	10 (84)	12 (83)	14 (80)	17 (82)	19 (80)	22 (78)	25 (80)	29 (81)	33 (79)
IL = 5.0 kN/m²	12 (87)	14 (86)	16 (84)	19 (83)	22 (84)	25 (83)	28 (84)	32 (83)	39 (90)
IL = 7.5 kN/m²	13 (90)	15 (88)	18 (86)	20 (85)	23 (85)	27 (84)	31 (87)	35 (88)	39 (84)
IL = 10.0 kN/m²	14 (95)	17 (92)	19 (89)	22 (90)	26 (89)	29 (87)	33 (90)	37 (88)	41 (86)
DESIGN NOTES			*a = imposed load, q_k,> 1.25 dead load, g_k*				*b = q_k > 5 kN/m²*		*g = T25s used*
IL = 2.5 kN/m²								g	g
IL = 5.0 kN/m²							g	g	g
IL = 7.5 kN/m²	a b	a b	b	b	b	bg	bg	bg	bg
IL = 10.0 kN/m²	a b	a b	a b	a b	b	bg	bg	bg	bg

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

Fire resistance	2 hours	+5 mm	4 hours	+25 mm
Exposure	Moderate	+15 mm	Severe, C40 concrete	+25 mm

One-way slabs *for use with 2400 mm wide band beams only*

(One-way slabs with wide beams)



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

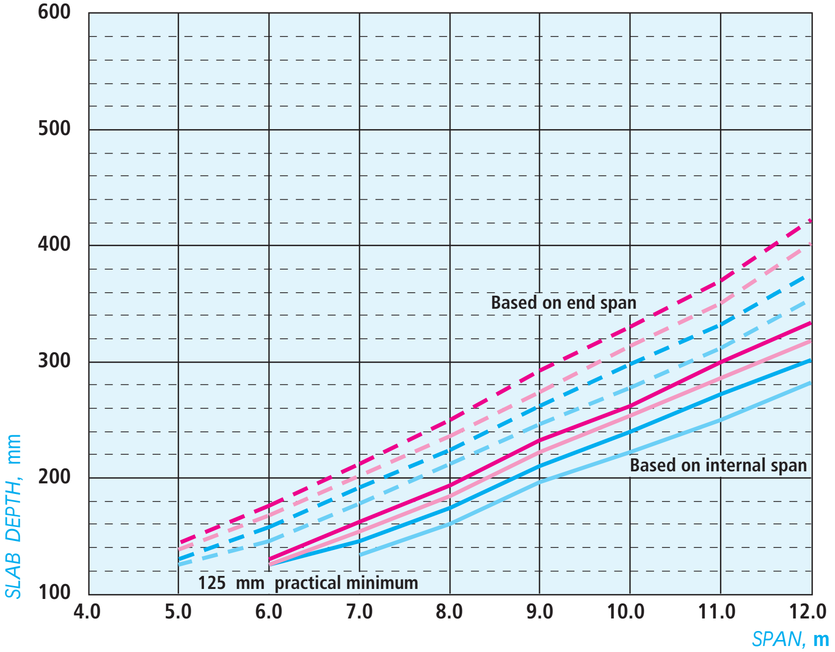
Slabs effectively span between edges of the relatively wide and shallow band beams; slab depth and overall depth of floor are thus minimized. Perimeter beams often take the form of upstands.

Economic for slab spans up to 9 m (centreline support to centreline support) and band beam spans up to 15 m in reinforced concrete (see pp 64 and 71) or up to 18 m using post- tensioned concrete (see pp 110 and 111). Thicknesses are typically governed by deflection and, to suit formwork, by ideally restricting the downstands of beams to 150 mm.

ADVANTAGES

- Medium range spans
- Simple
- Large and small holes can be accommodated
- Fast
- Amenable to simple distribution of horizontal services

SPAN:DEPTH CHART



KEY **Characteristic imposed load (IL)**

▬ = 2.5 kN/m² ▬ = 5.0 kN/m² ▬ = 7.5 kN/m² ▬ =10.0 kN/m²

DESIGN ASSUMPTIONS

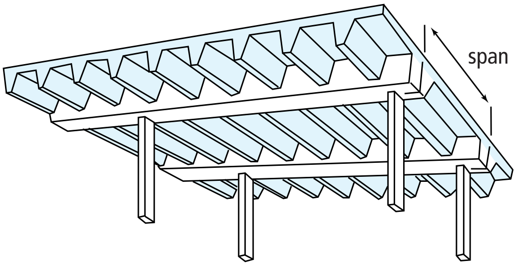
SUPPORTED BY	BEAMS. Internally, 2400 mm wide BEAMS. Refer to beam charts to estimate sizes.
DIMENSIONS	Square panels, minimum of two (for end spans) or three slab spans x three beam spans
SPANS	Spans quoted in charts and data are centreline support to centreline support (eg. grid to grid). However, the designs of these slabs are based on spans of end span - 1.2 m + d/2, or internal span - 2.4 m + d.
REINFORCEMENT	<7.5 m:T16T&B, >7.5 m: T20T&B uno. T10 @ 300 distribution. 10% allowed for wastage and laps. To comply with deflection criteria, service stress, f_s, may have been reduced. No A_sT in midspan.
LOADS	A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. Ultimate loads assume elastic reaction factors of 1.1 to internal beams and 0.5 to end beams.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.

BASED ON END SPAN, m	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm	*Add minimum 100 mm for minimum depth of 2400 spine beam*							
IL = 2.5 kN/m^2	125	146	178	212	246	278	312	354
IL = 5.0 kN/m^2	130	158	192	224	262	298	332	376
IL = 7.5 kN/m^2	138	168	202	236	274	314	350	402
IL = 10.0 kN/m^2	144	176	212	250	292	330	370	422
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m								
IL = 2.5 kN/m^2	56 (25)	73 (33)	93 (42)	115 (52)	142 (65)	168 (77)	201 (91)	238 (108)
IL = 5.0 kN/m^2	80 (36)	102 (46)	127 (58)	155 (70)	187 (85)	221 (101)	257 (117)	300 (136)
IL = 7.5 kN/m^2	103 (47)	130 (59)	160 (73)	194 (88)	231 (105)	271 (123)	313 (142)	364 (166)
IL = 10.0 kN/m^2	126 (57)	158 (72)	194 (88)	233 (106)	276 (126)	321 (146)	368 (167)	426 (194)
REINFORCEMENT, kg/m^2 (kg/m^3)								
IL = 2.5 kN/m^2	9 (78)	12 (79)	13 (74)	16 (77)	19 (77)	23 (83)	24 (78)	30 (84)
IL = 5.0 kN/m^2	11 (81)	13 (83)	15 (78)	18 (81)	22 (83)	24 (81)	28 (83)	33 (89)
IL = 7.5 kN/m^2	12 (84)	14 (85)	18 (88)	20 (84)	25 (91)	27 (85)	30 (87)	35 (86)
IL = 10.0 kN/m^2	13 (89)	16 (89)	19 (88)	21 (87)	25 (87)	29 (86)	33 (89)	37 (87)
DESIGN NOTES	*a = imposed load, q_k,> 1.25 dead load, g_k*				*b = q_k > 5 kN/m^2*		*g = T25s used*	
IL = 2.5 kN/m^2						g	g	g
IL = 5.0 kN/m^2						g	g	g
IL = 7.5 kN/m^2	a b	a b	b	b	b	bg	bg	bg
IL = 10.0 kN/m^2	a b	a b	a b	a b	b	bg	bg	bg

BASED ON INTERNAL SPAN, m	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm	*Add minimum 150 mm for minimum depth of 2400 spine beam*							
IL = 2.5 kN/m^2			134	160	196	222	250	282
IL = 5.0 kN/m^2		125	146	174	210	240	272	302
IL = 7.5 kN/m^2		125	154	184	222	254	286	318
IL = 10.0 kN/m^2		130	162	194	232	262	300	334
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m								
IL = 2.5 kN/m^2			82 (n/a)	101 (n/a)	126 (n/a)	149 (n/a)	175 (n/a)	206 (n/a)
IL = 5.0 kN/m^2		93 (n/a)	116 (n/a)	140 (n/a)	170 (n/a)	200 (n/a)	233 (n/a)	267 (n/a)
IL = 7.5 kN/m^2		121 (n/a)	148 (n/a)	178 (n/a)	213 (n/a)	249 (n/a)	287 (n/a)	327 (n/a)
IL = 10.0 kN/m^2		148 (n/a)	181 (n/a)	217 (n/a)	256 (n/a)	296 (n/a)	341 (n/a)	387 (n/a)
REINFORCEMENT, kg/m^2 (kg/m^3)								
IL = 2.5 kN/m^2			10 (76)	12 (74)	14 (71)	17 (75)	19 (78)	22 (76)
IL = 5.0 kN/m^2		10 (80)	11 (77)	13 (76)	16 (77)	19 (78)	21 (77)	24 (80)
IL = 7.5 kN/m^2		10 (83)	13 (83)	15 (81)	18 (81)	21 (82)	24 (83)	27 (85)
IL = 10.0 kN/m^2		11 (87)	14 (85)	16 (82)	20 (85)	24 (90)	26 (85)	29 (87)
DESIGN NOTES	*a = imposed load, q_k,> 1.25 dead load, g_k*				*b = q_k > 5 kN/m^2*		*g = T25s used*	
IL = 2.5 kN/m^2								g
IL = 5.0 kN/m^2		a						g
IL = 7.5 kN/m^2		a b	a b	a b	b	b	bg	bg
IL = 10.0 kN/m^2		a b	a b	a b	a b	a b	bg	bg

Ribbed slabs

(One-way joists)



Introducing voids to the soffit of a slab reduces dead weight and increases the efficiency of the concrete section. A slightly deeper section is required but these stiffer floors facilitate longer spans and provision of holes. Economic in the range 8 to 12 m.

The saving of materials tends to be offset by some complication in formwork. The advent of expanded polystyrene moulds has made the choice of trough profile infinite and largely superseded the use of standard T moulds. Ribs should be at least 125 mm wide to suit reinforcement detailing.

The chart and data assume line support (ie. beam or wall) and bespoke moulds.

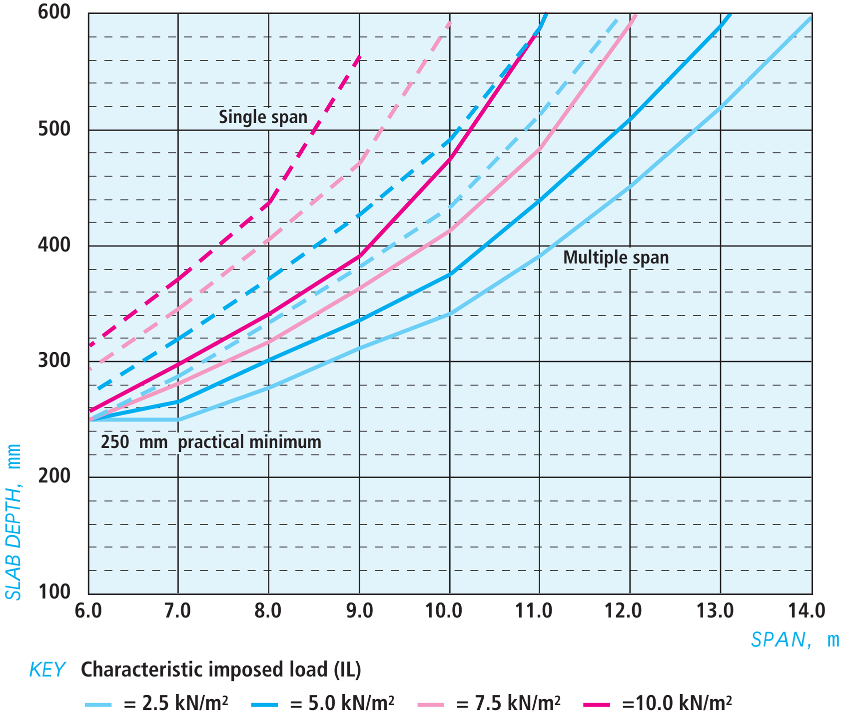
ADVANTAGES

- Medium to long spans
- Lightweight
- Holes in topping easily accommodated
- Large holes can be accommodated
- Profile may be expressed architecturally, or used for heat transfer in passive cooling

DISADVANTAGES

- Higher formwork costs than for other slab systems
- Slightly greater floor thicknesses
- Slower

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	BEAMS. Refer to beam charts and data to estimate beam sizes and reinforcement.
DIMENSIONS	Square panels, minimum of three slab spans. Ribs 150 mm wide @ 750 mm cc. Topping 100 mm. Moulds of bespoke depth. Rib/solid intersection at beam span/7 from centreline of internal support, and at span/9 from end support.
REINFORCEMENT	Maximum bar sizes in ribs: 2T25B, 2T20T (in top of web) and R8 links. 25 mm allowed for A142 mesh (@ 0.12%) in topping. 10% allowed for wastage and laps. f_s may have been reduced.
LOADS	A superimposed dead load (SDL) of 1.50 kN/m² (for finishes, services, etc.) is included. Ultimate loads assume elastic reaction factors of 1.1 to internal beams and 0.5 to end beams. Self weight used accounts for 10 degree slope to ribs and solid ends as described above.
CONCRETE	C35, 24 kN/m³, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.

SINGLE SPAN, m

THICKNESS, mm

SINGLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²	250	288	334	382	434	514	610	722	
IL = 5.0 kN/m²	272	320	372	428	492	588	772		
IL = 7.5 kN/m²	294	346	406	472	594				
IL = 10.0 kN/m²	314	372	438	564					

ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m

SINGLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²	n/a (35)	n/a (43)	n/a (52)	n/a (61)	n/a (72)	n/a (87)	n/a (105)	n/a (126)	
IL = 5.0 kN/m²	n/a (48)	n/a (58)	n/a (70)	n/a (83)	n/a (97)	n/a (116)	n/a (146)		
IL = 7.5 kN/m²	n/a (61)	n/a (74)	n/a (88)	n/a (104)	n/a (126)				
IL = 10.0 kN/m²	n/a (74)	n/a (89)	n/a (106)	n/a (129)					

REINFORCEMENT, kg/m² (kg/m³) *Slab only, add mesh and beam reinforcement*

SINGLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²	11 (42)	12 (41)	11 (34)	11 (30)	12 (27)	12 (23)	12 (20)	12 (17)	
IL = 5.0 kN/m²	11 (42)	11 (36)	11 (31)	12 (27)	12 (24)	12 (20)	12 (16)		
IL = 7.5 kN/m²	11 (39)	12 (34)	12 (29)	12 (25)	12 (20)				
IL = 10.0 kN/m²	11 (36)	12 (31)	12 (27)	12 (21)					

MULTIPLE SPAN, m

THICKNESS, mm

MULTIPLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²		250	278	312	342	392	452	520	598
IL = 5.0 kN/m²	250	266	302	336	376	440	510	590	688
IL = 7.5 kN/m²	250	282	318	364	414	484	592	732	
IL = 10.0 kN/m²	258	298	342	392	476	588	730		

ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m²

MULTIPLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²		89 (40)	105 (48)	123 (56)	142 (65)	165 (75)	193 (88)	224 (102)	261 (119)
IL = 5.0 kN/m²	101 (46)	122 (55)	144 (65)	167 (76)	192 (87)	223 (101)	257 (117)	297 (135)	346 (157)
IL = 7.5 kN/m²	129 (59)	154 (70)	181 (82)	210 (96)	242 (110)	279 (127)	328 (149)	389 (177)	
IL = 10.0 kN/m²	156 (71)	187 (85)	219 (100)	254 (115)	297 (135)	348 (158)	411 (187)		

REINFORCEMENT, kg/m² (kg/m³) *Slab only, add mesh and beam reinforcement*

MULTIPLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²		11 (45)	12 (44)	16 (51)	17 (51)	18 (46)	18 (40)	18 (35)	18 (31)
IL = 5.0 kN/m²	12 (53)	16 (59)	16 (54)	18 (53)	18 (48)	18 (41)	18 (36)	18 (31)	18 (27)
IL = 7.5 kN/m²	16 (64)	17 (60)	18 (57)	18 (50)	18 (44)	18 (38)	18 (31)	18 (25)	
IL = 10.0 kN/m²	17 (64)	17 (59)	18 (53)	18 (46)	18 (38)	18 (31)	18 (25)		

DESIGN NOTES *a = $q_k > 1.25\ g_k$* *b = $q_k > 5$ kN/m²* *c = 2T20B* *d = deflection critical* *e = designed links in ribs*

MULTIPLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²									e
IL = 5.0 kN/m²	e			e	de	de	de	e	e
IL = 7.5 kN/m²	abe	abe	abde	abde	abe	bde	be	be	
IL = 10.0 kN/m²	abe	abe	abde	abde	abe	abe	be		

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

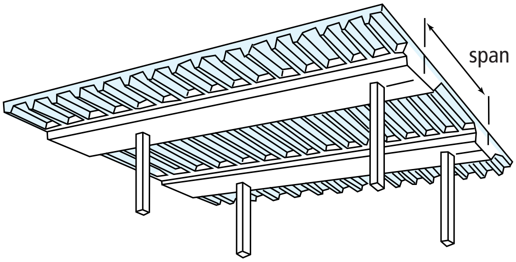
Fire resistance	2 hours, 150 rib & 115 topping	+5 mm	4 hours, 150 rib & topping	see below
Exposure	Moderate	+15 mm	Severe, C40 concrete	see below
Standard moulds	T moulds	see below	NB: T moulds 125 mm ribs @ 600 cc	

Thickness, mm

Span, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0
4 hrs,150 rib & topping	258	300	338	386	442	534	600
Severe, C40 concrete	248	288	326	366	416	494	576
T2 mould, 175 deep	265	291	305	347			
T3 mould, 250 deep			340	340	382		
T4 mould, 325 deep				415	415	450	
T5 mould, 400 deep					490	490	524

Ribbed slabs *for use with 2400 mm wide band beams only*

(One-way joists with wide beams)



As with solid slab arrangements, the band beam has a relatively wide, shallow cross section which reduces the overall depth of floor while permitting longer spans.

Used in car parks, offices, etc. where spans in one direction are predominant and live loads are relatively light. Slab spans up to 10 m (centreline support to centreline support) with beam spans up to 16 m are economic.

Charts and data assume wide beam support, minimum 100 or 180 mm downstand, and bespoke moulds. For beam thicknesses refer to pp 64, 71, 110 or 111). Thicknesses are typically governed by deflection and, to suit formwork, by restricting the downstands of beams.

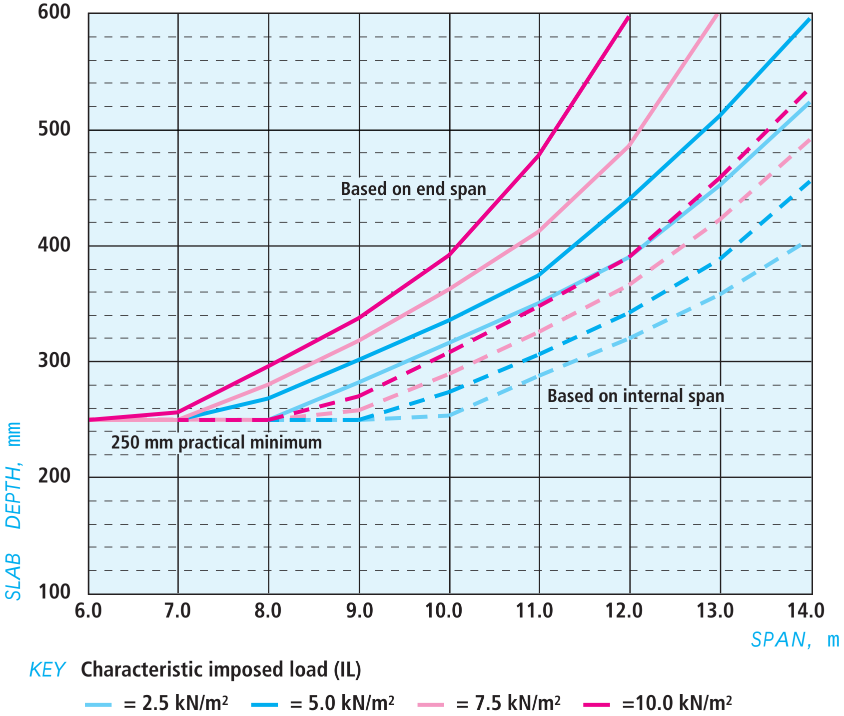
ADVANTAGES

- Medium to long spans
- Lightweight
- Holes in topping easily accommodated (but avoid beams)
- Large holes can be accommodated

DISADVANTAGES

- Higher formwork costs than for other slab systems
- Slightly greater floor heights
- Slower

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

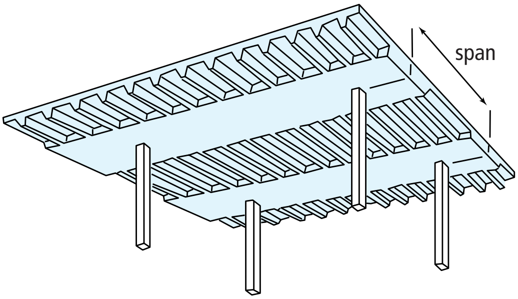
SUPPORTED BY	BEAMS. Internally, 2400 mm wide BEAMS. Refer to beam charts to estimate sizes.
DIMENSIONS	Square panels, minimum of two (for end spans) or three slab spans x three beam spans. Ribs 150 mm wide @ 750 mm cc. Topping 100 mm. Rib/solid intersection at beam span/7 from centreline of internal support, and at span/9 from end support.
SPANS	Spans quoted in charts and data are centreline support to centreline support (eg. grid to grid). However, the designs of these slabs are based on spans of end span - 1.2 m + d/2, or internal span - 2.4 m + d.
REINFORCEMENT	Maximum bar sizes in ribs: 2T25B, 2T20T (in top of web) and R8 links. 25 mm allowed for A142 mesh (@ 0.12%) in topping. 10% allowed for wastage and laps.
LOADS	SDL of 1.50 kN/m^2 (finishes) included. Ultimate loads assume elastic reaction factors of 1.1 to internal beams and 0.5 to end beams. Self weight used accounts for 10 degree slope to ribs and solid ends as described above.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.

BASED ON END SPAN, m	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
THICKNESS, mm			*Add minimum 100 mm for minimum depth of 2400 spine beam*					
IL = 2.5 kN/m^2		250	282	316	350	390	452	524
IL = 5.0 kN/m^2		268	302	336	374	440	512	596
IL = 7.5 kN/m^2	250	280	318	362	412	486	602	756
IL = 10.0 kN/m^2	256	296	338	392	478	598	754	
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m^2								
IL = 2.5 kN/m^2		101 (46)	118 (54)	139 (63)	156 (71)	180 (82)	209 (95)	242 (110)
IL = 5.0 kN/m^2		139 (63)	161 (73)	184 (84)	210 (96)	243 (110)	279 (127)	322 (146)
IL = 7.5 kN/m^2	151 (69)	176 (80)	203 (92)	233 (106)	266 (121)	305 (139)	357 (162)	425 (193)
IL = 10.0 kN/m^2	182 (83)	213 (97)	246 (112)	282 (128)	327 (148)	382 (174)	451 (205)	
REINFORCEMENT, kg/m^2 (kg/m^3)				*Slab only, add mesh and beam reinforcement*				
IL = 2.5 kN/m^2		11 (44)	12 (43)	11 (34)	16 (48)	18 (45)	18 (39)	18 (34)
IL = 5.0 kN/m^2		12 (43)	16 (54)	17 (52)	18 (48)	18 (40)	18 (35)	18 (30)
IL = 7.5 kN/m^2	11 (43)	16 (57)	17 (54)	18 (49)	18 (44)	18 (37)	18 (30)	18 (24)
IL = 10.0 kN/m^2	15 (60)	17 (56)	18 (53)	18 (45)	18 (38)	18 (30)	18 (24)	
DESIGN NOTES	*a = imposed load, q_k,> 1.25 dead load, g_k*			*b = q_k > 5 kN/m^2*		*e = designed links in ribs*		
IL = 2.5 kN/m^2								
IL = 5.0 kN/m^2		e	e	e	e	e	e	e
IL = 7.5 kN/m^2	abe	abe	abe	abe	abe	be	be	be
IL =10.0 kN/m≤	abe	abe	abe	abe	abe	abe	be	

BASED ON INTERNAL SPAN, m	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
THICKNESS, mm			*Add minimum 180 mm for minimum depth of 2400 spine beam*					
IL = 2.5 kN/m^2				254	288	320	358	406
IL = 5.0 kN/m^2			250	274	306	342	388	456
IL = 7.5 kN/m^2			258	290	326	366	422	492
IL = 10.0 kN/m^2		250	270	308	348	390	458	536
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m^2								
IL = 2.5 kN/m^2				128 (58)	147 (67)	166 (75)	188 (85)	214 (97)
IL = 5.0 kN/m^2			153 (70)	175 (80)	198 (90)	223 (101)	252 (114)	287 (131)
IL = 7.5 kN/m^2			195 (89)	222 (101)	250 (114)	281 (128)	316 (144)	358 (163)
IL = 10.0 kN/m^2		205 (93)	236 (107)	268 (122)	306 (139)	338 (154)	382 (173)	430 (195)
REINFORCEMENT, kg/m^2 (kg/m^3)				*Slab only, add mesh and beam reinforcement*				
IL = 2.5 kN/m^2				10 (41)	13 (46)	14 (44)	16 (43)	17 (41)
IL = 5.0 kN/m^2			13 (53)	14 (52)	15 (50)	16 (47)	17 (45)	17 (38)
IL = 7.5 kN/m^2			14 (54)	16 (53)	17 (51)	21 (56)	21 (50)	21 (43)
IL = 10.0 kN/m^2		14 (59)	15 (56)	16 (53)	19 (51)	21 (54)	21 (47)	22 (40)
DESIGN NOTES	*a = imposed load, q_k,> 1.25 dead load, g_k*			*b = q_k > 5 kN/m^2*		*e = designed links in ribs*		
IL = 2.5 kN/m^2								
IL = 5.0 kN/m^2			e	e	e	e	e	e
IL = 7.5 kN/m^2			abe	abe	abe	abe	abe	be
IL = 10.0 kN/m^2		abe	abe	abe	abe	abe	abe	abe

Troughed slabs

(Ribbed slabs with integral beams and level soffits, troughed flat slabs, one-way joist floors)



Troughed slabs are popular in spans up to 12 m as they combine the advantages of ribbed slabs with level soffits.

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 the longer direction.

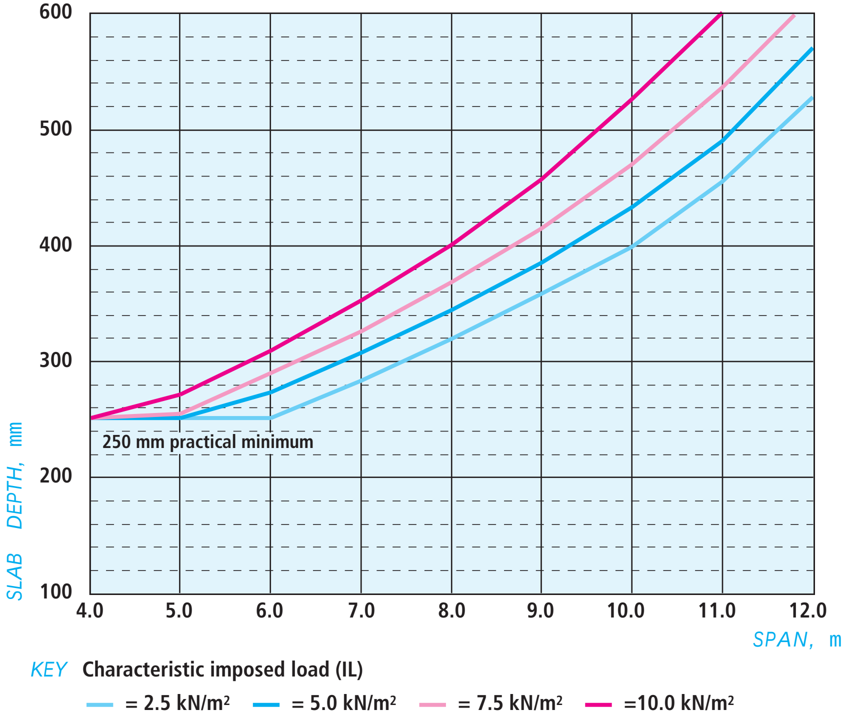
ADVANTAGES

- Longer spans than one-way solid or flat slabs
- Lightweight
- Level soffit
- Profile may be expressed architecturally, or used for heat transfer
- Holes in ribbed slab areas cause little or no problem

DISADVANTAGES

- Higher formwork costs than plain soffits

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS. Refer to column charts and data to estimate sizes, etc.
DIMENSIONS	Square panels, minimum of two rib spans x two beam spans. Ribs 150 mm wide @ 750 mm cc. Topping 100 mm. Moulds variable depth. Internal beams span/3.5 wide. Edge beams, span/9 + edge column width/2 wide. Edges flush with columns. Level soffits.
REINFORCEMENT	Max. bar sizes, ribs: 2T25B, 2T20T (in top of web) and R8 links; beams: T32 T & B, T8 links. 25 mm allowed for A142 mesh (@ 0.12%) in topping. 10% allowed for wastage, etc. To comply with deflection criteria, service stress, f_s, may have been reduced.
LOADS	SDL of 1.50 kN/m^2 (finishes) and perimeter load of 10 kN/m included. Ultimate loads to beams from slabs assume erfs of 1.2 internally and 0.46 at ends. Ultimate loads to columns assume erfs of 1.0 and 0.5. Self weight used accounts for 10 degree slope to ribs and solid ends as described above.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.

MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m^2			250	282	318	356	396	452	524
IL = 5.0 kN/m^2		250	272	306	342	382	430	486	566
IL = 7.5 kN/m^2		254	288	324	366	412	466	532	610
IL = 10.0 kN/m^2		270	308	350	398	454	522	596	720
ULTIMATE LOAD TO SUPPORTING COLUMNS, INTERNAL (EDGE) PER STOREY, MN									
IL = 2.5 kN/m^2			0.4 (0.4)	0.6 (0.5)	0.8 (0.7)	1.1 (0.8)	1.4 (1.0)	1.8 (1.3)	2.3 (1.6)
IL = 5.0 kN/m^2		0.4 (0.4)	0.6 (0.5)	0.8 (0.6)	1.1 (0.8)	1.4 (1.0)	1.8 (1.3)	2.3 (1.6)	3.0 (2.0)
IL = 7.5 kN/m^2		0.5 (0.4)	0.7 (0.6)	1.0 (0.8)	1.4 (1.0)	1.8 (1.3)	2.3 (1.6)	2.9 (2.0)	3.7 (2.4)
IL = 10.0 kN/m^2		0.6 (0.5)	0.9 (0.7)	1.2 (0.9)	1.7 (1.2)	2.1 (1.5)	2.8 (1.9)	3.5 (2.3)	4.5 (2.9)
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2			29 (114)	33 (119)	39 (127)	40 (114)	41 (106)	41 (92)	46 (88)
IL = 5.0 kN/m^2		30 (127)	32 (118)	36 (120)	38 (112)	45 (122)	50 (122)	48 (99)	49 (86)
IL = 7.5 kN/m^2		32 (125)	34 (118)	37 (114)	41 (111)	46 (112)	46 (100)	49 (91)	50 (82)
IL = 10.0 kN/m^2		37 (138)	35 (113)	41 (118)	44 (110)	46 (105)	47 (90)	50 (86)	49 (68)
DESIGN NOTES a = $q_k > 1.25\ g_k$ b = $q_k > 5$ kN/m^2 e = designed links in ribs. NB check punching shear at all columns									
IL = 2.5 kN/m^2									
IL = 5.0 kN/m^2						e	e	e	e
IL = 7.5 kN/m^2		ab	abe	abe	abe	abe	abe	be	be
IL = 10.0 kN/m^2		abe	abe	abe	abe	abe	abe	abe	abe
LINKS, %AGE BY WEIGHT OF REINFORCEMENT									*Links in ribs and beams*
IL = 2.5 kN/m^2		36%	29%	24%	18%	14%	13%	11%	11%
IL = 5.0 kN/m^2		34%	25%	20%	15%	13%	11%	9%	9%
IL = 7.5 kN/m^2		28%	20%	17%	13%	11%	10%	9%	9%
IL = 10.0 kN/m^2		25%	19%	15%	12%	9%	10%	9%	10%

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Fire resistance	2 hours, 150 rib & 115 topping	+5 mm	4 hours, 150 rib & topping	see below
Exposure	Moderate	+20 mm	Severe, C40 concrete	see below
Cladding load	No cladding load	-0 mm	20 kN/m cladding load	+25 mm
Dimensions	125 mm ribs @ 600	+0 mm	Beam widths:	
	125 mm ribs @ 750	+0 mm	Internal L/5, edge L/12 + col/2	see below
	150 mm ribs @ 900	+0 mm	Internal L/4, edge L/10 + col/2	+10 mm
	200 mm ribs @ 1200	+0 mm	**Internal L/3.5, edge L/9 + col/2**	**as original**
	250 mm ribs @ 1500	+0 mm	Internal L/3, edge L/8 + col/2	-10 mm
Other	25 mm cover	+10 mm	Rectangular beams (cf 'T' & 'L')	+0 mm
Single spans	Single slab span	see below	Single spine beam span	see below

Thickness, mm

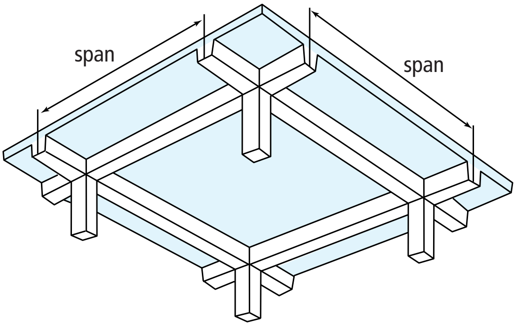
Span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
4 hrs,150 rib & topping	290	354	460	602	804		
Severe, C40 concrete	290	320	350	412	524	672	888
Beams L/5 & L/12 wide	296	332	368	410	496	544	624
1-span slab	282	320	364	420	482	578	748
1-span spine beam	304	354	410	470	532	632	748

Rectangular panels: equivalent spans, m

Use an equivalent square span, below, to derive thickness

Ribbed slab span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
Beam span = 5.0 m	5.4	6.2	6.5	7.7	9.0		
Beam span = 6.0 m	6.0	6.3	6.8	7.8	9.0	10.6	11.4
Beam span = 7.0 m	6.6	7.0	7.3	7.9	9.1	10.6	11.5
Beam span = 8.0 m	7.1	7.6	8.0	8.4	9.2	10.6	11.5
Beam span = 9.0 m	8.0	8.3	8.6	9.0	9.4	10.6	11.5
Beam span = 10.0 m	9.0	9.3	9.6	9.8	10.0	10.5	11.5
Beam span = 11.0 m	10.2	10.5	10.5	10.7	10.9	11.0	11.6
Beam span = 12.0 m	10.9	11.1	11.3	11.5	11.6	11.9	12.0

Two-way solid slabs



Two-way in-situ solid slabs are utilitarian and generally used for retail developments, warehouses, stores, etc. Economical for more heavily loaded spans from 9 to 12 m, but difficult to form when used with a grid of downstand beams.

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

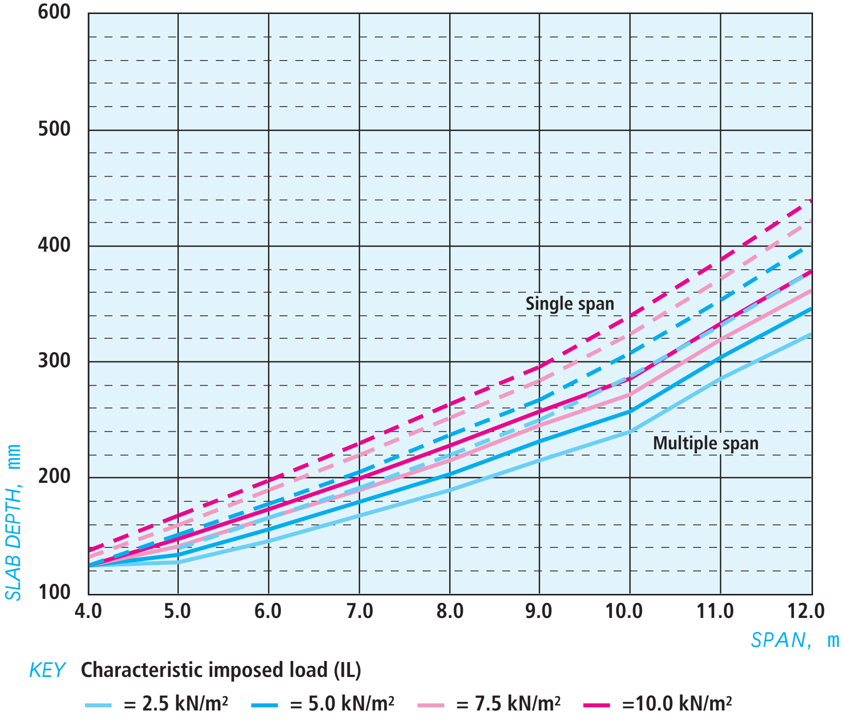
ADVANTAGES

- Economical for longer spans and high loads

DISADVANTAGES

- Presence of beams may require greater storey height
- Requires a regular column layout
- Grid of downstand beams deters fast formwork recycling
- Flexibility of partition location and horizontal service distribution may be compromised.

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	BEAMS in two orthogonal directions. Refer to beam charts and data to estimate sizes, etc.
DIMENSIONS	Square panels, minimum of two spans x two bays. Supports minimum 300 mm wide.
REINFORCEMENT	<8.5 m:T16T&B, >8.5 m: T20T&B uno. 10% allowed for wastage and laps. f_s may have been reduced.
LOADS	SDL of 1.50 kN/m^2 (finishes etc) included. Ultimate loads to internal beams assume two adjacent corner panels. Loads are applicable as a udl over 75% of the beam's length.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
DESIGN	Design based on corner panels. Single span (both ways) assumes torsional restraint.

SINGLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m^2		140	166	192	220	250	288	332	378
IL = 5.0 kN/m^2	125	152	178	206	238	268	308	354	402
IL = 7.5 kN/m^2	132	160	190	220	252	284	324	372	422
IL = 10.0 kN/m^2	138	168	198	230	264	296	340	388	440
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m					*Includes 1.5 kN/m^2 SDL. See note above*				
IL = 2.5 kN/m^2		n/a (18)	n/a (23)	n/a (29)	n/a (36)	n/a (43)	n/a (52)	n/a (63)	n/a (74)
IL = 5.0 kN/m^2	n/a (19)	n/a (25)	n/a (32)	n/a (39)	n/a (48)	n/a (57)	n/a (67)	n/a (80)	n/a (93)
IL = 7.5 kN/m^2	n/a (24)	n/a (32)	n/a (41)	n/a (50)	n/a (60)	n/a (70)	n/a (82)	n/a (97)	n/a (112)
IL = 10.0 kN/m^2	n/a (30)	n/a (39)	n/a (49)	n/a (60)	n/a (71)	n/a (83)	n/a (97)	n/a (113)	n/a (130)
REINFORCEMENT, kg/m^2 (kg/m^3)				*Including wastage but excluding beam reinforcement*					
IL = 2.5 kN/m^2	9 (75)	11 (77)	13 (77)	15 (79)	18 (84)	21 (82)	24 (84)	27 (82)	31 (82)
IL = 5.0 kN/m^2	11 (88)	12 (82)	15 (87)	18 (88)	21 (88)	24 (89)	27 (89)	31 (86)	34 (86)
IL = 7.5 kN/m^2	12 (92)	15 (91)	17 (90)	20 (90)	23 (93)	26 (93)	31 (95)	34 (91)	38 (89)
IL = 10.0 kN/m^2	14 (98)	16 (96)	19 (97)	22 (96)	26 (98)	29 (99)	33 (97)	37 (95)	41 (92)

MULTIPLE/TWO SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m^2	125	128	146	168	190	216	240	286	324
IL = 5.0 kN/m^2	125	134	156	180	204	232	258	304	346
IL = 7.5 kN/m^2	125	142	166	190	216	246	272	320	362
IL = 10.0 kN/m^2	125	148	174	200	228	258	286	334	378
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	36 (12)	42 (14)	52 (17)	66 (21)	80 (26)	96 (31)	114 (37)	138 (45)	164 (53)
IL = 5.0 kN/m^2	47 (15)	58 (19)	74 (24)	90 (29)	108 (35)	128 (42)	150 (49)	178 (58)	208 (68)
IL = 7.5 kN/m^2	60 (209)	76 (25)	94 (31)	114 (37)	136 (44)	162 (52)	186 (60)	218 (71)	252 (82)
IL = 10.0 kN/m^2	73 (23)	92 (30)	114 (37)	138 (45)	164 (54)	192 (63)	222 (72)	258 (84)	296 (96)
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	6 (51)	8 (60)	10 (71)	12 (73)	14 (76)	17 (78)	19 (79)	22 (76)	25 (77)
IL = 5.0 kN/m^2	8 (74)	10 (78)	13 (83)	15 (83)	17 (85)	20 (85)	22 (86)	25 (84)	28 (81)
IL = 7.5 kN/m^2	10 (83)	12 (84)	14 (87)	17 (91)	20 (91)	22 (90)	25 (92)	28 (89)	31 (87)
IL = 10.0 kN/m^2	12 (96)	14 (93)	16 (93)	19 (95)	21 (94)	24 (95)	27 (96)	31 (93)	34 (90)
DESIGN NOTES			*a = q_k > 1.25 g_k*		*b = q_k > 5 kN/m^2*		*d = deflection critical*		*g = T25s used B*
IL = 2.5 kN/m^2	d	d	d						
IL = 5.0 kN/m^2									
IL = 7.5 kN/m^2	ab	ab	ab	b	b	b	b	bg	bg
IL = 10.0 kN/m^2	ab	ab	ab	ab	ab	ab	b	bg	bg

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

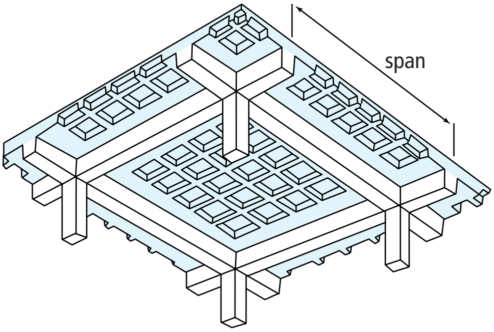
Fire resistance	2 hours	+10 mm	4 hours	+30 mm
Exposure	Moderate	+15 mm	Severe C40 concrete	+25 mm

Thickness, mm	**Span, m**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
	Internal panel	146	166	184	206	230	266	318

Rectangular panels: equivalent spans, m Use an equivalent square span, below, to derive thickness. See Section 2.6

Long span, m	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
Short span = 5.0 m	5.7	5.9	6.0				
Short span = 6.0 m	6.7	6.8	7.0	7.1	7.2		
Short span = 7.0 m	7.4	7.7	7.9	8.1	8.1	8.2	8.3
Short span = 8.0 m	8.0	8.4	8.7	8.9	9.0	9.2	9.3
Short span = 9.0 m		9.0	9.4	9.7	9.9	10.1	10.2
Short span = 10.0 m			10.0	10.2	10.4	10.6	10.7

Waffle slabs *designed as two-way slabs (standard moulds)*



Introducing voids to the soffit reduces dead weight and these deeper, stiffer floors permit longer spans which are economic for spans between 9 and 14 m. The saving of materials tends to be offset by complication in site operations.

Standard moulds are 225, 325 and 425 mm deep and are used to make ribs 125 mm wide on a 900 mm grid. Toppings are between 50 and 150 mm thick.

The chart and data assume surrounding and supporting downstand beams, which should be subject to separate consideration, and solid margins. Both waffles and downstand beams complicate formwork.

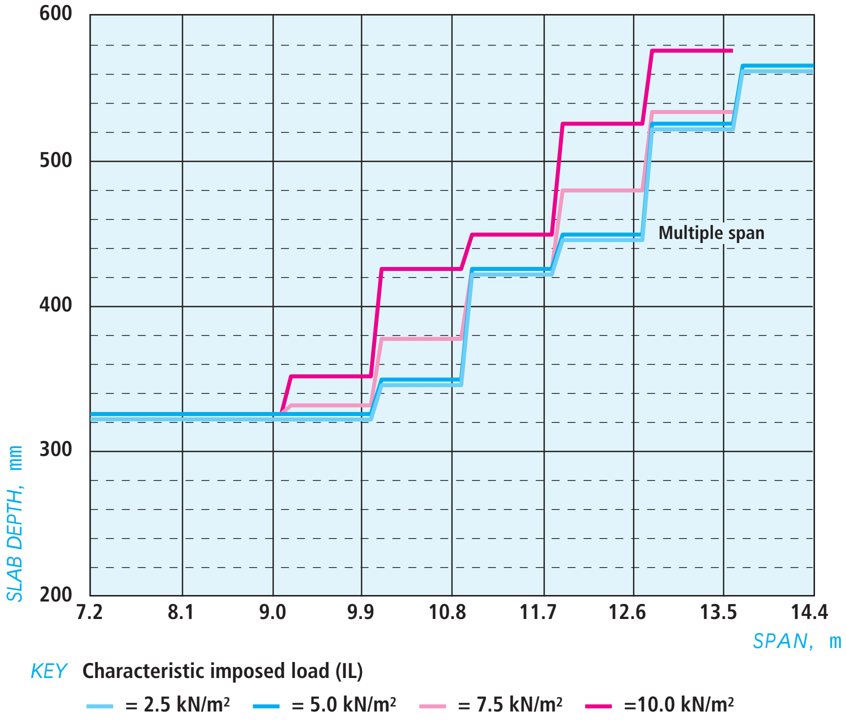
ADVANTAGES

- Medium to long spans
- Lightweight
- Profiles may be expressed architecturally, or used for heat transfer

DISADVANTAGES

- Higher formwork costs than for other slab systems
- Slightly deeper members result in greater floor heights
- Slow. Difficult to prefabricate reinforcement

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	BEAMS in two orthogonal directions. Refer to beam charts and data to estimate sizes, etc.
DIMENSIONS	Square panels, minimum of two spans x two bays. Ribs 125 mm wide @ 900 mm cc. Moulds 225, 325 or 425 mm deep. Topping 100 to 150 mm. Rib/solid intersection at 900 + 125/2 from centreline of support.
REINFORCEMENT	Maximum bar sizes in ribs: 2T25B, 2T20T (in top of web) and R8 links. 25 mm allowed for A142 or A193 mesh (@ 0.12%) in topping. 10% allowed for wastage and laps. f_s may have been reduced.
LOADS	SDL of 1.50 kN/m^2 (finishes etc) included. Ultimate loads to internal beams assume two adjacent corner panels. Loads are applicable as a udl over 75% of the beam's length. Self weight used accounts for 5:1 slope to ribs, solid edges as described above and topping as inferred.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
DESIGN	Design based on corner panels. Single span (both ways) assumes torsional restraint.

SINGLE SPAN, m	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**	**13.5**	**14.4**
THICKNESS, mm									
IL = 2.5 kN/m^2	325	325	350	375	435	525	565		
IL = 5.0 kN/m^2	325	325	365	425	470	535			
IL = 7.5 kN/m^2	325	350	425	440	525				
IL = 10.0 kN/m^2	325	375	425	470	540				
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	n/a (29)	n/a (32)	n/a (38)	n/a (45)	n/a (49)	n/a (59)	n/a (69)		
IL = 5.0 kN/m^2	n/a (38)	n/a (43)	n/a (52)	n/a (58)	n/a (68)	n/a (76)			
IL = 7.5 kN/m^2	n/a (48)	n/a (56)	n/a (64)	n/a (72)	n/a (83)				
IL = 10.0 kN/m^2	n/a (57)	n/a (69)	n/a (76)	n/a (89)	n/a (99)				
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	8 (24)	12 (35)	15 (44)	19 (51)	18 (42)	16 (31)	21 (38)		
IL = 5.0 kN/m^2	11 (33)	18 (56)	20 (53)	17 (40)	21 (45)	22 (40)			
IL = 7.5 kN/m^2	15 (45)	19 (55)	16 (37)	21 (48)	20 (37)				
IL = 10.0 kN/m^2	19 (57)	20 (53)	20 (46)	22 (47)	23 (42)				

MULTIPLE/TWO SPAN, m	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**	**13.5**	**14.4**
THICKNESS, mm									
IL = 2.5 kN/m^2	325	325	325	325	350	425	450	525	565
IL = 5.0 kN/m^2	325	325	325	325	350	425	450	525	565
IL = 7.5 kN/m^2	325	325	325	335	375	425	475	535	
IL = 10.0 kN/m^2	325	325	325	350	425	450	525	575	
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	66 (23)	75 (26)	83 (29)	91 (32)	106 (37)	122 (43)	139 (49)	158 (56)	184 (65)
IL = 5.0 kN/m^2	89 (31)	100 (35)	111 (39)	122 (43)	139 (49)	158 (55)	177 (62)	200 (70)	228 (80)
IL = 7.5 kN/m^2	111 (39)	124 (44)	138 (49)	154 (54)	180 (63)	193 (68)	226 (79)	244 (86)	
IL = 10.0 kN/m^2	133 (47)	149 (52)	166 (58)	189 (66)	212 (74)	237 (83)	264 (93)	300 (105)	
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	5 (16)	7 (20)	8 (25)	10 (32)	13 (37)	12 (27)	15 (32)	14 (27)	17 (30)
IL = 5.0 kN/m^2	7 (21)	9 (26)	11 (34)	15 (46)	19 (55)	16 (37)	20 (44)	19 (36)	22 (39)
IL = 7.5 kN/m^2	8 (26)	11 (33)	14 (44)	19 (58)	21 (55)	20 (48)	22 (47)	23 (43)	
IL = 10.0 kN/m^2	10 (31)	13 (40)	18 (55)	21 (59)	18 (43)	22 (50)	21 (41)	24 (42)	
DESIGN NOTES									
IL = 2.5 kN/m^2					e	e	e	e	e
IL = 5.0 kN/m^2			e	e	e	e	e	e	e
IL = 7.5 kN/m^2	ab	abe	abe	abe	be	be	be	be	
IL = 10.0 kN/m^2	abe	abe	abe	abe	abe	abe	be	be	

a = $q_k > 1.25\ g_k$ b = $q_k > 5$ kN/m^2 e = designed links may be required in ribs

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

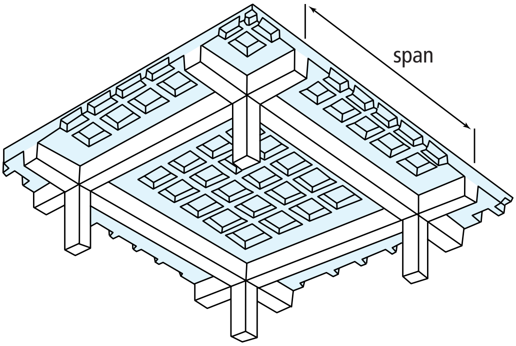
Thickness, mm

Span, m	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**
2 hrs fire, 115 topping	340	340	340	340	440	440	540
4 hrs 150 rib & topping	375	375	475	475	475	575	575
Moderate exposure	325	325	339	425	435	525	
Severe exposure (C40)	325	325	345	425	440	535	

Rectangular panels: economic thickness, mm

Long span, m	**12.6**	**13.5**	**14.4**	**15.3**	**16.2**	**17.1**	**18.0**
Short span = 9.0 m	325	325	325	325	325	325	325
Short span = 9.9 m	325	325	335	345	350	355	360
Short span = 10.8 m	355	365	375	425	425	425	425
Short span = 11.7 m	425	425	425	425	435	450	460
Short span = 12.6 m	450	450	455	475	525	525	525
Short span = 13.5 m		525	525	525	535	550	575

Waffle slabs *designed as two-way slabs (bespoke moulds)*



Bespoke moulds make the choice of profile infinite, but their cost will generally be charged to the particular project. Polypropylene, GRP or expanded polystyrene moulds can be manufactured to suit particular requirements and obtain overall economy in spans up to 16 m.

Minimum width of rib usually 125 mm, although 150 mm may be more practical to suit reinforcement detailing on longer spans. Minimum topping thickness is usually 90 mm to suit fire requirements.

The chart and data assume a 900 mm grid and solid margins adjacent to beams. Supporting downstand beams complicate formwork.

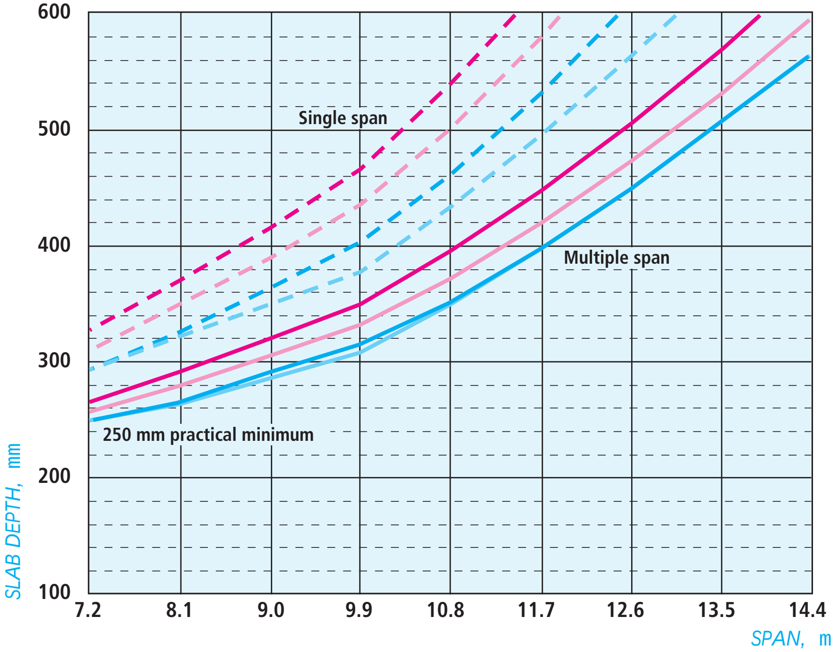
ADVANTAGES

- Medium to long spans
- Lightweight
- Profile may be expressed architecturally, or used for heat transfer

DISADVANTAGES

- Higher formwork costs than for standard moulds and other slab systems
- Slightly deeper members result in greater floor heights
- Slow. Difficult to prefabricate reinforcement

SPAN:DEPTH CHART



KEY **Characteristic imposed load (IL)**

— = 2.5 kN/m² — = 5.0 kN/m² — = 7.5 kN/m² — =10.0 kN/m²

DESIGN ASSUMPTIONS

SUPPORTED BY	BEAMS in two orthogonal directions. Refer to beam charts and data to estimate sizes, etc.
DIMENSIONS	Square panels, minimum of two spans x two bays. Ribs 125 mm wide @900 mm cc. Moulds variable depths. Rib/solid intersection @ 900 + 125/2 from centreline of support. Topping 100 mm.
REINFORCEMENT	Maximum bar sizes in ribs: 2T25B, 2T20T (in top of web) and R8 links. 25 mm allowed for A142 mesh (@ 0.12%) in topping. 10% allowed for wastage and laps.
LOADS	SDL of 1.50 kN/m^2 (finishes etc) included. Ultimate loads to internal beams assume two adjacent corner panels. Loads are applicable as a udl over 75% of the beam's length. Self weight used accounts for 5:1 slope to ribs and solid edges as described above.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
DESIGN	Design based on corner panels. Single span (both ways) assumes torsional restraint.

SINGLE SPAN, m	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**	**13.5**	**14.4**
THICKNESS, mm									
IL = 2.5 kN/m^2	294	322	350	378	434	496	564	636	734
IL = 5.0 kN/m^2	294	326	364	404	462	532	612	708	
IL = 7.5 kN/m^2	310	350	390	436	502	580	670		
IL = 10.0 kN/m^2	328	370	416	466	540	624			
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	n/a (28)	n/a (32)	n/a (37)	n/a (42)	n/a (49)	n/a (57)	n/a (67)	n/a (78)	n/a (93)
IL = 5.0 kN/m^2	n/a (37)	n/a (43)	n/a (49)	n/a (56)	n/a (65)	n/a (75)	n/a (87)	n/a (103)	
IL = 7.5 kN/m^2	n/a (47)	n/a (55)	n/a (63)	n/a (71)	n/a (82)	n/a (94)	n/a (109)		
IL = 10.0 kN/m^2	n/a (57)	n/a (66)	n/a (76)	n/a (86)	n/a (99)	n/a (113)			
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	10 (34)	12 (37)	14 (41)	17 (45)	18 (41)	19 (38)	20 (36)	22 (35)	23 (31)
IL = 5.0 kN/m^2	16 (55)	18 (55)	19 (52)	20 (49)	21 (45)	21 (40)	22 (36)	23 (32)	
IL = 7.5 kN/m^2	18 (58)	19 (54)	20 (52)	21 (48)	21 (43)	22 (38)	23 (34)		
IL = 10.0 kN/m^2	19 (56)	20 (54)	21 (50)	22 (47)	22 (42)	23 (37)			

MULTIPLE/TWO SPAN, m	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**	**13.5**	**14.4**
THICKNESS, mm									
IL = 2.5 kN/m^2	250	264	286	308	350	398	450	508	566
IL = 5.0 kN/m^2	250	266	292	316	352	398	450	508	566
IL = 7.5 kN/m^2	258	280	306	332	372	420	474	532	598
IL = 10.0 kN/m^2	266	292	320	350	396	448	506	570	640
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	61 (21)	70 (24)	79 (28)	90 (31)	103 (36)	118 (41)	135 (47)	155 (55)	177 (62)
IL = 5.0 kN/m^2	83 (29)	95 (33)	108 (38)	121 (42)	136 (48)	154 (54)	174 (61)	197 (69)	222 (78)
IL = 7.5 kN/m^2	106 (37)	121 (42)	136 (48)	153 (54)	172 (60)	193 (68)	216 (76)	242 (85)	273 (96)
IL = 10.0 kN/m^2	128 (45)	146 (51)	165 (58)	185 (65)	208 (73)	233 (82)	260 (91)	291 (102)	326 (115)
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	7 (28)	9 (33)	10 (36)	12 (38)	12 (35)	13 (33)	14 (31)	15 (29)	16 (29)
IL = 5.0 kN/m^2	10 (40)	13 (49)	14 (49)	16 (51)	18 (51)	19 (47)	19 (43)	20 (39)	21 (37)
IL = 7.5 kN/m^2	12 (48)	15 (54)	17 (55)	19 (57)	20 (55)	21 (50)	22 (46)	23 (43)	23 (39)
IL = 10.0 kN/m^2	15 (55)	17 (58)	19 (59)	20 (58)	21 (54)	22 (49)	23 (45)	24 (42)	25 (38)
DESIGN NOTES a = $q_k > 1.25\, g_k$ b = $q_k > 5$ kN/m^2 d = deflection critical e = designed links may be required in ribs									
IL = 2.5 kN/m^2					ed	ed	ed	ed	ed
IL = 5.0 kN/m^2		e	e	e	ed	ed	ed	ed	ed
IL = 7.5 kN/m^2	abe	abe	abe	abe	be	be	be	be	be
IL = 10.0 kN/m^2	abe	abe	abe	abe	abe	abe	abe	be	be

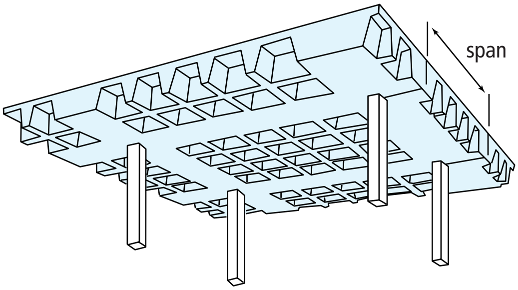
VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Thickness, mm	**Span, m**	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**
	2 hrs fire, 115 topping	270	296	322	350	396	444	496
	4 hrs 150 rib & topping	314	344	388	412	450	502	566
	Moderate exposure	270	302	338	376	430	520	660
	Severe exposure (C40)	276	308	342	382	436	528	670

Rectangular panels: equivalent spans, m Use an equivalent square span, below, to derive thickness. See Section 2.6

Long span, m	**12.6**	**13.5**	**14.4**	**15.3**	**16.2**	**17.1**	**18.0**
Short span = 9.0 m	9.3	9.4	9.5	9.6	9.7	9.8	9.9
Short span = 9.9 m	10.2	10.3	10.5	10.6	10.7	10.8	10.9
Short span = 10.8 m	10.9	11.1	11.3	11.5	11.7	11.8	11.9
Short span = 11.7 m	11.7	11.8	12.0	12.2	12.4	12.6	12.7
Short span = 12.6 m	12.6	12.7	12.8	13.0	13.2	13.4	13.6
Short span = 13.5 m		13.5	13.6	13.7	13.9	14.1	14.3

Waffle slabs *designed as two-way slabs with integral beams and level soffits (standard moulds)*



These slabs are popular in spans up to 10 m. They combine the advantages of waffle slabs with those of level soffits.

Standard moulds are 225, 325 and 425 mm deep and are used with toppings between 50 and 150 mm thick. The ribs are 125 mm wide on a 900 mm grid.

Depth is governed by deflection of the beams, which, therefore, tend to be heavily reinforced. The chart and data assume internal beams at least 1925 mm wide (ie. two waffles wide) and perimeter beams at least 962 mm (ie. one waffle) plus column width/2, wide. They include an allowance for an edge loading of 10 kN/m.

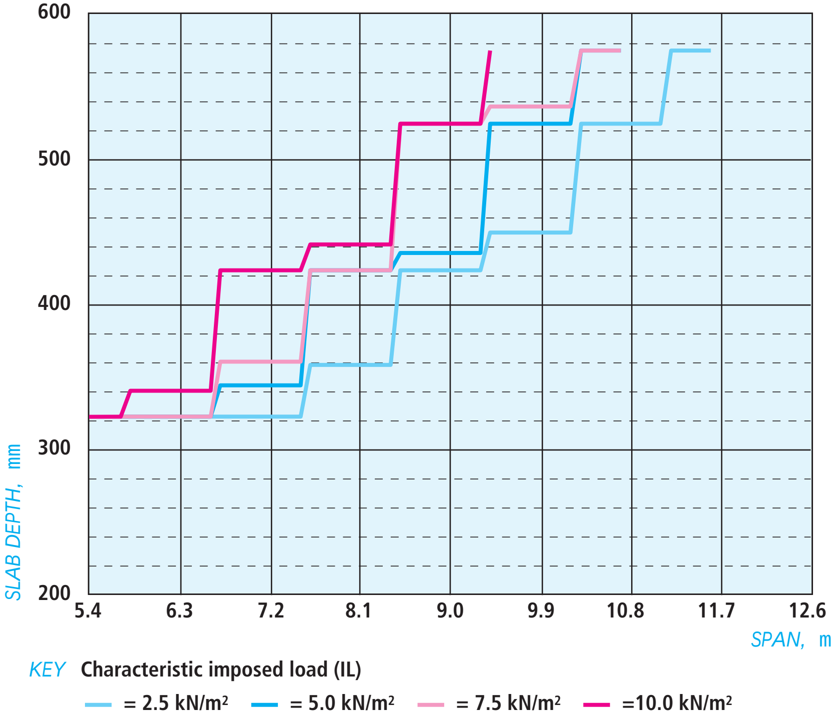
ADVANTAGES

- Medium spans
- Lightweight
- Level soffit
- Profile may be expressed architecturally, or used for heat transfer

DISADVANTAGES

- Higher formwork costs than for plain soffits
- Slow. Difficult to prefabricate reinforcement

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS. Refer to column charts and data to estimate sizes, etc.
DIMENSIONS	Square panels, minimum of two spans x two bays. Ribs 125 mm wide @ 900 mm cc. Moulds 225, 325 or 425 mm deep. Topping 100 to 150 mm deep. Internal beam two waffles wide, edge beam one waffle wide, ie. rib/solid intersection at 900 +125/2 from centreline of support.
REINFORCEMENT	Maximum bar sizes, ribs: 2T25B, 2T20T (in top of web) and R8 links; beams: T32T, T32B and T8 links. 25 mm allowed for A142 or A193 mesh (@ 0.12%) in topping. 10% allowed for wastage and laps. f_s may have been reduced.
LOADS	SDL of 1.50 kN/m^2 (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads to columns assume elastic reaction factors of 1.0 internally and 0.5 at ends. Self weight used accounts for 5:1 slope to ribs, solid beam areas as described above and topping as inferred.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
DESIGN	Slab design based on corner panels.

MULTIPLE SPAN, m	**5.4**	**6.3**	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**
THICKNESS, mm									
IL = 2.5 kN/m^2	325	325	325	361	425	451	525		
IL = 5.0 kN/m^2	325	325	347	425	437	525			
IL = 7.5 kN/m^2	325	325	363	425	525	537			
IL = 10.0 kN/m^2	325	343	425	443	525				
ULTIMATE LOAD TO SUPPORTING COLUMNS, INTERNAL (EDGE) PER STOREY, MN									
IL = 2.5 kN/m^2	0.5 (0.3)	0.6 (0.4)	0.8 (0.5)	1.0 (0.7)	1.3 (0.8)	1.7 (1.0)	2.2 (1.3)		
IL = 5.0 kN/m^2	0.6 (0.4)	0.8 (0.5)	1.0 (0.6)	1.4 (0.8)	1.7 (1.0)	2.2 (1.3)			
IL = 7.5 kN/m^2	0.7 (0.4)	0.9 (0.6)	1.2 (0.7)	1.6 (1.0)	2.2 (1.3)	2.7 (1.5)			
IL = 10.0 kN/m^2	0.8 (0.5)	1.1 (0.7)	1.5 (0.9)	1.9 (1.1)	2.5 (1.4)				
REINFORCEMENT, kg/m^2 (kg/m^3)							*Including beam reinforcement*		
IL = 2.5 kN/m^2	23 (70)	24 (75)	28 (85)	29 (81)	28 (67)	33 (72)	33 (63)		
IL = 5.0 kN/m^2	25 (78)	28 (86)	32 (91)	29 (69)	34 (78)	34 (65)			
IL = 7.5 kN/m^2	28 (86)	32 (99)	34 (95)	34 (80)	34 (64)	39 (74)			
IL = 10.0 kN/m^2	30 (94)	35 (101)	32 (76)	38 (86)	38 (73)				
DESIGN NOTES			*a = $q_k > 1.25 g_k$*	*b = $q_k > 5$ kN/m^2*		*e = designed links may be required in ribs*			
IL = 2.5 kN/m^2									
IL = 5.0 kN/m^2						e			
IL = 7.5 kN/m^2	ab	ab	be	be	be	be			
IL = 10.0 kN/m^2	ab	ab	abe	abe	be				
LINKS (%age by weight of reinforcement)								*Links in ribs and beams*	
IL = 2.5 kN/m^2	(58%)	(46%)	(36%)	(28%)	(22%)	(19%)	(15%)		
IL = 5.0 kN/m^2	(52%)	(40%)	(32%)	(24%)	(19%)	(16%)			
IL = 7.5 kN/m^2	(47%)	(35%)	(26%)	(21%)	(17%)	(14%)			
IL = 10.0 kN/m^2	(43%)	(32%)	(25%)	(19%)	(16%)				

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

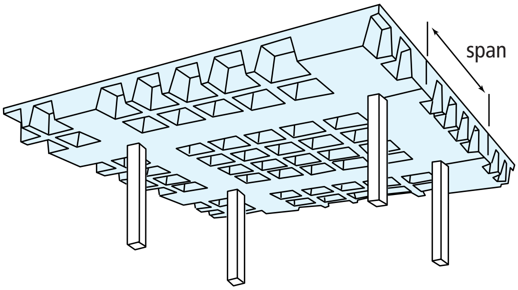
Fire resistance	2 hours, 115 topping	+20 mm	4 hours, 150 rib & topping	see below
Exposure	Moderate exposure	+0 to 25 mm	Severe, C40 concrete	+0 to 25 mm
Cladding load	No cladding load	-0 mm	20 kN/m cladding load	+0 to 12 mm
Dimensions	125 mm rib @ 800 cc	see below	150 mm rib @ 925 cc	+0 to 25 mm
	175 mm rib @ 950 cc	+0 to 25 mm	225 mm rib @ 1000 cc	see below
Single spans	One way	+0 to 12 mm	Both ways	+0 to 12 mm

Thickness, mm							
Span, m	**5.5**	**6.5**	**7.4**	**8.3**	**9.3**	**10.2**	**11.1**
4 hrs, 150 rib & topping	375	375	475	475	575		
Span, m	**7.2**	**8.0**	**8.8**	**9.6**	**10.4**	**11.2**	**12.0**
125 ribs @ 800 cc	325	357	425	429	525	525	
Span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
225 ribs @ 1000 cc	325	325	367	425	525	571	

Rectangular panels: economic thickness, mm

Long span, m	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**
Short span = 5.4 m	325	325	359	525	525		
Short span = 6.3 m	325	333	425	425	525		
Short span = 7.2 m	347	347	425	431	475	550	
Short span = 8.1 m		425	425	441	525	563	
Short span = 9.0 m			437	445	525	575	
Short span = 9.9 m				525	525		

Waffle slabs *designed as two-way slabs with integral beams and level soffits (bespoke moulds)*



These slabs are popular in spans up to 10 m as they combine the advantages of bespoke waffle slabs with level soffits. Bespoke moulds can overcome the dimensional and aesthetic restrictions imposed by standard moulds. However, site operations remain complicated.

Economic depths are a function of the beam width. The beams are governed by deflection and, therefore, tend to be heavily reinforced. The ribs are a minimum of 125 mm wide.

For simplicity, the chart and data assume a 900 mm grid, internal beams at least 1925 mm wide (ie. two waffles wide) and perimeter beams at least 962 mm (ie. one waffle) plus column width/2, wide. They include an allowance for an edge loading of 10 kN/m.

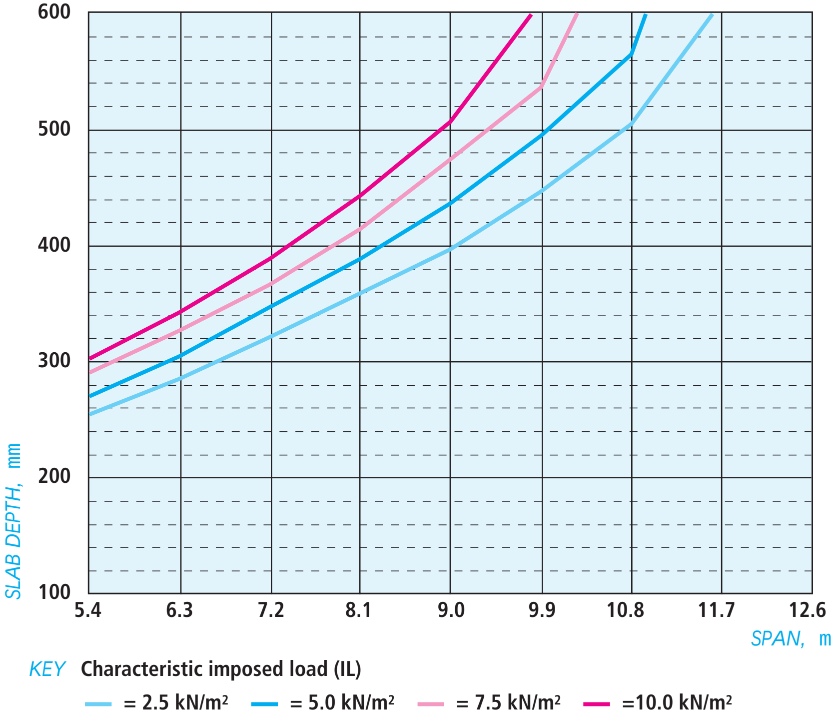
ADVANTAGES

- Medium spans
- Lightweight
- Profile may be expressed architecturally, or used for heat transfer

DISADVANTAGES

- Higher formwork costs than for standard moulds and other slab systems
- Slightly deeper members result in greater floor heights
- Slow. Difficult to prefabricate reinforcement

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS. Refer to column charts and data to estimate sizes, etc.
DIMENSIONS	Square panels, minimum of two spans x two bays. Ribs 125 mm wide @900 mm cc. Topping 100 mm. Moulds variable depth. Internal beam two waffles wide; edge beam one waffle wide, ie. rib/solid intersection at 900 + 125/2 from centreline of support.
REINFORCEMENT	Max. bar sizes, ribs: 2T25B, 2T20T (in top of web) and R8 links; beams: T32 T & B, T8 links. 25 mm allowed for A142 mesh (@ 0.12%) in topping. 10% allowed for wastage, etc.
LOADS	SDL of 1.50 kN/m^2 (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads to columns assume elastic reaction factors of 1.0 internally and 0.5 at ends. Self weight used accounts for 5:1 slope to ribs and solid beam areas as described above.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
DESIGN	Slab design based on corner panels.

MULTIPLE SPAN, m	**5.4**	**6.3**	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**
THICKNESS, mm									
IL = 2.5 kN/m^2	254	284	320	358	396	446	504	610	
IL = 5.0 kN/m^2	270	304	346	388	436	494	564	774	
IL = 7.5 kN/m^2	290	326	366	414	474	536	694		
IL = 10.0 kN/m^2	302	342	388	442	506	610			
ULTIMATE LOAD TO SUPPORTING COLUMNS, INTERNAL (EDGE) PER STOREY, MN									
IL = 2.5 kN/m^2	0.4 (0.3)	0.6 (0.4)	0.8 (0.5)	1.0 (0.6)	1.3 (0.8)	1.6 (1.0)	2.1 (1.3)	2.8 (1.6)	
IL = 5.0 kN/m^2	0.5 (0.4)	0.7 (0.5)	1.0 (0.6)	1.3 (0.8)	1.7 (1.0)	2.1 (1.3)	2.7 (1.6)	3.9 (2.2)	
IL = 7.5 kN/m^2	0.7 (0.4)	0.9 (0.6)	1.2 (0.7)	1.6 (1.0)	2.1 (1.2)	2.6 (1.5)	3.6 (2.0)		
IL = 10.0 kN/m^2	0.8 (0.5)	1.1 (0.7)	1.5 (0.9)	1.9 (1.1)	2.5 (1.4)	3.2 (1.8)			
REINFORCEMENT, kg/m^2 (kg/m^3)							*Including beam reinforcement*		
IL = 2.5 kN/m^2	31 (124)	31 (109)	30 (95)	29 (80)	31 (78)	32 (72)	34 (67)	35 (58)	
IL = 5.0 kN/m^2	33 (124)	32 (105)	31 (90)	33 (86)	34 (78)	36 (72)	38 (67)	37 (48)	
IL = 7.5 kN/m^2	32 (110)	32 (99)	34 (95)	35 (85)	37 (78)	39 (73)	39 (56)		
IL = 10.0 kN/m^2	34 (114)	34 (101)	37 (95)	38 (85)	40 (78)	41 (67)			
DESIGN NOTES			*$a = q_k > 1.25\ g_k$*	*$b = q_k > 5$ kN/m^2*		*e = designed links may be required in ribs*			
IL = 2.5 kN/m^2									
IL = 5.0 kN/m^2						e	e	e	
IL = 7.5 kN/m^2	ab	ab	b	be	be	be	be	be	
IL = 10.0 kN/m^2	ab	ab	abe	abe	abe	be			
LINKS (%age by weight of reinforcement)							*Links in ribs and beams*		
IL = 2.5 kN/m^2	(60%)	(50%)	(39%)	(28%)	(22%)	(19%)	(15%)	(14%)	
IL = 5.0 kN/m^2	(54%)	(42%)	(32%)	(25%)	(19%)	(15%)	(14%)	(15%)	
IL = 7.5 kN/m^2	(47%)	(34%)	(26%)	(21%)	(17%)	(14%)	(15%)		
IL = 10.0 kN/m^2	(44%)	(32%)	(25%)	(19%)	(15%)	(14%)			

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Fire resistance	2 hours, 115 topping	+5 mm up to 10 m	4 hours, 150 rib & topping	+15 mm up to 10 m
Exposure	Moderate	+25 mm up to 10 m	Severe, C40 concrete	+15 mm up to 10 m
Cladding load	No cladding load	-0 mm	20 kN/m cladding load	+10 mm up to 10 m
Single spans	One way	+25 mm up to 10 m	Both ways	+25 mm up to 10 m
Dimensions	Var. rib widths & cc,	see below		

Thickness, mm

Span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
125 ribs @ 750 #	308	355	408	474	585	729	
150 ribs @ 750 #	308	355	409	476	595	752	
125 ribs @ 900 #	as orig	338	376	436	502	611	
150 ribs @ 900 #	290	338	376	438	506	637	
125 ribs @ 1000	288	326	362	416	476	540	740
150 ribs @ 1000	288	326	362	418	478	544	760
150 ribs @ 1200 #		309	346	390	441	500	580
225 ribs @ 1200 #		309	346	392	446	508	596
Internal beams 3 waffles wide #			352	378	432	478	560

Data interpolated from modular spans

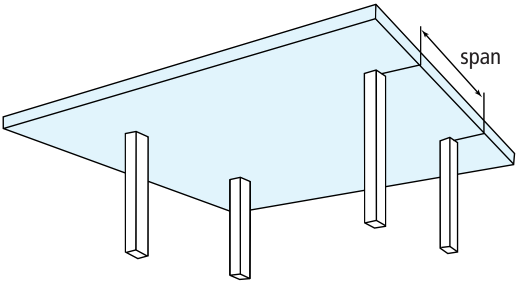
Rectangular panels

For non-square panels use an equivalent square span to derive thickness

Long span, m	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**
Short span = 5.4 m	6.3	6.9	7.5	8.3	9.1		
Short span = 6.3 m	6.3	6.9	7.8	8.7	9.3	10.4	11.0
Short span = 7.2 m	7.2	7.2	8.1	8.9	9.5	10.7	11.3
Short span = 8.1 m		8.1	8.2	9.1	9.7	10.8	11.5
Short span = 9.0 m			9.0	9.1	9.9	10.9	11.7
Short span = 9.9 m				9.9	10.1	10.9	11.8
Short span =10.8 m					10.8	11.0	11.9
Short span =11.7 m						11.7	11.9

Flat slabs

(Solid flat slabs. Flat plates in US and Australia)



Flat slabs are quick and easy to construct but punching shear, deflections and holes around columns need to be considered. Nonetheless, flat slabs are popular for office buildings, hospitals, hotels, blocks of flats, etc. as they are quick, allow easy service distribution and are very economical for square panels with a span of 5 to 9 m.

The chart and data assume a perimeter loading of 10 kN/m and one 150 mm hole adjacent to each column. They assume column sizes will at least equal those given in the data

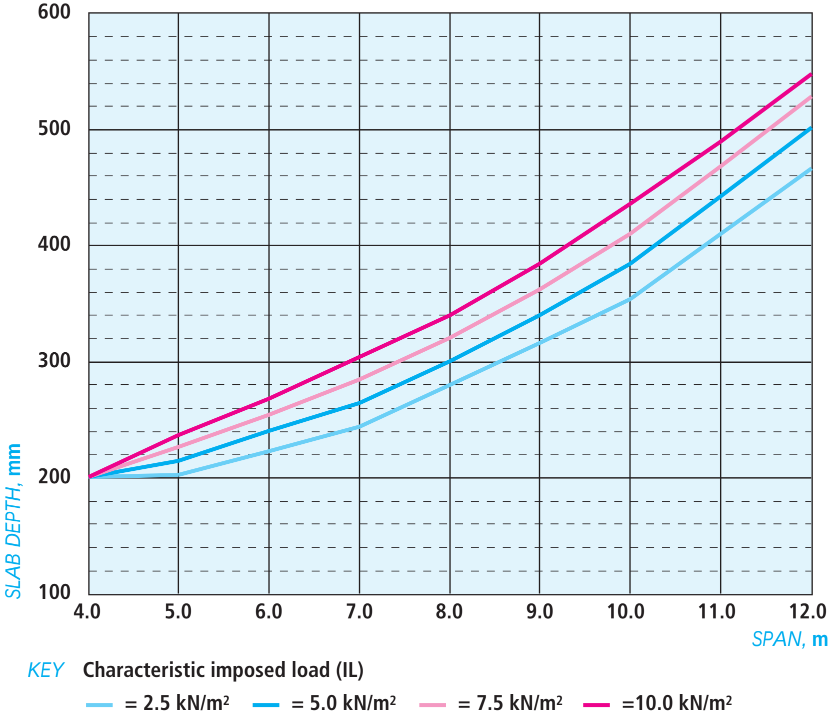
ADVANTAGES

- Simple and fast formwork and construction
- Absence of beams allows lower storey heights
- Flexibility of partition location and horizontal service distribution
- Architectural finish can be applied directly to the underside of slab

DISADVANTAGES

- Holes can prove difficult, especially large holes near columns
- Shear provision around columns may need to be resolved using larger columns, column heads, drop panels or proprietary systems
- Deflections, especially of edges supporting cladding, may cause concern

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS. Refer to column charts and data to estimate sizes, etc. Minimum dimensions of columns as data.
DIMENSIONS	Square panels, minimum of three spans x three bays. Outside edge flush with columns.
REINFORCEMENT	Main bars: T20 uno. Links R8. To help deflection, 25% A_sT at first internal support used as A_s' at midspan of end spans. Service stress, f_s, may have been reduced.10% allowed for wastage and laps.
LOADS	SDL of 1.50 kN/m^2 (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads assume elastic reaction factors of 1.0 to internal columns and 0.5 to end columns.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
HOLES	One 150 mm square hole assumed to adjoin each column. Larger holes may invalidate the data below.

MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m^2	200	202	222	244	280	316	354	410	466
IL = 5.0 kN/m^2	200	214	240	264	300	340	384	442	502
IL = 7.5 kN/m^2	200	226	254	284	320	362	410	468	528
IL = 10.0 kN/m^2	200	236	268	304	340	384	436	490	548
ULTIMATE LOAD TO SUPPORTING COLUMNS, INTERNAL (EDGE) PER STOREY, MN									
IL = 2.5 kN/m^2	0.2 (0.2)	0.3 (0.3)	0.5 (0.4)	0.7 (0.5)	1.0 (0.7)	1.4 (0.9)	1.8 (1.1)	2.4 (1.4)	3.1 (1.9)
IL = 5.0 kN/m^2	0.3 (0.2)	0.4 (0.3)	0.7 (0.4)	0.9 (0.6)	1.3 (0.8)	1.7 (1.1)	2.3 (1.4)	3.0 (1.8)	3.9 (2.3)
IL = 7.5 kN/m^2	0.3 (0.2)	0.5 (0.4)	0.8 (0.5)	1.2 (0.7)	1.6 (1.0)	2.1 (1.3)	2.8 (1.6)	3.6 (2.1)	4.6 (2.6)
IL = 10.0 kN/m^2	0.4 (0.3)	0.7 (0.4)	1.0 (0.6)	1.4 (0.9)	1.9 (1.1)	2.5 (1.5)	3.3 (1.9)	4.2 (2.4)	5.2 (3.0)
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	10 (52)	15 (75)	19 (87)	25 (104)	28 (101)	32 (101)	38 (108)	43 (104)	50 (108)
IL = 5.0 kN/m^2	13 (65)	18 (86)	22 (92)	29 (108)	33 (109)	39 (115)	44 (114)	50 (114)	54 (107)
IL = 7.5 kN/m^2	16 (80)	21 (93)	26 (103)	32 (112)	39 (123)	44 (121)	52 (127)	53 (114)	59 (111)
IL = 10.0 kN/m^2	20 (100)	24 (101)	29 (108)	34 (113)	43 (126)	52 (134)	54 (123)	58 (118)	65 (120)
COLUMN SIZES ASSUMED, mm square, internal (perimeter)									
IL = 2.5 kN/m^2	250 (225)	250 (225)	270 (250)	320 (290)	380 (340)	440 (400)	510 (460)	590 (530)	680 (610)
IL = 5.0 kN/m^2	250 (225)	250 (230)	310 (280)	370 (330)	430 (380)	500 (450)	580 (510)	660 (590)	750 (670)
IL = 7.5 kN/m^2	250 (225)	280 (250)	340 (300)	410 (360)	480 (420)	560 (490)	640 (560)	730 (640)	820 (730)
IL = 10.0 kN/m^2	250 (225)	310 (270)	380 (330)	450 (390)	530 (450)	610 (520)	690 (600)	780 (690)	870 (770)
DESIGN NOTES a = $q_k > 1.25\ g_k$ b = $q_k > 5$ kN/m^2 f = shear critical (initially $v>2v_c$) g = T25s used h = T32s used									
IL = 2.5 kN/m^2				f	f	g	g	h	h
IL = 5.0 kN/m^2				f	f	g	g	h	h
IL = 7.5 kN/m^2	a b	a b	b	b f	b f	b g	b g	b h	b h
IL = 10.0 kN/m^2	a b	a b	a b	a b f	b f	b g	b g	b h	b h
LINKS, MAXIMUM NUMBER OF PERIMETERS (and percentage by weight of reinforcement), no. (%)									
IL = 2.5 kN/m^2	6 (4.8%)	7 (4.1%)	7 (2.8%)	6 (1.9%)	7 (2.6%)	7 (2.7%)	7 (2.5%)	7 (2.7%)	6 (2.2%)
IL = 5.0 kN/m^2	7 (6.0%)	6 (3.6%)	6 (2.9%)	6 (2.7%)	6 (2.7%)	6 (2.6%)	6 (2.6%)	7 (3.1%)	7 (3.0%)
IL = 7.5 kN/m^2	7 (5.9%)	6 (3.7%)	6 (3.7%)	6 (3.1%)	6 (2.8%)	6 (3.0%)	6 (2.8%)	7 (3.6%)	7 (3.6%)
IL = 10.0 kN/m^2	6 (4.6%)	6 (3.7%)	6 (3.9%)	6 (3.9%)	6 (3.4%)	6 (3.0%)	7 (3.7%)	7 (3.9%)	7 (3.9%)

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Fire resistance	2 hours	+10 mm	4 hours	+35 mm
Exposure	Moderate	+20 mm	Severe, C40 concrete	+25 mm
Cladding load	No cladding load	-0 mm	20 kN/m cladding load	+25 mm
Other	No holes	-0 mm	Rectangular columns (1:2)	+0 mm
	Using T25s cf T20s	+10 mm	2 spans	+10 mm

Thickness, mm

Span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
Shear <1.6 v_c	256	310	376	416	486	550	520
No shear links	402	490	586	654			
225 holes adj. cols.	324	326	344	370	412	442	498
300 holes adj. cols.	452	454	456	458	468	480	510
Stiff edge (basic l/d = 40)	266	302	344	386	428	498	572

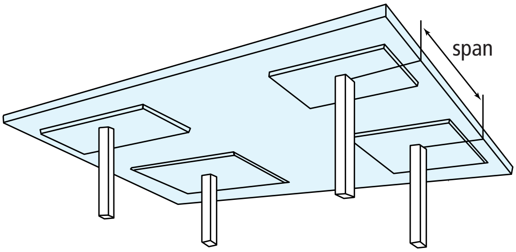
Rectangular panels: equivalent spans, m

Use an equivalent square span, below, to derive thickness

Long span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
Short span = 5.0 m	5.5	6.0	6.5	7.1	7.8		
Short span = 6.0 m	6.0	6.5	7.0	7.7	8.4	9.3	10.1
Short span = 7.0 m		7.0	7.5	8.0	8.7	9.5	10.3
Short span = 8.0 m			8.0	8.5	9.0	9.7	10.5
Short span = 9.0 m				9.0	9.5	10.0	10.7
Short span =10.0 m					10.0	10.5	11.1
Short span =11.0 m						11.0	11.6

Flat slabs with drops

(Flat slab in US and Australia)



Drop panels, formed by thickening the bottom of the slab around columns, increase shear capacity and increase the stiffness of the slab, allowing thinner slabs to be used. Popular for office buildings, hospitals, hotels, etc. Very economical for more heavily loaded spans of from 5 to 10 m. Square panels are most economical.

The chart and data assume an edge loading of 10 kN/m and one 150 mm hole adjacent to each column. They assume column sizes will at least equal those given in the data.

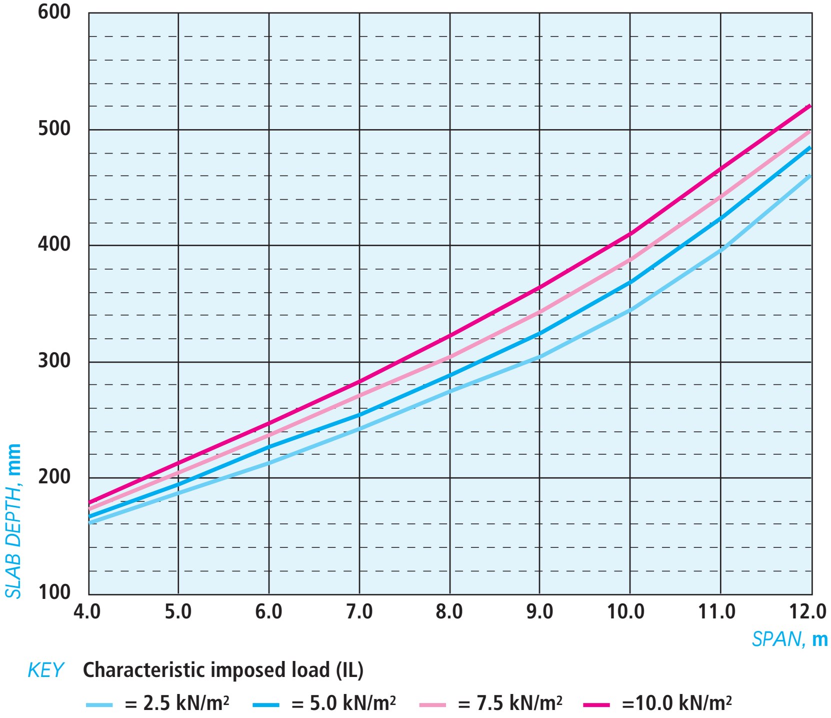
ADVANTAGES

- Relatively simple and fast formwork and construction
- Absence of beams allows lower storey heights
- Flexibility of partition location and horizontal service distribution

DISADVANTAGES

- Holes can prove difficult, especially large holes near columns
- Shear provision around columns may be considered a complication
- Deflections, especially at edges supporting cladding, may cause concern
- Drops may cause some disruption to formwork

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS. Refer to column charts and data to estimate sizes, etc. Minimum dimensions of columns as data.
DIMENSIONS	Square panels, minimum of three spans x three bays. Outside edge flush with columns. Drops: span/2 x span/2 x 50 mm deep.
REINFORCEMENT	Main bars: T20 uno. Links R8. To help deflection, 25% A_sT at first internal support used as A_s' at midspan of end spans. Service stress, f_s, may have been reduced.10% allowed for wastage and laps.
LOADS	SDL of 1.50 kN/m² (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads assume elastic reaction factors of 1.0 to internal columns and 0.5 to end columns.
CONCRETE	C35, 24 kN/m³, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
HOLES	One 150 mm square hole assumed to adjoin each column. Larger holes may invalidate the data below.

MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, (excluding drop), mm									
IL = 2.5 kN/m²	162	188	214	244	276	306	346	398	464
IL = 5.0 kN/m²	168	196	228	256	290	326	370	426	488
IL = 7.5 kN/m²	174	206	238	272	306	344	390	444	502
IL = 10.0 kN/m²	180	214	248	284	324	366	412	468	524
ULTIMATE LOAD TO SUPPORTING COLUMNS, internal (edge) per storey, MN									
IL = 2.5 kN/m²	0.2 (0.2)	0.3 (0.2)	0.5 (0.3)	0.7 (0.5)	1.0 (0.6)	1.4 (0.8)	1.8 (1.1)	2.4 (1.4)	3.2 (1.8)
IL = 5.0 kN/m²	0.3 (0.2)	0.4 (0.3)	0.7 (0.4)	1.0 (0.6)	1.3 (0.8)	1.8 (1.0)	2.3 (1.4)	3.0 (1.8)	3.9 (2.2)
IL = 7.5 kN/m²	0.3 (0.2)	0.5 (0.4)	0.8 (0.5)	1.2 (0.7)	1.6 (1.0)	2.1 (1.3)	2.8 (1.6)	3.6 (2.1)	4.6 (2.6)
IL = 10.0 kN/m²	0.4 (0.3)	0.7 (0.4)	1.0 (0.6)	1.4 (0.8)	1.9 (1.1)	2.5 (1.5)	3.3 (1.9)	4.2 (2.4)	5.2 (3.0)
REINFORCEMENT, kg/m² (kg/m³)									
IL = 2.5 kN/m²	10 (58)	14 (66)	17 (74)	21 (79)	24 (83)	30 (92)	35 (97)	39 (93)	44 (90)
IL = 5.0 kN/m²	12 (66)	16 (76)	20 (80)	25 (90)	29 (94)	36 (103)	40 (103)	46 (103)	49 (97)
IL = 7.5 kN/m²	14 (73)	18 (81)	23 (88)	28 (95)	35 (107)	40 (109)	46 (113)	50 (109)	56 (107)
IL = 10.0 kN/m²	16 (80)	21 (88)	26 (97)	31 (101)	37 (109)	45 (118)	50 (116)	55 (112)	61 (112)
COLUMN SIZES ASSUMED, mm square, internal (perimeter)									
IL = 2.5 kN/m²	250 (225)	250 (225)	270 (240)	330 (290)	390 (340)	450 (400)	520 (460)	600 (530)	680 (610)
IL = 5.0 kN/m²	250 (225)	250 (225)	310 (270)	370 (320)	440 (380)	510 (440)	580 (510)	670 (580)	760 (670)
IL = 7.5 kN/m²	250 (225)	280 (240)	350 (300)	410 (360)	480 (420)	560 (480)	640 (550)	720 (630)	810 (720)
IL = 10.0 kN/m²	250 (225)	310 (260)	380 (320)	450 (380)	530 (450)	610 (520)	690 (600)	780 (680)	870 (770)
DESIGN NOTES a = $q_k > 1.25 g_k$ b = $q_k > 5$ kN/m² f = shear critical (initially $v>2v_c$) g = T25s used h = T32s used									
IL = 2.5 kN/m²						f	fg	fg	h
IL = 5.0 kN/m²				f	f	fg	fg	fh	fh
IL = 7.5 kN/m²	b	b	bf	bf	bfg	bfg	bfh	bfh	bfh
IL = 10.0 kN/m²	ab	abf	abf	bf	bfg	bfh	bfh	bfh	bfh
LINKS, MAXIMUM NUMBER OF PERIMETERS (and percentage by weight of reinforcement), no. (%)									
IL = 2.5 kN/m²	3 (1.2%)	4 (2.0%)	4 (1.6%)	5 (1.9%)	5 (2.2%)	5 (2.0%)	5 (1.9%)	5 (2.0%)	5 (2.0%)
IL = 5.0 kN/m²	3 (1.0%)	4 (2.1%)	5 (2.6%)	5 (2.5%)	5 (2.4%)	6 (2.3%)	6 (2.7%)	6 (2.2%)	5 (2.3%)
IL = 7.5 kN/m²	4 (2.5%)	5 (3.2%)	5 (3.1%)	5 (2.8%)	6 (2.5%)	6 (2.9%)	6 (2.7%)	5 (2.6%)	5 (2.7%)
IL = 10.0 kN/m²	4 (3.3%)	5 (3.3%)	5 (3.2%)	6 (3.2%)	6 (2.8%)	7 (2.7%)	6 (3.1%)	5 (2.9%)	5 (3.0%)

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

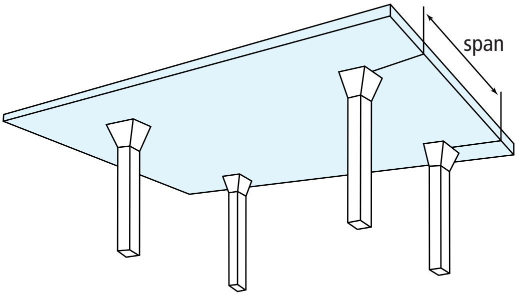
Fire resistance	2 hours	+10 mm	4 hours	+30 mm
Exposure	Moderate	+20 mm	Severe, C40 concrete	+25 mm
Cladding load	No cladding load	-0 mm	20 kN/m cladding load	+20 mm
Other	Drops L/3 wide	+15 mm	No holes	-0 mm
	150 mm drop	+5 mm	Limiting shear to $v<1.6v_c$	+5 mm
	Using T25s cf T20s	+10 mm	2 spans	+5 mm

Thickness, mm	**Span, m**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
	No shear links	274	370	474	526	628	630	
	No links, 150 drops	230	262	338	384	484	486	562
	Stiff edge; (basic l/d = 40)	258	294	330	374	412	478	548

Rectangular panels: equivalent spans, m Use an equivalent square span, below, to derive thickness

Long span m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
Short span = 5.0 m	5.3	5.8	6.8	7.7	8.3		
Short span = 6.0 m	6.0	6.4	6.9	7.8	8.5	9.3	10.2
Short span = 7.0 m		7.0	7.5	8.1	8.7	9.7	10.5
Short span = 8.0 m			8.0	8.5	9.1	9.9	10.6
Short span = 9.0 m				9.0	9.5	10.2	10.8
Short span =10.0 m					10.0	10.5	11.0
Short span =11.0 m						11.0	11.5

Flat slabs with column heads



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

Popular for office buildings, retail developments, hospitals, hotels, etc. Economical for more heavily loaded spans of from 6 to 10 m in square panels. However, unless the whole column can be poured at one time, column heads can disrupt cycle times.

The chart and data assume an edge loading of 10 kN/m and one 150 mm hole adjacent to each column head. They assume column head sizes will at least equal those given in the data.

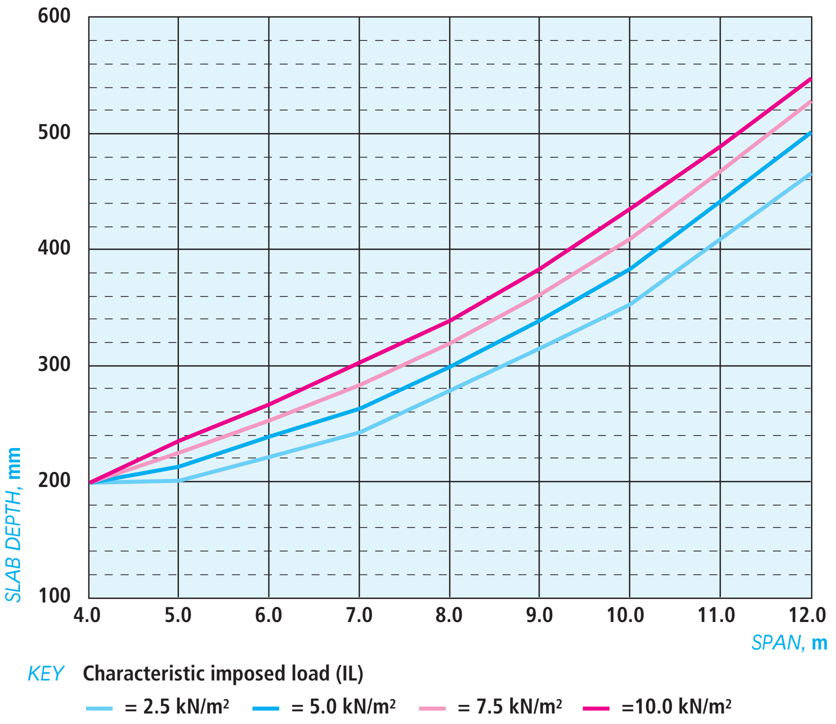
ADVANTAGES

- Relatively simple and fast formwork and construction
- Absence of beams allows lower storey heights
- Flexibility of partition location and horizontal service distribution

DISADVANTAGES

- Holes can prove difficult, especially large holes near columns
- Shear provision around columns may be considered difficult
- Deflections, especially at edges supporting cladding, may cause concern
- Column heads can disrupt cycle times

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS with column heads. Refer to column charts etc to estimate sizes of columns. Min. dimensions of column heads as data (internal, span/10; ends, span/20+150 mm).
DIMENSIONS	Square panels, minimum of three spans x three bays. Outside edge flush with columns.
REINFORCEMENT	Main bars: T20 uno. Links R8. To help deflection, 25% A_sT at first internal support used as A_s' at midspan of end spans. Service stress, f_s, may have been reduced.10% allowed for wastage and laps.
LOADS	SDL of 1.50 kN/m^2 (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads assume elastic reaction factors of 1.0 to internal columns and 0.5 to end columns.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
HOLES	One 150 mm square hole assumed to adjoin each column. Larger holes may invalidate the data below.

MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m^2	200	202	214	244	274	316	358	414	464
IL = 5.0 kN/m^2	200	202	226	256	290	330	374	430	482
IL = 7.5 kN/m^2	200	202	234	266	306	348	394	452	506
IL = 10.0 kN/m^2	200	204	242	278	318	362	412	468	528
ULTIMATE LOAD TO SUPPORTING COLUMNS, internal (edge) per storey, MN									
IL = 2.5 kN/m^2	0.2 (0.2)	0.3 (0.2)	0.5 (0.3)	0.7 (0.5)	1.0 (0.6)	1.4 (0.8)	1.8 (1.1)	2.4 (1.4)	3.1 (1.7)
IL = 5.0 kN/m^2	0.3 (0.2)	0.4 (0.3)	0.6 (0.4)	0.9 (0.6)	1.3 (0.7)	1.7 (1.0)	2.3 (1.3)	3.0 (1.6)	3.8 (2.1)
IL = 7.5 kN/m^2	0.3 (0.2)	0.5 (0.3)	0.8 (0.5)	1.1 (0.7)	1.6 (0.9)	2.1 (1.2)	2.7 (1.5)	3.5 (1.9)	4.4 (2.4)
IL = 10.0 kN/m^2	0.4 (0.3)	0.6 (0.4)	1.0 (0.6)	1.3 (0.8)	1.8 (1.0)	2.5 (1.4)	3.2 (1.7)	4.1 (2.2)	5.2 (2.8)
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	9 (46)	13 (63)	19 (90)	23 (93)	27 (99)	31 (99)	34 (95)	41 (99)	51 (111)
IL = 5.0 kN/m^2	11 (55)	17 (82)	22 (98)	27 (106)	33 (115)	38 (118)	42 (113)	49 (115)	58 (122)
IL = 7.5 kN/m^2	13 (66)	22 (108)	26 (112)	32 (121)	35 (116)	42 (120)	46 (118)	55 (121)	65 (130)
IL = 10.0 kN/m^2	17 (84)	27 (130)	30 (123)	36 (130)	40 (127)	47 (129)	51 (124)	61 (130)	68 (128)
COLUMN HEAD SIZES ASSUMED, mm									
Internal	400 sq.	500 sq.	600 sq.	700 sq.	800 sq.	900 sq.	1000 sq.	1100 sq.	1200 sq.
Perimeter	400 x 350	500 x 400	600 x 450	700 x 500	800 x 550	900 x 600	1000 x 650	1100 x 700	1200 x 750
Corner	350 sq.	400 sq.	450 sq.	500 sq.	550 sq.	600 sq.	650 sq.	700 sq.	750 sq.
DESIGN NOTES a = $q_k > 1.25\ g_k$ b = $q_k > 5$ kN/m^2 f = shear critical (initially $v>2v_c$) g = T25s used h = T32s used									
IL = 2.5 kN/m^2						g	g	h	h
IL = 5.0 kN/m^2					f	g	fg	fh	fh
IL = 7.5 kN/m^2	b	bf	bf	bf	bf	bfg	bfg	bfh	bfh
IL = 10.0 kN/m^2	abf	abf	abf	bf	bf	bfg	bfg	bfh	bfh
LINKS, MAXIMUM NUMBER OF PERIMETERS (and percentage by weight of reinforcement), no. (%)									
IL = 2.5 kN/m^2	4 (3.5%)	5 (2.4%)	5 (1.8%)	5 (1.8%)	6 (1.9%)	5 (1.7%)	6 (2.0%)	6 (2.0%)	6 (1.8%)
IL = 5.0 kN/m^2	5 (4.3%)	6 (3.3%)	6 (2.6%)	6 (2.3%)	6 (2.1%)	6 (1.8%)	6 (2.6%)	6 (2.8%)	6 (2.6%)
IL = 7.5 kN/m^2	6 (6.0%)	6 (3.7%)	6 (2.7%)	6 (2.8%)	6 (2.1%)	6 (2.7%)	6 (2.5%)	6 (3.1%)	6 (3.0%)
IL = 10.0 kN/m^2	6 (5.5%)	6 (3.1%)	6 (3.5%)	7 (3.2%)	6 (3.0%)	6 (2.9%)	6 (3.0%)	6 (3.1%)	6 (3.5%)

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

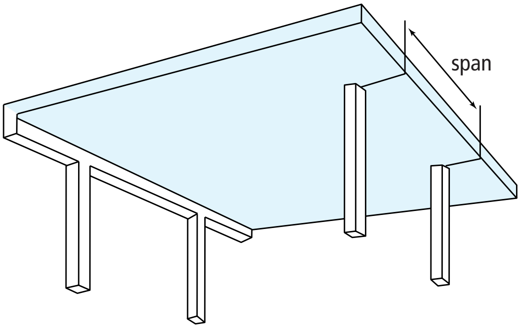
Fire resistance	2 hours	+10 mm	4 hours	+30 mm
Exposure	Moderate	+20 mm	Severe, C40 concrete	+25 mm
Cladding load	No cladding load	-0 mm	20 kN/m cladding load	+10 mm
Holes	No holes	-0 mm	300 square holes	+10 mm
Other	Using T25s cf T20s	+10 mm	2 spans	+10 mm

Thickness, mm	**Span, m**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
	No shear links	260	358	418	490	602	648	
	Stiff edge (basic l/d = 40)	264	300	340	384	426	496	568

Rectangular panels: equivalent spans, m Use an equivalent square span, below, to derive thickness

Long span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
Short span = 5.0 m	5.2	5.7	6.7	7.8	8.4	9.1	
Short span = 6.0 m	6.0	6.3	7.0	8.0	8.6	9.4	10.1
Short span = 7.0 m		7.0	7.4	8.2	8.8	9.7	10.4
Short span = 8.0 m			8.0	8.4	9.1	9.9	10.6
Short span = 9.0 m				9.0	9.4	10.2	10.8
Short span =10.0 m					10.0	10.5	11.0
Short span =11.0 m						11.0	11.5

Flat slabs with edge beams



Introducing edge beams to flat slabs overcomes many of the problems associated with shear at perimeter columns and edge deflection. These slabs are popular for use in office buildings, retail developments, hospitals, hotels, etc. and commonly incorporate upstands rather than downstand perimeter beams. They are economical for spans up to 10 m in square panels.

The chart and data assume an edge loading of 10 kN/m and one 150 mm hole in the slab adjacent to each column. They assume internal columns sizes will at least equal those given in the data. The overall depth of edge beams must be at least 50% greater than the slab thickness.

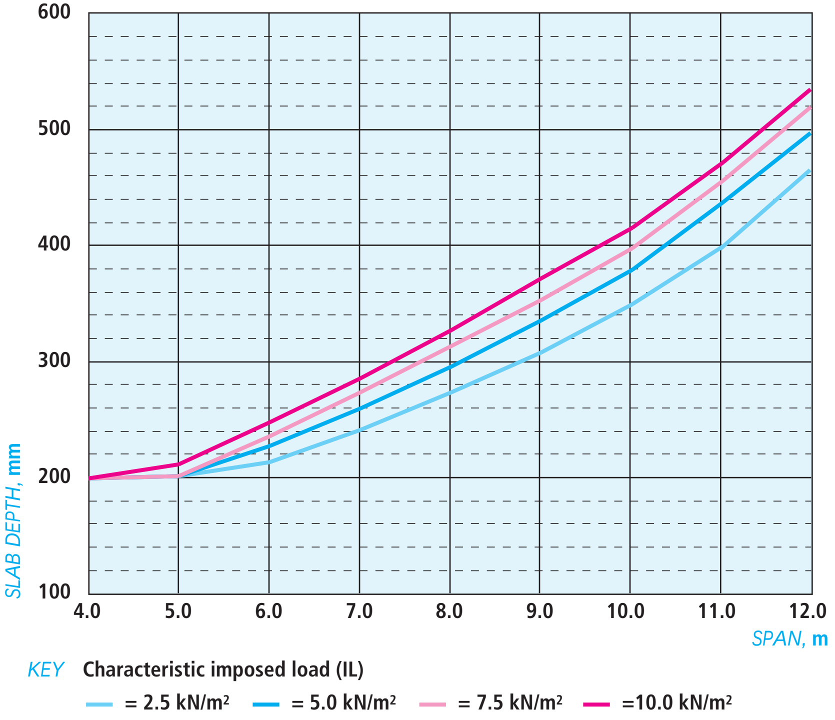
ADVANTAGES

- Relatively simple and fast formwork and construction
- Architectural finish can be applied directly to the underside of the slab
- Absence of internal beams allows lower storey heights
- Flexibility of partition location and horizontal service distribution.
- Perimeter holes present few problems

DISADVANTAGES

- Perimeter downstand beams may hinder use of table forms

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS internally and BEAMS around perimeter. Refer to appropriate charts and data to estimate sizes, etc. Minimum column size as data. Edge beams at least 50% deeper than slab.
DIMENSIONS	Square panels, minimum of three spans x three bays. Outside edge flush with columns.
REINFORCEMENT	Main bars: T20 uno. Links R8. To help with deflection, 25% A_sT used as A_s' at midspan of end spans. f_s may have been reduced. 10% allowed for wastage and laps. Beam reinforcement to be added.
LOADS	SDL of 1.50 kN/m^2 (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads assume elastic reaction factors of 1.0 to internal columns and 0.5 to edge beams.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.
HOLES	One 150 mm square hole assumed to adjoin each column. Larger holes may invalidate the data below.

MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
THICKNESS, mm									
IL = 2.5 kN/m^2	200	202	214	242	274	308	350	400	468
IL = 5.0 kN/m^2	200	202	228	260	296	336	380	438	500
IL = 7.5 kN/m^2	200	202	236	274	314	354	398	456	522
IL = 10.0 kN/m^2	200	212	248	286	328	372	416	472	538
ULTIMATE LOAD TO SUPPORTING COLUMNS, internal (edge) per storey, MN									
IL = 2.5 kN/m^2	0.2	0.3	0.5	0.7	1.0	1.3	1.8	2.4	3.1
IL = 5.0 kN/m^2	0.3	0.4	0.6	0.9	1.3	1.7	2.3	3.0	3.9
IL = 7.5 kN/m^2	0.3	0.5	0.8	1.1	1.6	2.1	2.7	3.6	4.6
IL = 10.0 kN/m^2	0.4	0.6	1.0	1.4	1.9	2.5	3.2	4.1	5.2
ULTIMATE LOADS ON (EDGE) BEAMS, kN/m *Includes perimeter load of 10 kN/m but excludes beam self weight*									
IL = 2.5 kN/m^2	(28)	(31)	(35)	(40)	(46)	(52)	(61)	(70)	(83)
IL = 5.0 kN/m^2	(32)	(36)	(42)	(49)	(56)	(64)	(74)	(86)	(99)
IL = 7.5 kN/m^2	(36)	(41)	(49)	(57)	(66)	(76)	(86)	(100)	(115)
IL = 10.0 kN/m^2	(40)	(47)	(56)	(65)	(75)	(87)	(99)	(113)	(129)
REINFORCEMENT, kg/m^2 (kg/m^3) *Beam reinforcement to be added*									
IL = 2.5 kN/m^2	8 (39)	10 (50)	15 (70)	18 (75)	22 (80)	26 (85)	30 (85)	35 (87)	40 (85)
IL = 5.0 kN/m^2	9 (46)	13 (65)	17 (75)	21 (81)	25 (85)	30 (88)	34 (90)	39 (89)	44 (88)
IL = 7.5 kN/m^2	11 (53)	17 (83)	20 (86)	24 (89)	29 (92)	33 (93)	39 (99)	43 (95)	47 (90)
IL = 10.0 kN/m^2	12 (61)	19 (87)	22 (91)	27 (97)	33 (101)	39 (104)	44 (106)	48 (101)	52 (96)
COLUMN SIZES ASSUMED, mm square, internal									
IL = 2.5 kN/m^2	250	250	260	320	380	440	510	590	680
IL = 5.0 kN/m^2	250	250	310	370	430	500	580	660	750
IL = 7.5 kN/m^2	250	280	340	410	480	550	630	720	820
IL = 10.0 kN/m^2	250	300	370	440	520	600	680	770	870
DESIGN NOTES a = $q_k > 1.25\ g_k$ b = $q_k > 5$ kN/m^2 f = shear critical (initially $v>2v_c$) g = T25s used h = T32s used									
IL = 2.5 kN/m^2							g	g	h
IL = 5.0 kN/m^2						g	g	h	h
IL = 7.5 kN/m^2	b	b	b	b	bg	bg	bh	bh	bh
IL = 10.0 kN/m^2	ab	ab	ab	b	bfg	bfh	bfh	bfh	bh
LINKS, MAXIMUM NUMBER OF PERIMETERS (and percentage by weight of reinforcement), no. (%)									
IL = 2.5 kN/m^2	0 (0.0%)	2 (0.7%)	3 (0.7%)	3 (0.6%)	4 (0.9%)	4 (1.0%)	4 (1.0%)	4 (1.1%)	4 (1.1%)
IL = 5.0 kN/m^2	2 (1.2%)	3 (1.0%)	3 (0.9%)	4 (1.2%)	4 (1.2%)	4 (1.2%)	4 (1.3%)	4 (1.2%)	5 (1.4%)
IL = 7.5 kN/m^2	3 (1.8%)	4 (1.5%)	4 (1.3%)	4 (1.3%)	4 (1.4%)	4 (1.4%)	4 (1.3%)	5 (1.6%)	5 (1.6%)
IL = 10.0 kN/m^2	3 (1.6%)	4 (1.7%)	4 (1.5%)	4 (1.6%)	5 (1.7%)	6 (2.0%)	5 (1.6%)	5 (1.6%)	5 (1.8%)

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

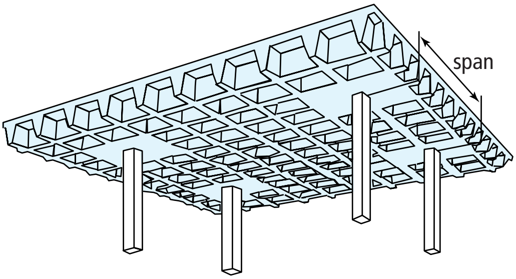
Fire resistance	2 hours	+5 mm	4 hours	+30 mm
Exposure	Moderate	+20 mm	Severe, C40 concrete	+25 mm
Other	300 square holes	+0 mm	50 mm drops, L/3 wide	-5 mm
	Using T25s cf T20s	+5 mm	2 spans	+ 0 mm

Thickness, mm	**Span, m**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
	No shear links	242	308	394	460	498	554	640

Rectangular panels: equivalent spans, m Use an equivalent square span, below, to derive thickness

Long span, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
Short span = 5.0 m	5.1	5.7	6.4	7.2	7.9	8.8	
Short span = 6.0 m	6.0	6.3	6.9	7.5	8.2	9.2	10.0
Short span = 7.0 m		7.0	7.4	8.0	8.6	9.4	10.2
Short span = 8.0 m			8.0	8.5	9.0	9.6	10.4
Short span = 9.0 m				9.0	9.3	9.9	10.7
Short span =10.0 m					10.0	10.4	11.0
Short span =11.0 m						11.0	11.4

Waffle slabs *designed as flat slabs (bespoke moulds)*



Introducing voids to the soffit of a flat slab reduces dead weight and these slabs are economical in spans up to 12 m in square panels. Thickness 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. The chart and data include an allowance for an edge loading of 10 kN/m.

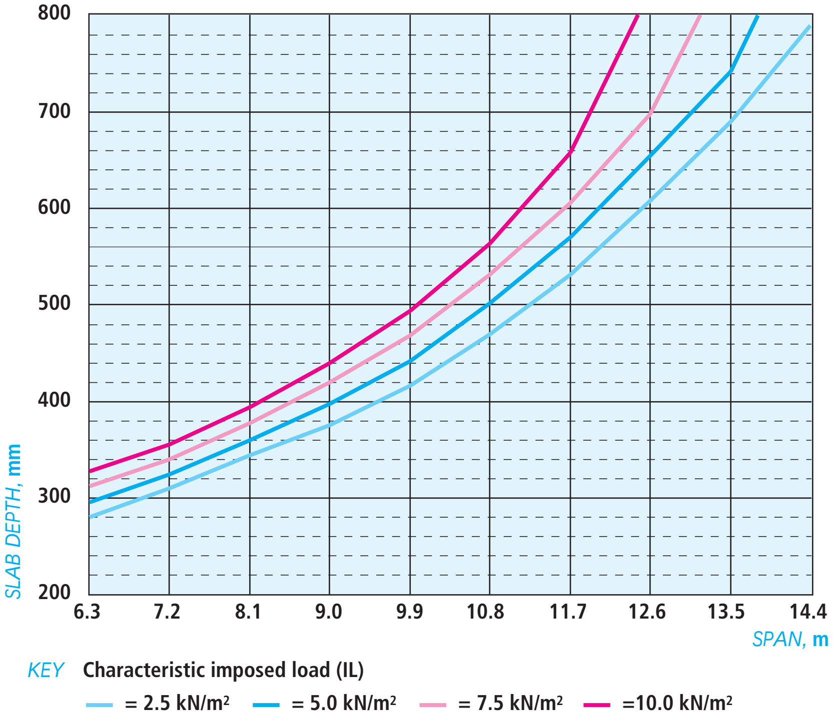
ADVANTAGES

- Profile may be expressed architecturally
- Flexibility of partition location and horizontal service distribution
- Lightweight

DISADVANTAGES

- Higher formwork costs than for other slab systems
- Slightly deeper members result in greater floor heights
- Difficult to prefabricate, therefore reinforcement may be slow to fix

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS. Refer to column charts and data to estimate sizes, etc.
DIMENSIONS	Square panels, minimum of three spans x three bays. Ribs 150 mm wide @ 900 mm cc. Topping 100 mm. Moulds variable depth. Solid area ≯ span/2 in each direction.
REINFORCEMENT	Max. bar sizes, ribs: 2T32B, T32T and R8 links. 25 mm allowed for A142 mesh T (@ 0.12%) in topping. 10% allowed for wastage, etc. f_s may have been reduced.
LOADS	SDL of 1.50 kN/m^2 (finishes) and perimeter load of 10 kN/m (cladding) included. Ultimate loads to columns assume elastic reaction factors of 1.0 internally and 0.5 at ends. Self weight used accounts for 5:1 slope to ribs and solid areas as described above.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.

MULTIPLE SPAN, m	**6.3**	**7.2**	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**	**13.5**
THICKNESS, mm									
IL = 2.5 kN/m^2	280	310	344	376	416	470	532	608	690
IL = 5.0 kN/m^2	296	324	360	398	442	502	570	654	742
IL = 7.5 kN/m^2	312	340	378	420	468	532	606	698	862
IL = 10.0 kN/m^2	328	356	394	440	494	564	658	826	
ULTIMATE LOAD TO SUPPORTING COLUMNS, internal (edge) per storey, MN									
IL = 2.5 kN/m^2	0.5 (0.3)	0.7 (0.4)	0.9 (0.5)	1.2 (0.7)	1.5 (0.8)	1.9 (1.1)	2.4 (1.3)	3.1 (1.7)	3.9 (2.1)
IL = 5.0 kN/m^2	0.7 (0.4)	0.9 (0.5)	1.2 (0.7)	1.5 (0.9)	1.9 (1.1)	2.5 (1.4)	3.1 (1.7)	4.0 (2.1)	4.9 (2.7)
IL = 7.5 kN/m^2	0.8 (0.5)	1.1 (0.6)	1.5 (0.8)	1.9 (1.0)	2.4 (1.3)	3.0 (1.7)	3.8 (2.0)	4.8 (2.6)	6.2 (3.4)
IL = 10.0 kN/m^2	1.0 (0.6)	1.4 (0.8)	1.7 (1.0)	2.3 (1.2)	2.8 (1.5)	3.6 (1.9)	4.5 (2.4)	5.9 (3.2)	
REINFORCEMENT, kg/m^2 (kg/m^3)									
IL = 2.5 kN/m^2	16 (56)	28 (99)	31 (98)	25 (67)	27 (66)	30 (64)	32 (60)	34 (56)	37 (53)
IL = 5.0 kN/m^2	20 (66)	26 (79)	28 (78)	31 (78)	33 (76)	35 (70)	36 (63)	39 (60)	42 (56)
IL = 7.5 kN/m^2	23 (72)	31 (92)	33 (88)	35 (84)	38 (83)	38 (72)	41 (68)	44 (63)	45 (52)
IL = 10.0 kN/m^2	26 (78)	35 (100)	37 (94)	40 (90)	43 (88)	43 (77)	45 (68)	46 (56)	
COLUMN SIZES ASSUMED, mm square, internal (perimeter)									
IL = 2.5 kN/m^2	270 (260)	310 (300)	350 (340)	420 (390)	460 (440)	530 (510)	600 (570)	680 (650)	760 (730)
IL = 5.0 kN/m^2	310 (290)	360 (340)	410 (390)	470 (440)	530 (500)	600 (570)	670 (640)	760 (720)	850 (810)
IL = 7.5 kN/m^2	350 (320)	410 (370)	460 (420)	530 (490)	590 (550)	670 (620)	740 (700)	830 (790)	950 (900)
IL = 10.0 kN/m^2	380 (340)	440 (400)	500 (460)	570 (530)	640 (590)	720 (670)	810 (750)	930 (870)	
DESIGN NOTES a = $q_k > 1.25\ g_k$ b = $q_k > 5$ kN/m^2 f = shear critical (initially $v>2v_c$) j = links in ribs close to solid area									
IL = 2.5 kN/m^2	fj	fj	fj	fj	fj	fj	fj	fj	fj
IL = 5.0 kN/m^2	fj	fj	fj	fj	fj	fj	fj	fj	fj
IL = 7.5 kN/m^2	bfj	bfj	bfj	bfj	bfj	bfj	bfj	bfj	bj
IL = 10.0 kN/m^2	abfj	abfj	abfj	bfj	bfj	bfj	bfj	bfj	
LINKS, MAXIMUM NUMBER OF PERIMETERS IN SOLID AREAS (and percentage by weight of reinforcement, all links), no. (%)									
IL = 2.5 kN/m^2	7 (6.7%)	6 (2.4%)	7 (2.3%)	7 (3.9%)	7 (4.2%)	6 (3.7%)	6 (4.4%)	5 (4.6%)	4 (5.2%)
IL = 5.0 kN/m^2	7 (7.6%)	7 (4.1%)	7 (4.9%)	7 (4.3%)	7 (5.6%)	7 (5.1%)	5 (5.6%)	5 (6.6%)	5 (7.4%)
IL = 7.5 kN/m^2	7 (7.5%)	6 (4.6%)	7 (6.0%)	7 (5.6%)	7 (6.9%)	6 (5.7%)	5 (7.1%)	5 (7.5%)	4 (9.8%)
IL = 10.0 kN/m^2	6 (7.7%)	6 (5.5%)	7 (6.5%)	7 (6.5%)	6 (7.5%)	5 (6.7%)	5 (7.9%)	5 (9.8%)	

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Fire resistance	2 hours,115 mm topping#	+20 mm	4 hours	not usually feasible
Exposure	Moderate #	+20 mm	Severe, C40 concrete#	+25 mm
Cladding load	No cladding load	- 20 mm	20 kN/m cladding	+ 40 mm if <12.6 m
	Edge beams	- 20 mm	Column head L/10 square	-0 mm
Holes	No holes, 225 holes	+0 mm	300 mm sq. holes	+ 0 mm if >8.1 m

175 rib width required

Thickness, mm

Span, m	**8.1**	**9.0**	**9.9**	**10.8**	**11.7**	**12.6**	**13.5**
No shear links	486	550	608	644	700	794	882
50 mm drop, L/2 wi	348	388	426	484	548	628	714
2 spans	378	422	466	536	614	746	

Rectangular panels

For non-square panels use an equivalent square span to derive thickness

Long span, m	**9.0**	**10.8**	**12.6**	**14.4**	**16.2**	**18.0**
Short span = 9.0 m	9.0	9.4	10.7	12.1	13.4	14.4
Short span = 9.9 m		9.7	11.0	12.4	13.5	14.4
Short span =10.8 m		10.8	11.3	12.7	13.8	
Short span =11.7m			11.7	12.9	14.0	
Short span =12.6 m			12.6	13.2	14.3	
Short span =13.5 m				13.4	14.5	

3.2 Beams

3.2.1 USING IN-SITU BEAMS

In-situ beams provide support: they transfer loads from slabs to columns and walls. They offer strength, robustness and versatility, eg. 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. Beams and columns of the same width give maximum formwork efficiency as formwork can proceed along a continuous line. However, used internally, these relatively deep beams result in additional perimeter cladding and tend to disrupt service runs. Deep 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.

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

3.2.2 USING THE CHARTS AND DATA

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:
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

In the charts, sizes of singly reinforced beams are shown using solid lines; sizes of beams with two layers of reinforcement are shown using dashed lines. As the use of beams with two layers of reinforcement should normally be avoided, no further information is given.

The user must determine which form of beam is appropriate and, therefore, which chart and data to use. From the appropriate chart(s) and data, use the maximum span and appropriate **ultimate** applied uniformly distributed loads (uaudl) to interpolate between values given in the charts and data. The user is expected to make adjustments for two-span configurations, etc. and to round up both the depth and loads to supports in line with his or her confidence in the design criteria used and normal modular sizing.

3.2.3 DESIGN ASSUMPTIONS

Design
The charts and data are based on moment and shear factors in BS 8110, Pt 1(2), table 3.6 assuming end spans are critical. Assumptions about dimensions are given in the table below. See also Section 7.

In order to satisfy defection criteria, service stress, f_s, is, in very many cases (particularly with shallow beams), reduced by increasing steel contents.

Dimensions

Beam type	Rect.	'L'	'T'
Flange width, single span	bw	bw + 0.10L *where L = span*	bw + 0.20L
Flange width, continuous spans	bw	bw + 0.07L *where L = span*	bw + 0.14L
Top flange thickness		100	100
Nom. top bars	T16	T16	T16
Allowance above T1 bars in continuous beams for links, mesh, bars, etc. perpendicular to span	13 mm	15 mm	23 mm

Reinforcement
Main bars: maximum T32s top and bottom, T10 links. 10% allowed for wastage and laps. Nominal top steel in mid-span. Minimum 50 mm between bars.

For reinforcement quantities, please refer to Section 2.2.4

Concrete
C35, 24 kN/m^3, 20 mm aggregate. For severe exposure, C40 is assumed.

Fire and durability
Fire resistance 1 hour; mild exposure.

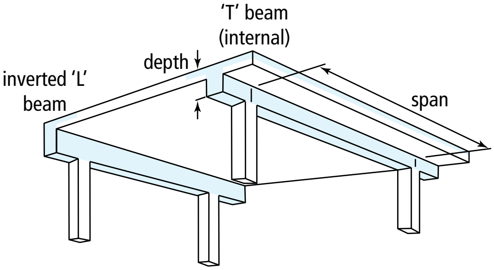
Loads
Beam self-weight (extra over an assumed 200 mm depth of solid slab) allowed for and included.

In line with BS 8110, Pt 1, Cl 3.8.2.3, ultimate loads to columns assume elastic reaction factors of 1.0 to internal columns supporting continuous beams and 0.5 to end columns.

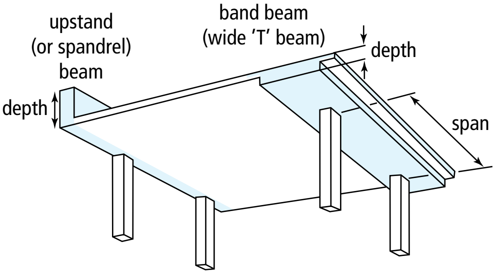
3.2.4 DESIGN NOTES

Different design criteria can be critical across the range of beams described. The sizes given in the charts and data are critical on the following parameters:

K	Beams 20 mm shallower than those given in the charts cannot be designed because K, (M/bd^2f_{cu}) at supports, exceeds maximum allowable (0.225)
a	A_sB (area of steel, bottom) restricted by end support width or length
b	Compression steel required at internal supports but does not exceed nominal percentage of 30% A_sB
c	Compression steel required at internal supports exceeds 30% A_sB (ie. special curtailment required)
d	Two layers of reinforcement
e	Compression steel required in top of span



In-situ concrete beams: 'T' and inverted 'L' beams
The slab data assume that internal beams support three-span slabs. Internal beams supporting two-span slabs might attract more load.



Upstand and band beams
Upstand beams and shallow downstand beams can be easier to construct and have less impact on horizontal services distribution and floor-to-floor heights.

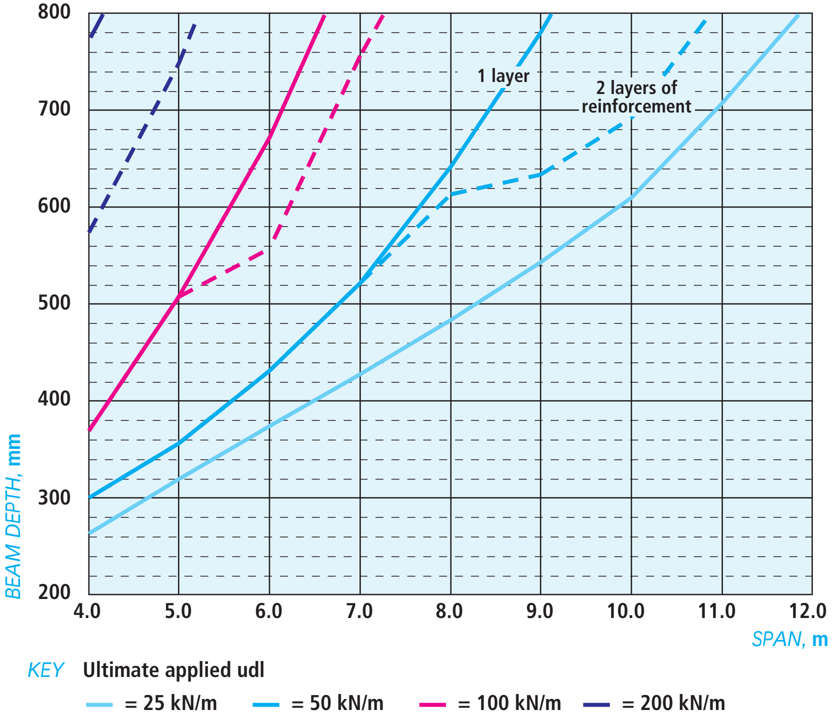
Rectangular beams

300 mm

wide

single span

SPAN:DEPTH CHART



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	264	320	374	428	484	544	610	708	816
uaudl = 50 kN/m	300	356	432	522	642	780	942		
uaudl = 100 kN/m	370	508	672						
uaudl = 200 kN/m	788								
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	n/a (55)	n/a (71)	n/a (86)	n/a (102)	n/a (120)	n/a (137)	n/a (156)	n/a (177)	n/a (199)
uaudl = 50 kN/m	n/a (106)	n/a (134)	n/a (163)	n/a (193)	n/a (226)	n/a (260)	n/a (297)		
uaudl = 100 kN/m	n/a (207)	n/a (263)	n/a (320)						
uaudl = 200 kN/m	n/a (416)								
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 25 kN/m	20 (249)	16 (170)	20 (181)	21 (164)	22 (148)	24 (144)	24 (133)	25 (117)	26 (105)
uaudl = 50 kN/m	21 (228)	24 (223)	25 (195)	26 (168)	26 (136)	27 (114)	27 (97)		
uaudl = 100 kN/m	29 (257)	27 (176)	26 (131)						
uaudl = 200 kN/m	21 (87)								
DESIGN NOTES							*See Section 3.2.4 on p 47*		
uaudl = 25 kN/m	a		a	a	a	a	ad	ad	ad
uaudl = 50 kN/m	ae	ae	ade	ade	ad	ad	ad		
uaudl = 100 kN/m	ae	ade	ad						
uaudl = 200 kN/m	d								

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl, mm

	4.0	5.0	6.0	7.0	8.0
2 hours fire	+5 mm				
4 hours fire	368	452	576	732	924
Moderate exposure	318	418	542	696	888
Severe exposure (C40)	330	414	538	692	884

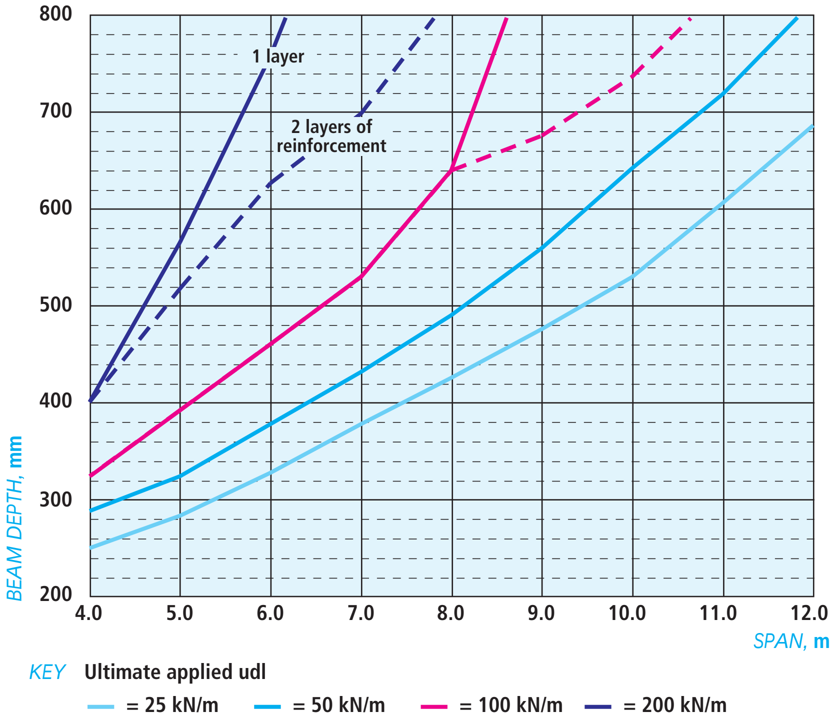
Rectangular beams

600 mm

wide

single span

SPAN:DEPTH CHART

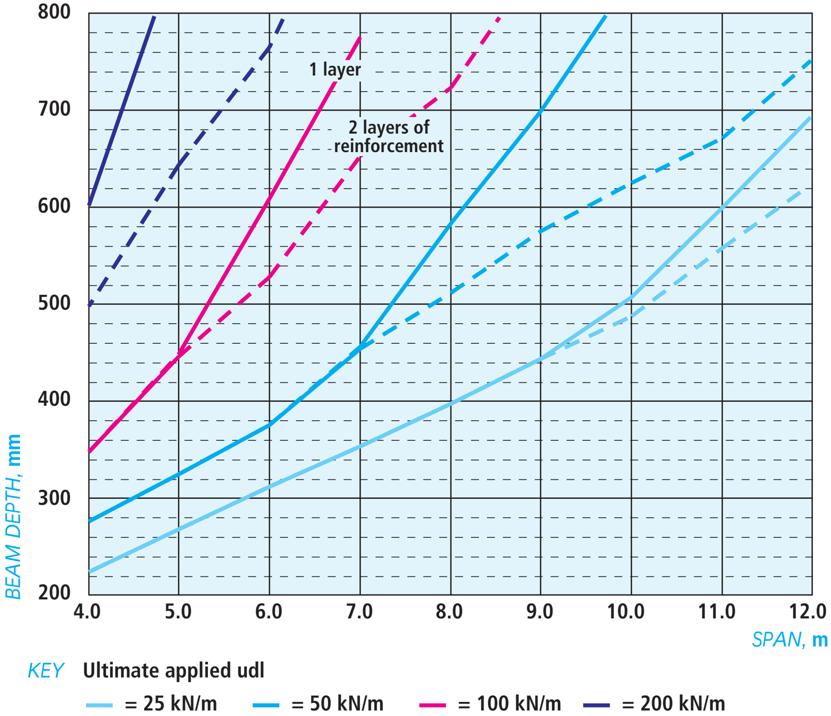


SINGLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
DEPTH, mm									
uaudl = 25 kN/m	252	286	330	380	428	478	532	608	688
uaudl = 50 kN/m	290	326	380	434	492	562	644	720	816
uaudl = 100 kN/m	326	394	462	532	642	898			
uaudl = 200 kN/m	402	568	760	994					
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	n/a (60)	n/a (77)	n/a (95)	n/a (115)	n/a (134)	n/a (157)	n/a (178)	n/a (205)	n/a (233)
uaudl = 50 kN/m	n/a (112)	n/a (141)	n/a (173)	n/a (206)	n/a (240)	n/a (276)	n/a (315)	n/a (354)	n/a (399)
uaudl = 100 kN/m	n/a (213)	n/a (270)	n/a (328)	n/a (387)	n/a (452)	n/a (531)			
uaudl = 200 kN/m	n/a (416)	n/a (529)	n/a (646)	n/a (770)					
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 25 kN/m	27 (177)	29 (167)	29 (145)	28 (122)	33 (129)	32 (111)	37 (117)	40 (109)	44 (106)
uaudl = 50 kN/m	30 (170)	30 (154)	33 (146)	36 (140)	39 (133)	42 (124)	44 (115)	48 (114)	50 (103)
uaudl = 100 kN/m	35 (178)	39 (166)	44 (159)	50 (159)	51 (132)	47 (87)			
uaudl = 200 kN/m	46 (192)	44 (129)	46 (100)	47 (80)					
DESIGN NOTES								*See Section 3.2.4 on p 47*	
uaudl = 25 kN/m									a
uaudl = 50 kN/m	d				a	a	ad		d
uaudl = 100 kN/m	e	ad	ad	de	d	d			
uaudl = 200 kN/m	ae	d	d	d					

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

2 hours fire	+5 mm	up to 10 m only
4 hours fire	+35 mm	up to 10 m only
Moderate exposure	+15 mm	up to 10 m only
Severe exposure (C40)	+20 mm	up to 10 m only

SPAN:DEPTH CHART



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	224	268	312	354	398	444	508	600	696
uaudl = 50 kN/m	276	324	376	456	584	700	838	1000	
uaudl = 100 kN/m	348	448	610	776					
uaudl = 200 kN/m	602	870							
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	109 (55)	139 (69)	169 (84)	200 (100)	232 (116)	265 (133)	301 (151)	342 (171)	384 (192)
uaudl = 50 kN/m	212 (106)	266 (133)	322 (161)	382 (191)	447 (224)	514 (257)	584 (292)	661 (330)	
uaudl = 100 kN/m	414 (207)	523 (261)	637 (318)	755 (377)					
uaudl = 200 kN/m	824 (412)	1044 (522)							
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 25 kN/m	19 (286)	22 (268)	22 (236)	23 (213)	25 (213)	28 (210)	29 (191)	29 (160)	29 (139)
uaudl = 50 kN/m	20 (219)	25 (259)	29 (263)	30 (220)	29 (164)	29 (139)	30 (119)	31 (102)	
uaudl = 100 kN/m	28 (267)	31 (230)	30 (162)	30 (129)					
uaudl = 200 kN/m	27 (149)	27 (105)							
DESIGN NOTES							*See Section 3.2.4 on p 47*		
uaudl = 25 kN/m	abe	ab	a	ab	ab	ab	ad	ad	ad
uaudl = 50 kN/m	ab	K ace	K ace	abd	ad	ad	ad	ad	
uaudl = 100 kN/m	K ace	abde	ad	ad					
uaudl = 200 kN/m	d	d							

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl, mm

	4.0	5.0	6.0	7.0	8.0
2 hours fire	+5 mm				
4 hours fire	328	398	506	638	800
Moderate exposure	302	384	494	626	788
Severe exposure (C40)	304	382	490	622	784

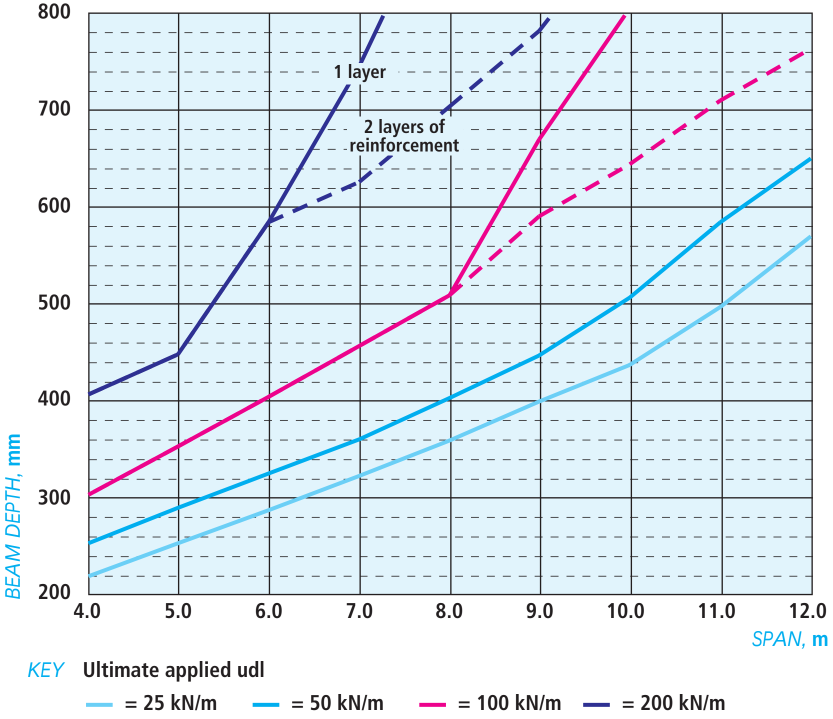
Rectangular beams

600 mm

wide

multiple span

SPAN:DEPTH CHART



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	218	252	286	322	358	398	436	496	570
uaudl = 50 kN/m	252	288	324	360	402	446	506	584	650
uaudl = 100 kN/m	302	352	404	456	508	670	806	966	
uaudl = 200 kN/m	406	448	584	746	942				
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	116 (58)	150 (75)	185 (92)	221 (111)	258 (129)	297 (149)	338 (169)	385 (192)	438 (219)
uaudl = 50 kN/m	220 (110)	277 (139)	339 (170)	401 (200)	465 (232)	531 (265)	602 (301)	680 (340)	757 (379)
uaudl = 100 kN/m	423 (212)	535 (268)	645 (322)	764 (382)	881 (441)	1022 (511)	1162 (581)	1314 (657)	
uaudl = 200 kN/m	833 (416)	1045 (523)	1271 (635)	1505 (753)	1752 (876)				
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 25 kN/m	32 (262)	26 (171)	28 (162)	29 (147)	33 (154)	36 (149)	38 (147)	41 (138)	42 (123)
uaudl = 50 kN/m	29 (190)	38 (235)	36 (183)	41 (190)	50 (207)	54 (200)	57 (186)	58 (164)	61 (155)
uaudl = 100 kN/m	39 (225)	41 (193)	55 (238)	59 (214)	67 (221)	60 (148)	60 (125)	61 (106)	
uaudl = 200 kN/m	43 (175)	61 (226)	61 (174)	62 (137)	62 (110)				
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 25 kN/m									
uaudl = 50 kN/m	a	ab		b	ab	ab	ad	a	
uaudl = 100 kN/m	abe	b	ab	abd	be	d	d	d	
uaudl = 200 kN/m		abe	d	d	d				

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

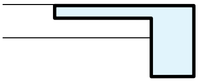
2 hours fire	+5 mm
4 hours fire	+25 mm
Moderate exposure	+15 mm
Severe exposure (C40)	+20 mm

Inverted 'L' beams

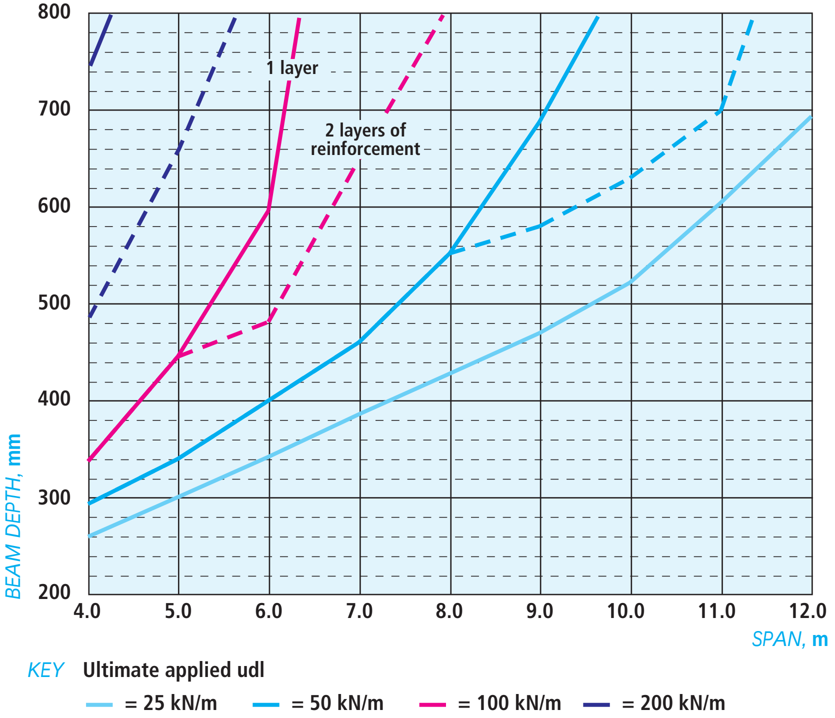
300 mm

wide web

single span



SPAN:DEPTH CHART



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	260	300	342	386	428	470	522	604	694
uaudl = 50 kN/m	294	340	400	460	552	688	856		
uaudl = 100 kN/m	338	446	596						
uaudl = 200 kN/m	748								
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	n/a (53)	n/a (68)	n/a (82)	n/a (98)	n/a (113)	n/a (129)	n/a (146)	n/a (165)	n/a (186)
uaudl = 50 kN/m	n/a (104)	n/a (131)	n/a (159)	n/a (188)	n/a (218)	n/a (252)	n/a (288)		
uaudl = 100 kN/m	n/a (205)	n/a (259)	n/a (315)						
uaudl = 200 kN/m	n/a (413)								
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 25 kN/m	19 (251)	16 (182)	20 (199)	20 (173)	23 (183)	24 (173)	25 (158)	25 (138)	26 (124)
uaudl = 50 kN/m	18 (203)	22 (212)	24 (199)	26 (185)	27 (160)	26 (128)	27 (105)		
uaudl = 100 kN/m	26 (254)	27 (199)	27 (149)						
uaudl = 200 kN/m	21 (91)								
DESIGN NOTES							*See Section 3.2.4 on p 47*		
uaudl = 25 kN/m	a	a	a	a	a	a	ad	a	ad
uaudl = 50 kN/m	a	a	a	ad	ad	ad	ad		
uaudl = 100 kN/m	a	ad	ad						
uaudl = 200 kN/m	d								

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl, mm

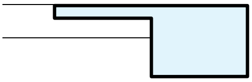
	4.0	5.0	6.0	7.0	8.0
2 hours fire	+5 mm				
4 hours fire	340	408	520	680	874
Moderate exposure	308	374	486	642	836
Severe exposure (C40)	312	380	488	648	842

Inverted 'L' beams

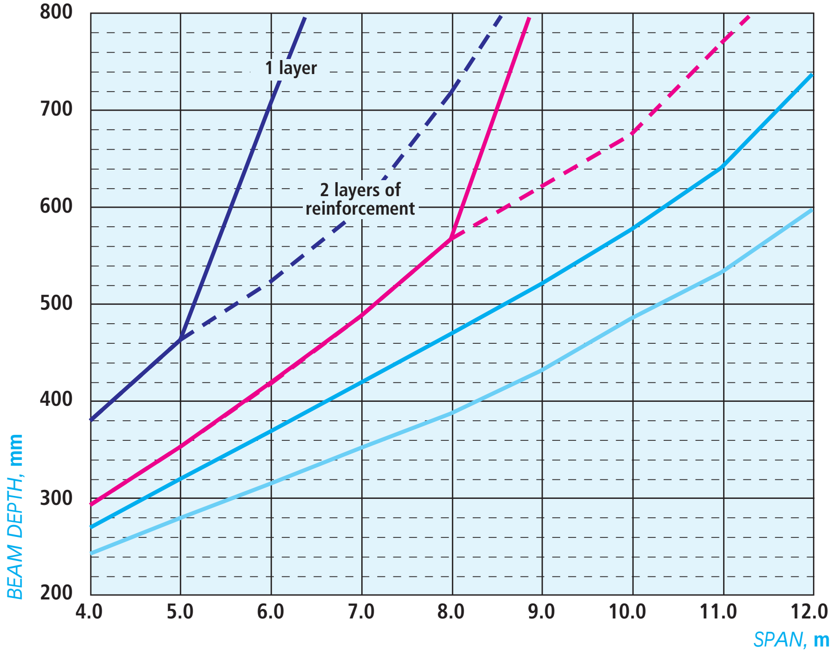
600 mm

wide web

single span



SPAN:DEPTH CHART



KEY **Ultimate applied udl**

— = 25 kN/m — = 50 kN/m — = 100 kN/m — = 200 kN/m

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	244	280	316	352	388	432	486	534	598
uaudl = 50 kN/m	270	322	370	420	470	520	584	642	738
uaudl = 100 kN/m	294	354	420	482	568	830			
uaudl = 200 kN/m	380	464	708						
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	n/a (55)	n/a (71)	n/a (88)	n/a (105)	n/a (123)	n/a (143)	n/a (164)	n/a (186)	n/a (210)
uaudl = 50 kN/m	n/a (107)	n/a (136)	n/a (166)	n/a (198)	n/a (229)	n/a (263)	n/a (299)	n/a (335)	n/a (377)
uaudl = 100 kN/m	n/a (208)	n/a (263)	n/a (319)	n/a (377)	n/a (438)	n/a (516)			
uaudl = 200 kN/m	n/a (411)	n/a (518)	n/a (637)						
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 25 kN/m	29 (205)	31 (198)	31 (161)	30 (140)	33 (143)	35 (136)	35 (120)	41 (129)	45 (126)
uaudl = 50 kN/m	32 (202)	28 (144)	33 (152)	34 (133)	39 (141)	40 (128)	43 (122)	48 (124)	49 (111)
uaudl = 100 kN/m	40 (224)	40 (187)	43 (172)	48 (166)	50 (147)	46 (93)			
uaudl = 200 kN/m	45 (196)	51 (182)	46 (107)						
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 25 kN/m		a							
uaudl = 50 kN/m	a				a	a	ad		d
uaudl = 100 kN/m	a	a	ad		d	d			
uaudl = 200 kN/m	ad	d	d						

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

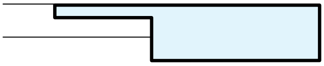
2 hours fire	+5 mm up to 10 m only
4 hours fire	+30 mm up to 10 m only
Moderate exposure	+16 mm up to 10 m only
Severe exposure (C40)	+20 mm up to 10 m only

Inverted 'L' beams

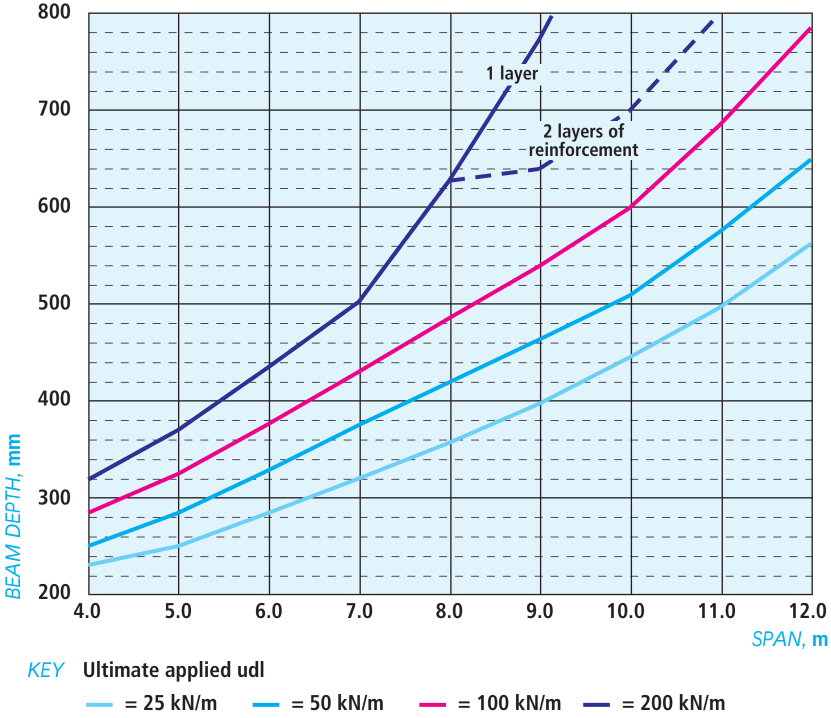
1200 mm

wide web

single span



SPAN:DEPTH CHART



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	232	252	286	322	358	398	446	498	564
uaudl = 50 kN/m	252	286	330	376	420	464	510	576	650
uaudl = 100 kN/m	286	326	378	432	486	540	600	686	786
uaudl = 200 kN/m	320	362	436	502	628	774	948		
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	n/a (62)	n/a (78)	n/a (97)	n/a (121)	n/a (141)	n/a (167)	n/a (196)	n/a (225)	n/a (262)
uaudl = 50 kN/m	n/a (112)	n/a (144)	n/a (179)	n/a (215)	n/a (252)	n/a (291)	n/a (333)	n/a (381)	n/a (433)
uaudl = 100 kN/m	n/a (215)	n/a (272)	n/a (333)	n/a (397)	n/a (462)	n/a (529)	n/a (599)	n/a (680)	n/a (766)
uaudl = 200 kN/m	n/a (416)	n/a (526)	n/a (641)	n/a (756)	n/a (885)	n/a (1020)	n/a (1170)		
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 25 kN/m	50 (165)	55 (181)	52 (151)	46 (114)	54 (127)	57 (120)	56 (103)	65 (110)	71 (105)
uaudl = 50 kN/m	56 (184)	53 (155)	50 (123)	51 (111)	58 (116)	64 (114)	68 (112)	76 (110)	82 (106)
uaudl = 100 kN/m	57 (166)	64 (166)	64 (143)	68 (132)	72 (124)	81 (126)	88 (125)	92 (111)	96 (102)
uaudl = 200 kN/m	80 (222)	82 (188)	87 (166)	96 (162)	97 (128)	99 (106)	101 (89)		
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 25 kN/m									
uaudl = 50 kN/m									
uaudl = 100 kN/m									d
uaudl = 200 kN/m	a		a		d	d	d		

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

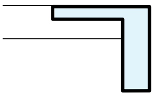
2 hours fire	+5 mm
4 hours fire	+35 mm up to 10 m only
Moderate exposure	+16 mm up to 10 m only
Severe exposure (C40)	+20 mm up to 10 m only

Inverted 'L' beams

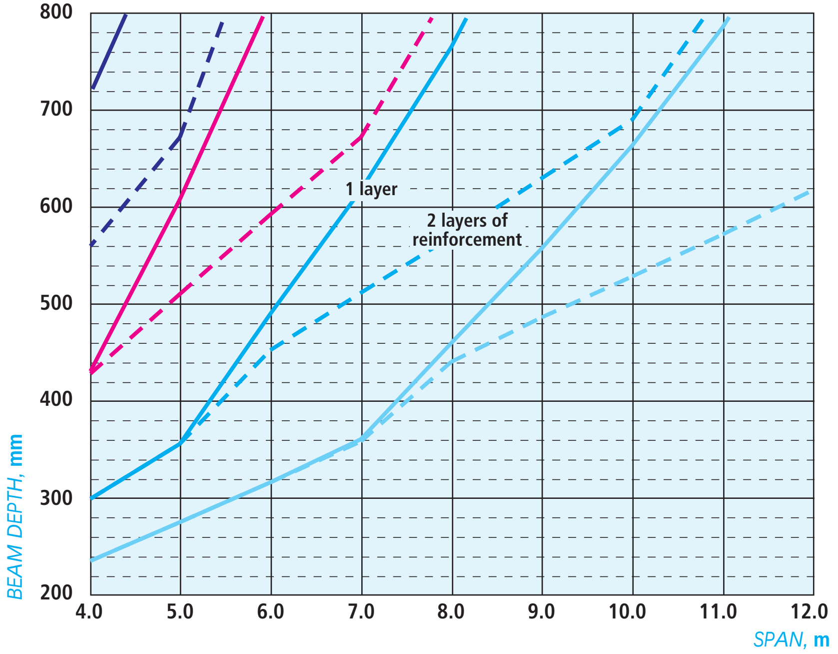
225 mm

wide web

multiple span



SPAN:DEPTH CHART



KEY **Ultimate applied udl**

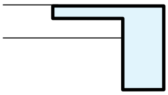
▬ = 25 kN/m ▬ = 50 kN/m ▬ = 100 kN/m ▬ = 200 kN/m

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	234	274	316	360	460	558	664	786	932
uaudl = 50 kN/m	298	356	490	618	766	944			
uaudl = 100 kN/m	430	608	814						
uaudl = 200 kN/m	724								
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	104 (52)	132 (66)	160 (80)	189 (94)	222 (111)	256 (128)	293 (146)	332 (166)	375 (188)
uaudl = 50 kN/m	206 (103)	260 (130)	318 (159)	377 (189)	440 (220)	507 (254)			
uaudl = 100 kN/m	410 (205)	519 (260)	632 (316)						
uaudl = 200 kN/m	819 (409)								
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 25 kN/m	15 (288)	17 (279)	19 (271)	21 (263)	21 (202)	21 (168)	22 (144)	22 (124)	22 (106)
uaudl = 50 kN/m	18 (275)	22 (269)	21 (189)	21 (154)	22 (127)	22 (105)			
uaudl = 100 kN/m	22 (229)	22 (162)	23 (123)						
uaudl = 200 kN/m	24 (145)								
DESIGN NOTES							*See Section 3.2.4 on p 47*		
uaudl = 25 kN/m	K ac	K ac	K c	cd	d	d	d	d	d
uaudl = 50 kN/m	K ac	K c	d	d	d	d			
uaudl = 100 kN/m	bd	d	d						
uaudl = 200 kN/m	d								

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

2 hours fire	+10 mm
4 hours fire	not appropriate
Moderate exposure	not appropriate
Severe exposure (C40)	not appropriate

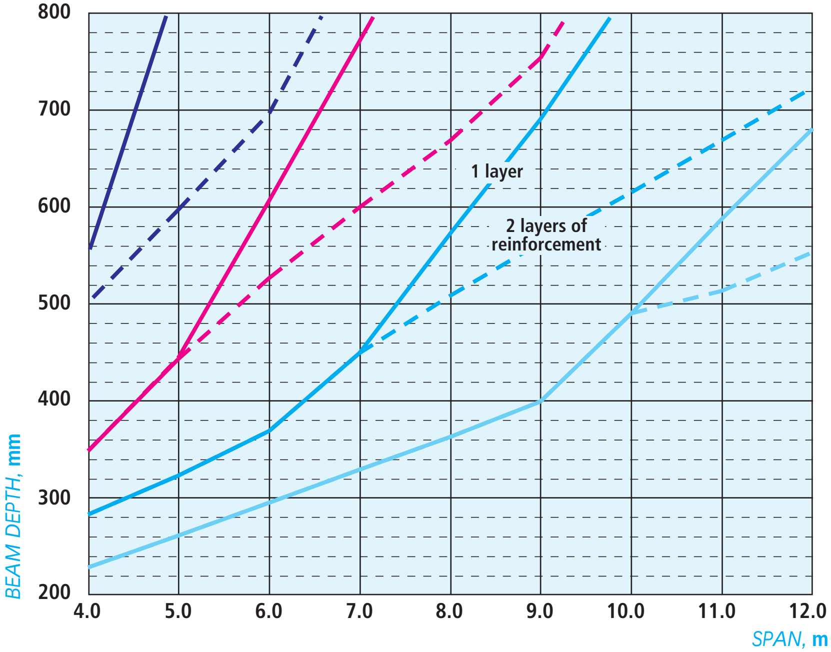
Inverted 'L' beams *multiple span*



300 mm

wide web

SPAN:DEPTH CHART



KEY **Ultimate applied udl**

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

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	230	264	296	330	362	398	494	586	682
uaudl = 50 kN/m	284	324	370	450	574	692	828	988	
uaudl = 100 kN/m	350	446	608	772					
uaudl = 200 kN/m	552	836							
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	105 (52)	133 (66)	162 (81)	191 (95)	221 (111)	252 (126)	290 (145)	329 (164)	370 (185)
uaudl = 50 kN/m	207 (104)	261 (131)	316 (158)	375 (187)	438 (219)	504 (252)	573 (287)	648 (324)	
uaudl = 100 kN/m	410 (205)	517 (259)	631 (315)	747 (374)					
uaudl = 200 kN/m	818 (409)	1037 (519)							
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 25 kN/m	19 (283)	21 (277)	21 (235)	24 (243)	26 (240)	29 (244)	27 (183)	27 (154)	28 (135)
uaudl = 50 kN/m	23 (267)	23 (235)	26 (235)	28 (207)	27 (159)	28 (135)	29 (116)	30 (100)	
uaudl = 100 kN/m	26 (243)	29 (215)	28 (155)	29 (125)					
uaudl = 200 kN/m	29 (178)	28 (110)							
DESIGN NOTES							*See Section 3.2.4 on p 47*		
uaudl = 25 kN/m	ac	ac	ab	ab	ab	ac	ad	ad	ad
uaudl = 50 kN/m	acd	K ac	K ac	abd	ad	ad	ad	ad	
uaudl = 100 kN/m	K ac	abd	ad	ad					
uaudl = 200 kN/m	ad	d							

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl, mm

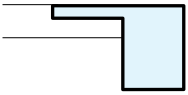
	4.0	5.0	6.0	7.0	8.0	9.0
2 hours fire	+5 mm					
4 hours fire	328	392	498	630	788	980
Moderate exposure	302	380	488	618	776	968
Severe exposure (C40)	302	380	484	614	774	964

Inverted 'L' beams

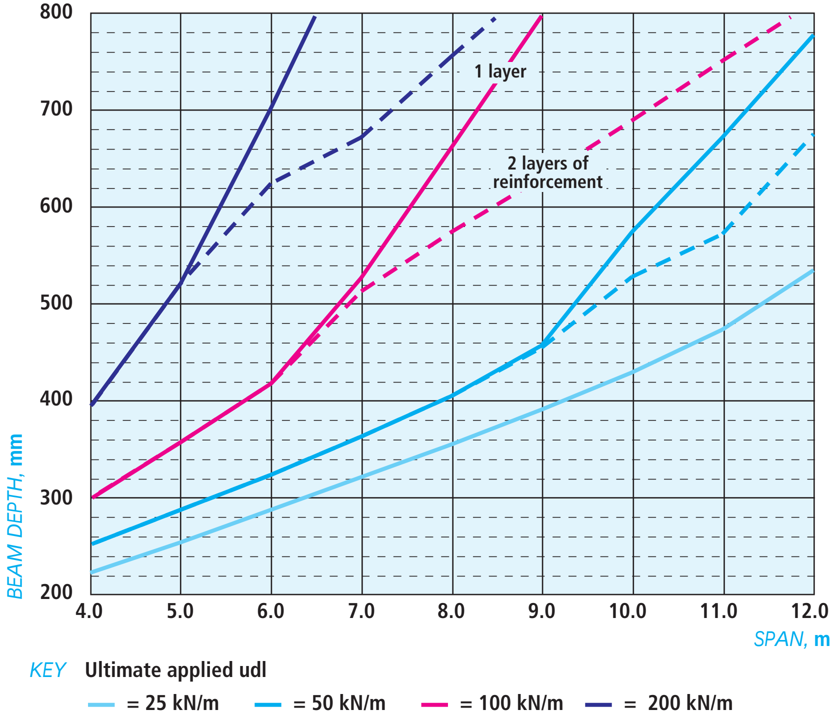
450 mm

wide web

multiple span



SPAN:DEPTH CHART



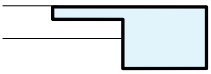
MULTIPLE SPAN, m	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**
DEPTH, mm									
uaudl = 25 kN/m	220	252	286	320	352	390	428	470	534
uaudl = 50 kN/m	236	286	324	362	404	456	574	672	778
uaudl = 100 kN/m	298	356	416	526	662	798	960		
uaudl = 200 kN/m	392	520	702	898					
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	107 (54)	136 (68)	165 (83)	198 (99)	230 (115)	264 (132)	296 (148)	337 (168)	379 (189)
uaudl = 50 kN/m	208 (104)	264 (132)	320 (160)	378 (189)	437 (218)	498 (249)	572 (286)	645 (323)	723 (362)
uaudl = 100 kN/m	412 (206)	519 (260)	629 (314)	745 (373)	868 (434)	995 (497)	1130 (565)		
uaudl = 200 kN/m	818 (409)	1032 (516)	1255 (627)	1484 (742)					
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 25 kN/m	20 (206)	22 (196)	28 (231)	26 (182)	30 (188)	32 (181)	38 (210)	38 (181)	39 (163)
uaudl = 50 kN/m	29 (276)	29 (225)	33 (229)	37 (229)	44 (242)	47 (231)	46 (176)	46 (153)	48 (136)
uaudl = 100 kN/m	34 (252)	38 (237)	47 (250)	47 (201)	47 (158)	49 (136)	50 (116)		
uaudl = 200 kN/m	46 (263)	50 (215)	49 (154)	51 (125)					
DESIGN NOTES *See Section 3.2.4 on p 47*									
uaudl = 25 kN/m	a	a	ab				ab	a	a
uaudl = 50 kN/m	ac	ac	ac	ac	ac	ac	ad	ad	d
uaudl = 100 kN/m	K ac	K ac	acd	abd	d	d	d		
uaudl = 200 kN/m	K ac	bd	d	d					
VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	+5 mm								
4 hours fire	280	312	344	398	486	582	686	810	
Moderate exposure	268	302	334	386	478	574	680	804	
Severe exposure (C40)	272	306	340	392	478	568	674	796	

Inverted ‘L’ beams

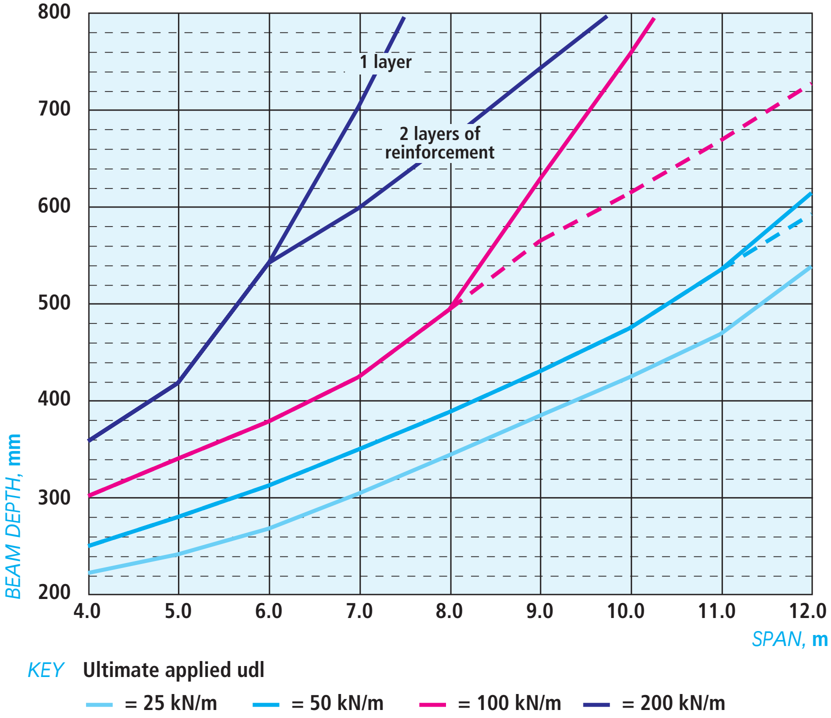
600 mm

wide web

multiple span



SPAN:DEPTH CHART



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	224	244	268	304	346	386	424	470	540
uaudl = 50 kN/m	252	282	310	352	390	432	474	536	616
uaudl = 100 kN/m	304	342	380	426	496	630	758	898	
uaudl = 200 kN/m	360	420	544	704	888				
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	108 (54)	140 (70)	170 (85)	204 (102)	241 (120)	277 (138)	311 (156)	354 (177)	406 (203)
uaudl = 50 kN/m	212 (106)	268 (134)	325 (162)	386 (193)	447 (223)	510 (255)	575 (287)	647 (323)	725 (362)
uaudl = 100 kN/m	415 (207)	524 (262)	633 (316)	745 (373)	864 (432)	996 (498)	1130 (565)	1277 (638)	
uaudl = 200 kN/m	820 (410)	1032 (516)	1254 (627)	1485 (743)	1727 (864)				
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 25 kN/m	31 (256)	25 (163)	31 (195)	32 (174)	30 (142)	33 (141)	39 (161)	41 (150)	39 (122)
uaudl = 50 kN/m	28 (183)	34 (200)	39 (210)	40 (189)	47 (203)	50 (193)	55 (196)	58 (181)	59 (159)
uaudl = 100 kN/m	37 (215)	42 (202)	52 (233)	58 (231)	62 (208)	61 (160)	63 (141)	63 (116)	
uaudl = 200 kN/m	53 (251)	61 (242)	63 (194)	63 (149)	65 (122)				
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 25 kN/m			a						
uaudl = 50 kN/m	a	a	ab	b	b	b	ab	ab	
uaudl = 100 kN/m	ab	b	K ac	K ac	bd		d	d	
uaudl = 200 kN/m	cd	K ac	bd		d				

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

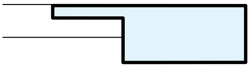
2 hours fire	+5 mm
4 hours fire	+25 mm
Moderate exposure	+20 mm
Severe exposure (C40)	+25 mm

Inverted 'L' beams

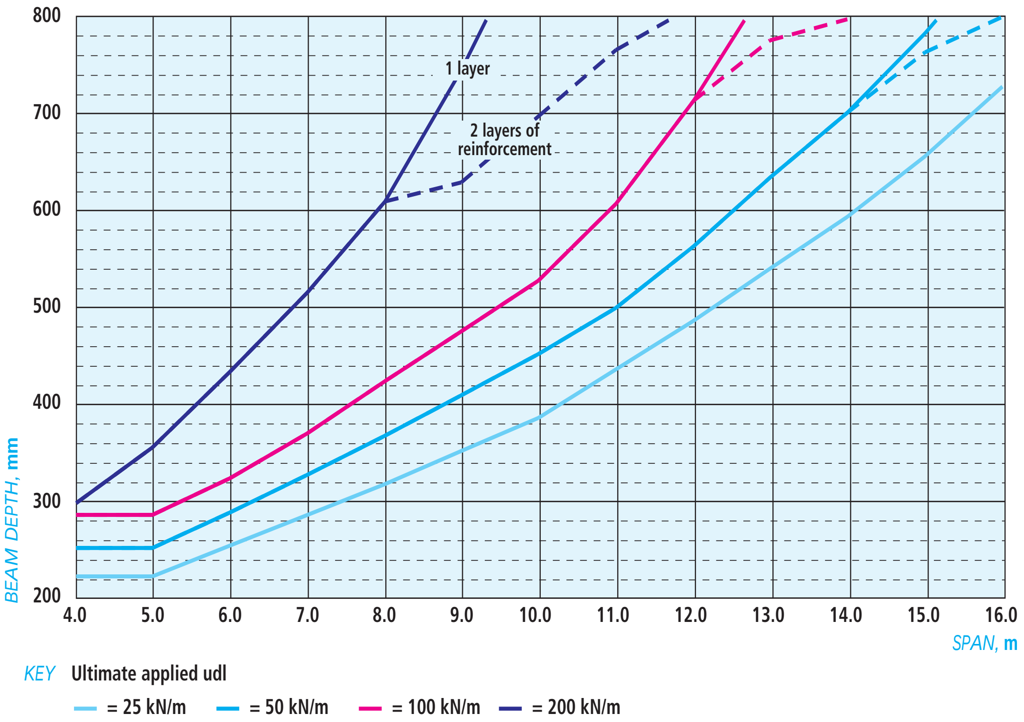
900 mm

wide web

multiple span



SPAN:DEPTH CHART



MULTIPLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
DEPTH, mm									
uaudl = 25 kN/m	254	286	318	352	386	436	486	594	728
uaudl = 50 kN/m	288	328	368	410	452	500	564	702	876
uaudl = 100 kN/m	324	370	424	476	530	608	714	982	
uaudl = 200 kN/m	434	516	610	746					
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	180 (90)	214 (107)	257 (129)	294 (147)	336 (168)	392 (196)	440 (220)	559 (280)	704 (352)
uaudl = 50 kN/m	334 (167)	400 (200)	464 (232)	537 (269)	606 (303)	683 (342)	768 (384)	955 (477)	1175 (588)
uaudl = 100 kN/m	641 (320)	757 (379)	878 (439)	998 (499)	1124 (562)	1269 (634)	1423 (711)	1773 (887)	
uaudl = 200 kN/m	1261 (630)	1479 (740)	1723 (862)	1976 (988)					
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 25 kN/m	38 (160)	42 (162)	39 (130)	46 (146)	50 (145)	50 (122)	59 (135)	70 (130)	79 (120)
uaudl = 50 kN/m	50 (191)	50 (166)	56 (170)	56 (149)	62 (153)	69 (153)	73 (143)	82 (130)	88 (112)
uaudl = 100 kN/m	67 (231)	71 (214)	76 (198)	89 (215)	95 (208)	96 (176)	98 (152)	100 (113)	
uaudl = 200 kN/m	81 (207)	100 (235)	101 (183)	100 (150)					
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 25 kN/m									
uaudl = 50 kN/m									d
uaudl = 100 kN/m	b	b	ab	b	b	bd	d	d	
uaudl = 200 kN/m	ab	K c	bd	d					

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

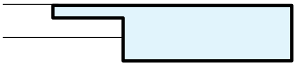
2 hours fire	+5 mm
4 hours fire	+25 mm
Moderate exposure	+20 mm
Severe exposure (C40)	+20 mm

Inverted ‘L’ beams

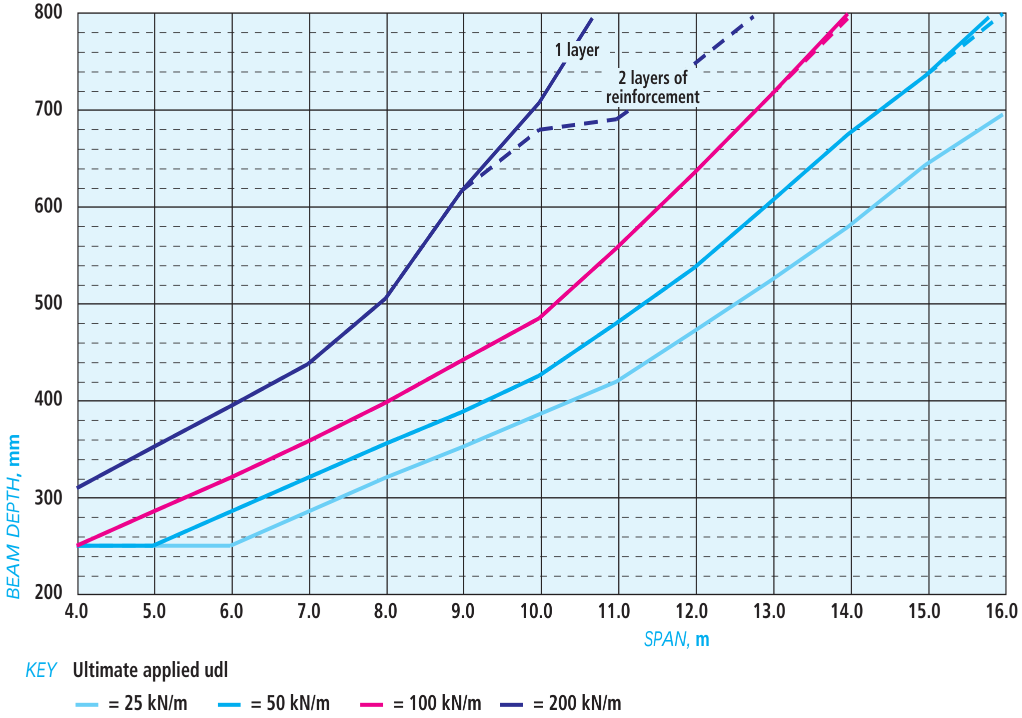
1200 mm

wide web

multiple span



SPAN:DEPTH CHART



MULTIPLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	14.0	16.0
DEPTH, mm									
uaudl = 25 kN/m	250	286	320	352	386	420	472	580	696
uaudl = 50 kN/m	286	320	356	388	426	480	538	676	810
uaudl = 100 kN/m	320	358	398	442	486	558	636	800	
uaudl = 200 kN/m	394	438	506	618	708	836	988		
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 25 kN/m	187 (93)	227 (114)	271 (135)	316 (158)	365 (183)	417 (208)	487 (243)	621 (310)	784 (392)
uaudl = 50 kN/m	345 (172)	412 (206)	483 (241)	555 (277)	629 (315)	723 (361)	812 (406)	1025 (513)	1258 (629)
uaudl = 100 kN/m	653 (327)	772 (386)	898 (449)	1024 (512)	1156 (578)	1303 (652)	1459 (730)	1795 (898)	
uaudl = 200 kN/m	1266 (633)	1494 (747)	1731 (865)	1988 (994)	2245 (1123)	2526 (1263)	2830 (1415)		
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 25 kN/m	49 (161)	47 (138)	49 (127)	50 (119)	54 (117)	63 (125)	63 (108)	81 (116)	98 (117)
uaudl = 50 kN/m	55 (159)	59 (154)	62 (145)	69 (148)	76 (150)	75 (127)	84 (130)	95 (118)	115 (118)
uaudl = 100 kN/m	74 (194)	81 (190)	86 (178)	98 (185)	106 (182)	109 (163)	113 (147)	123 (128)	
uaudl = 200 kN/m	104 (231)	115 (223)	119 (196)	121 (163)	134 (158)	137 (136)	137 (116)		
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 25 kN/m									
uaudl = 50 kN/m									
uaudl = 100 kN/m	b	b	b	b	b				
uaudl = 200 kN/m	b	ac	b		d	d	d		

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 50 kN/m uaudl

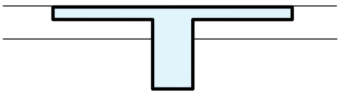
2 hours fire	+5 mm
4 hours fire	+25 mm
Moderate exposure	+20 mm
Severe exposure (C40)	+20 mm

'T' beams

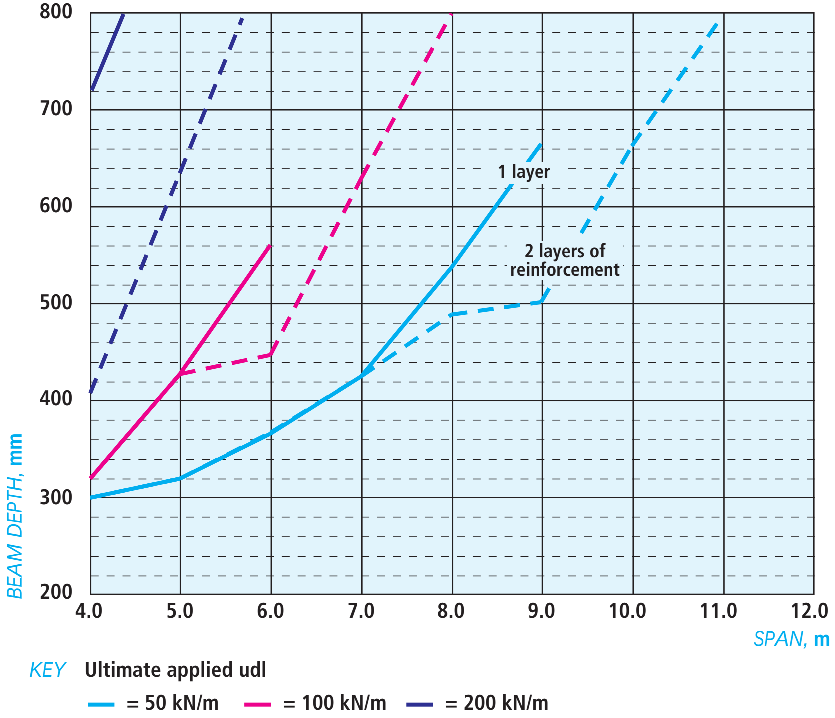
300 mm

wide web

single span



SPAN:DEPTH CHART



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	300	320	368	426	538	666			
uaudl = 100 kN/m	320	428	562						
uaudl = 200 kN/m	724								
uaudl = 400 kN/m									
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	n/a (102)	n/a (128)	n/a (155)	n/a (183)	n/a (214)	n/a (246)			
uaudl = 100 kN/m	n/a (202)	n/a (256)	n/a (311)						
uaudl = 200 kN/m	n/a (411)								
uaudl = 400 kN/m									
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	18 (197)	22 (225)	24 (219)	26 (207)	27 (165)	27 (134)			
uaudl = 100 kN/m	26 (268)	27 (207)	28 (163)						
uaudl = 200 kN/m	21 (96)								
uaudl = 400 kN/m									
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 50 kN/m	a	a	a	ad	ad	Dad			
uaudl = 100 kN/m	a	ad	ad						
uaudl = 200 kN/m	d								
uaudl = 400 kN/m									

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl, mm

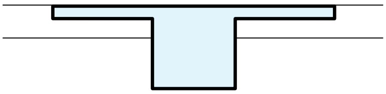
	4.0	5.0	6.0	7.0	8.0
2 hours fire	+5 mm up to 10 m only				
4 hours fire	456	648	968	680	874
Moderate exposure	424	600	856	642	836
Severe exposure (C40)	428	620	858	648	842

‘T’ beams

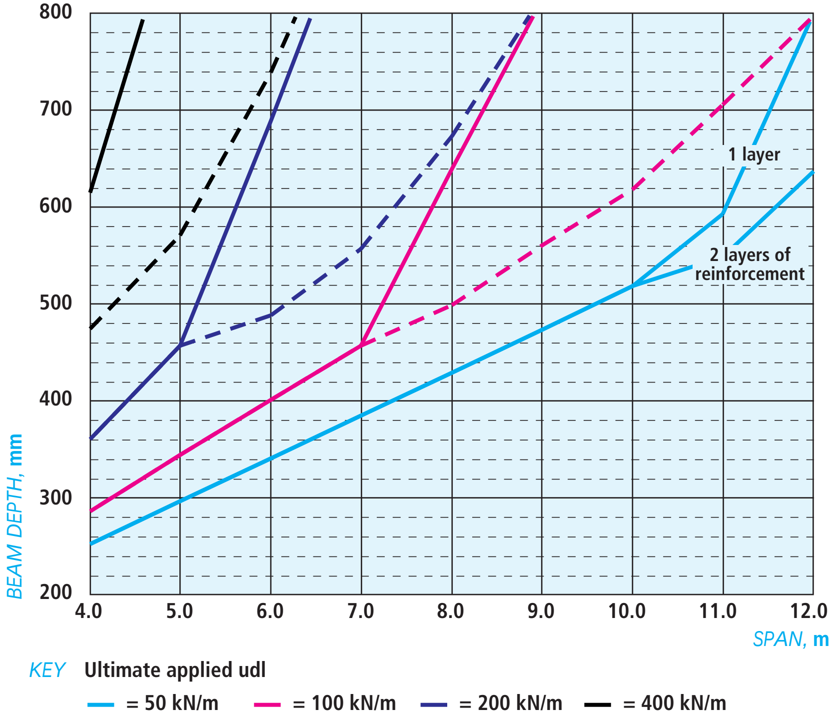
600 mm

wide web

single span



SPAN:DEPTH CHART



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	254	298	342	386	430	474	520	594	800
uaudl = 100 kN/m	288	346	402	458	640	814			
uaudl = 200 kN/m	362	458	690	932					
uaudl = 400 kN/m	616	920							
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (end), kN ult									
uaudl = 50 kN/m	n/a (102)	n/a (130)	n/a (159)	n/a (188)	n/a (218)	n/a (249)	n/a (282)	n/a (319)	n/a (373)
uaudl = 100 kN/m	n/a (204)	n/a (257)	n/a (313)	n/a (368)	n/a (435)	n/a (506)			
uaudl = 200 kN/m	n/a (407)	n/a (513)	n/a (630)	n/a (752)					
uaudl = 400 kN/m	n/a (817)	n/a (1036)							
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	31 (205)	33 (187)	34 (164)	36 (157)	40 (156)	44 (158)	47 (151)	48 (134)	46 (95)
uaudl = 100 kN/m	38 (221)	40 (198)	44 (182)	48 (174)	45 (117)	46 (95)			
uaudl = 200 kN/m	45 (208)	49 (179)	46 (110)	47 (84)					
uaudl = 400 kN/m	45 (122)	47 (86)							
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 50 kN/m	a				a	a		d	d
uaudl = 100 kN/m	a	a	a		d	d			
uaudl = 200 kN/m	ad	d	d	d					
uaudl = 400 kN/m	d	d							

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

2 hours fire	+5 mm up to 10 m only
4 hours fire	+40 mm
Moderate exposure	+20 mm
Severe exposure (C40)	+30 mm

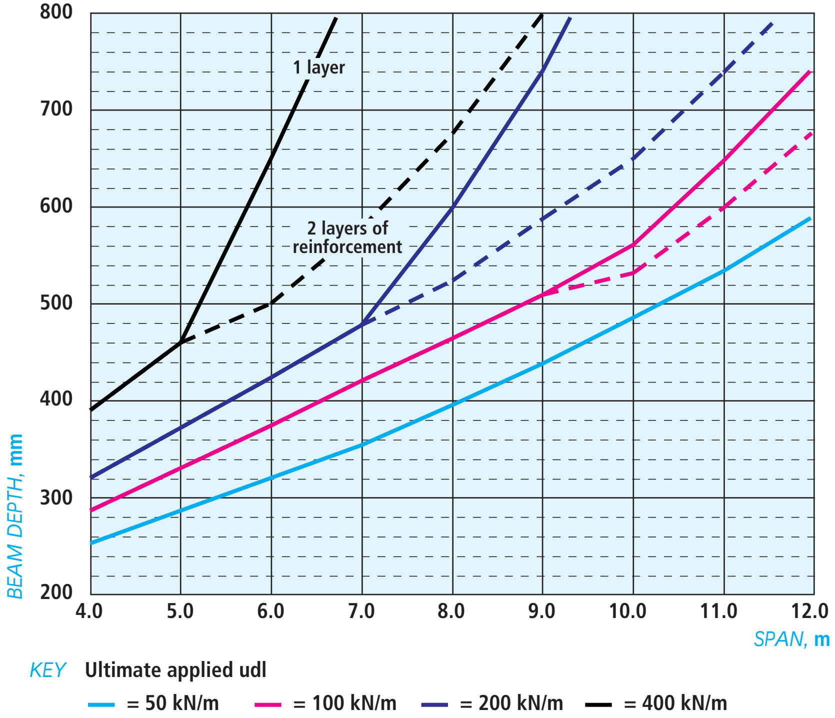
‘T’ beams

1200 mm

wide web

single span

SPAN:DEPTH CHART



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	252	286	320	354	392	438	486	534	592
uaudl = 100 kN/m	286	330	374	424	464	508	560	648	746
uaudl = 200 kN/m	320	372	424	478	600	742	922		
uaudl = 400 kN/m	390	460	650	852					
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	n/a (104)	n/a (134)	n/a (165)	n/a (197)	n/a (231)	n/a (268)	n/a (308)	n/a (344)	n/a (395)
uaudl = 100 kN/m	n/a (207)	n/a (262)	n/a (320)	n/a (381)	n/a (443)	n/a (504)	n/a (573)	n/a (649)	n/a (732)
uaudl = 200 kN/m	n/a (408)	n/a (517)	n/a (627)	n/a (739)	n/a (865)	n/a (998)	n/a (1146)		
uaudl = 400 kN/m	n/a (815)	n/a (1026)	n/a (1254)	n/a (1492)					
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	54 (179)	51 (149)	54 (141)	56 (131)	61 (130)	64 (122)	66 (112)	88 (144)	87 (123)
uaudl = 100 kN/m	56 (162)	61 (160)	63 (143)	66 (131)	73 (131)	85 (142)	88 (131)	91 (117)	96 (107)
uaudl = 200 kN/m	77 (219)	80 (189)	85 (167)	95 (166)	96 (134)	98 (111)	101 (91)		
uaudl = 400 kN/m	86 (183)	101 (183)	98 (125)	100 (98)					
DESIGN NOTES (*See Section 3.2.4 on p 47*)									
uaudl = 50 kN/m									d
uaudl = 100 kN/m							d	d	d
uaudl = 200 kN/m	a		a		d	d	d		
uaudl = 400 kN/m	a		d	d					

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

2 hours fire	+5 mm
4 hours fire	+35 mm up to 10 m only
Moderate exposure	+20 mm
Severe exposure (C40)	+25 mm up to 10 m only

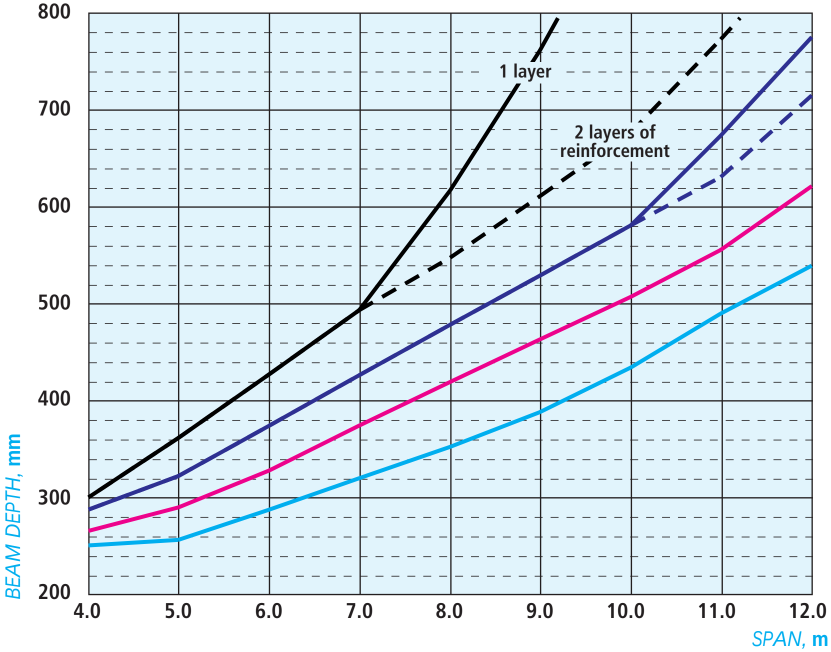
‘T’ beams

2400 mm

wide web

single span

SPAN:DEPTH CHART



KEY **Ultimate applied udl**

— = 50 kN/m **— = 100 kN/m** **— = 200 kN/m** **— = 400 kN/m**

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	256	288	320	352	388	434	490	540
uaudl = 100 kN/m	266	290	328	374	420	464	508	556	622
uaudl = 200 kN/m	288	322	374	426	478	530	582	674	776
uaudl = 400 kN/m	300	362	428	494	618	762	932		
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	n/a (108)	n/a (135)	n/a (171)	n/a (209)	n/a (249)	n/a (293)	n/a (344)	n/a (403)	n/a (453)
uaudl = 100 kN/m	n/a (211)	n/a (268)	n/a (330)	n/a (397)	n/a (471)	n/a (546)	n/a (621)	n/a (702)	n/a (804)
uaudl = 200 kN/m	n/a (414)	n/a (524)	n/a (642)	n/a (764)	n/a (890)	n/a (1012)	n/a (1154)	n/a (1310)	n/a (1479)
uaudl = 400 kN/m	n/a (825)	n/a (1032)	n/a (1256)	n/a (1482)	n/a (1735)	n/a (2004)	n/a (2295)		
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	101 (167)	108 (178)	99 (144)	100 (130)	103 (121)	110 (118)	113 (108)	128 (109)	174 (140)
uaudl = 100 kN/m	108 (169)	105 (152)	109 (140)	111 (126)	112 (111)	123 (110)	140 (117)	174 (134)	180 (120)
uaudl = 200 kN/m	115 (166)	124 (161)	128 (144)	135 (132)	149 (129)	185 (151)	182 (131)	189 (117)	195 (105)
uaudl = 400 kN/m	164 (232)	165 (192)	174 (168)	198 (168)	200 (135)	203 (111)	208 (93)		
DESIGN NOTES								*See Section 3.2.4 on p 47*	
uaudl = 50 kN/m									
uaudl = 100 kN/m								d	d
uaudl = 200 kN/m							d	d	d
uaudl = 400 kN/m			a		d	d	d		

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

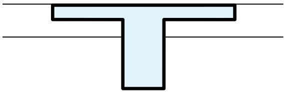
2 hours fire	+5 mm
4 hours fire	+35 mm up to 10 m only
Moderate exposure	+20 mm
Severe exposure (C40)	+25 mm up to 10 m only

'T' beams

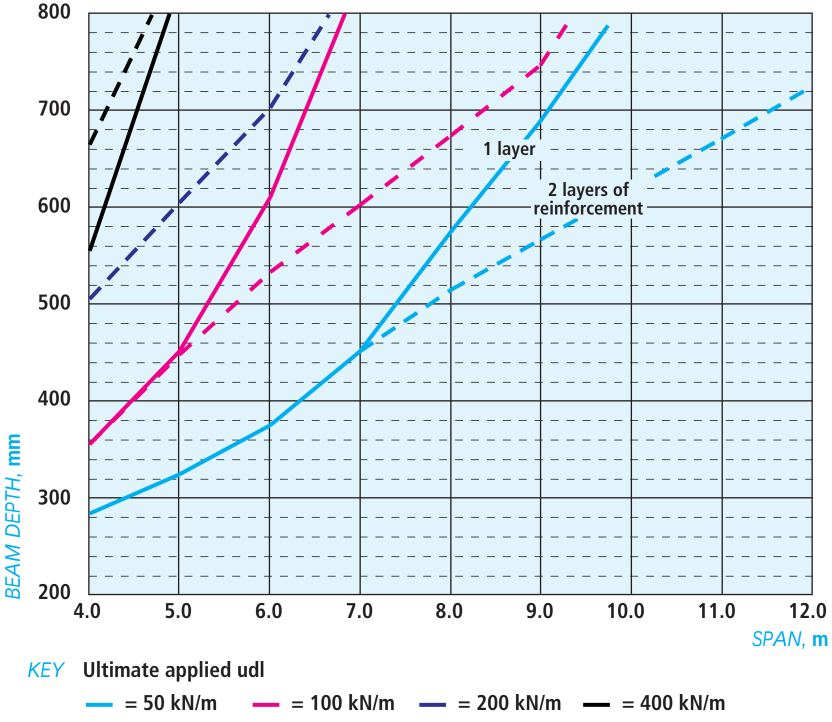
300 mm

wide web

multiple span



SPAN:DEPTH CHART



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	284	324	376	452	576	690	824		
uaudl = 100 kN/m	356	452	612	840					
uaudl = 200 kN/m	556	832							
uaudl = 400 kN/m									
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	203 (102)	256 (128)	311 (155)	368 (184)	430 (215)	494 (247)	563 (281)		
uaudl = 100 kN/m	406 (203)	513 (256)	625 (312)	745 (373)					
uaudl = 200 kN/m	814 (407)	1032 (516)							
uaudl = 400 kN/m									
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	23 (264)	22 (224)	25 (223)	27 (198)	27 (156)	27 (133)	29 (116)		
uaudl = 100 kN/m	24 (229)	28 (205)	28 (152)	27 (107)					
uaudl = 200 kN/m	29 (174)	28 (111)							
uaudl = 400 kN/m									
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 50 kN/m	acd	K c	K ac	abd	ad	ad	ad		
uaudl = 100 kN/m	K ac	abd	ad	d					
uaudl = 200 kN/m	d	d							
uaudl = 400 kN/m									

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl, mm

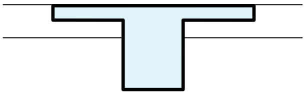
2 hours fire	+0 mm up to 10 m only		
4 hours fire	448	612	820
Moderate exposure	446	610	818
Severe exposure (C40)	442	606	814

‘T’ beams

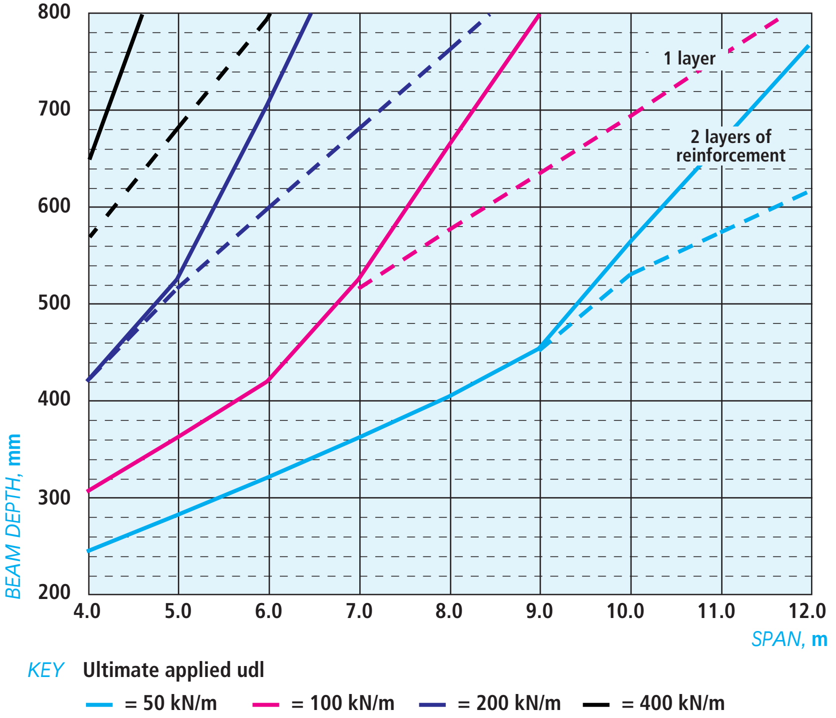
450 mm

wide web

multiple span



SPAN:DEPTH CHART



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	244	282	320	362	404	454	564	666	770
uaudl = 100 kN/m	306	362	420	526	664	798	956		
uaudl = 200 kN/m	420	526	706	902					
uaudl = 400 kN/m	642	898							
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	203 (101)	256 (128)	311 (155)	367 (184)	425 (212)	485 (242)	555 (278)	628 (314)	703 (352)
uaudl = 100 kN/m	406 (203)	512 (256)	620 (310)	735 (367)	856 (428)	981 (491)	1114 (557)		
uaudl = 200 kN/m	813 (407)	1025 (512)	1246 (623)	1474 (737)					
uaudl = 400 kN/m	1627 (813)	2053 (1026)							
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 50 kN/m	28 (253)	30 (236)	32 (222)	36 (221)	44 (239)	46 (225)	46 (179)	46 (153)	47 (136)
uaudl = 100 kN/m	32 (234)	36 (224)	45 (236)	47 (197)	46 (155)	49 (135)	50 (116)		
uaudl = 200 kN/m	40 (213)	49 (207)	48 (151)	50 (124)					
uaudl = 400 kN/m	50 (174)	52 (129)							
DESIGN NOTES							*See Section 3.2.4 on p 47*		
uaudl = 50 kN/m	acd	acd	K c	K ac	K ac	acd	ad	d	d
uaudl = 100 kN/m	K ac	K ac	K ac	bd	d	d	d		
uaudl = 200 kN/m	ab	bd	d	d					
uaudl = 400 kN/m	d	d							

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl, mm

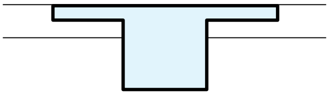
	4.0	5.0	6.0	7.0	8.0
2 hours fire	+0 mm up to 10 m only				
4 hours fire	328	394	506	630	780
Moderate exposure	320	388	510	634	782
Severe exposure (C40)	328	392	502	628	776

‘T’ beams

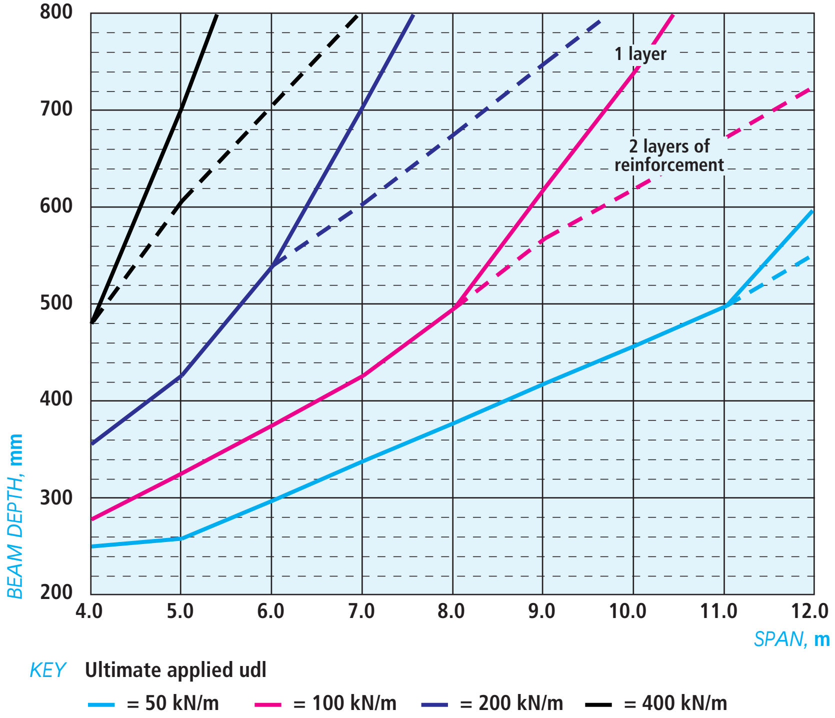
600 mm

wide web

multiple span



SPAN:DEPTH CHART



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	252	258	298	340	376	418	456	498	602
uaudl = 100 kN/m	278	328	376	426	496	618	738	880	
uaudl = 200 kN/m	356	426	540	704	876				
uaudl = 400 kN/m	480	702	952						
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	204 (102)	256 (128)	312 (156)	371 (186)	428 (214)	488 (244)	551 (275)	616 (308)	697 (349)
uaudl = 100 kN/m	406 (203)	515 (257)	621 (311)	732 (366)	848 (424)	978 (489)	1109 (555)	1251 (625)	
uaudl = 200 kN/m	813 (406)	1023 (511)	1241 (621)	1471 (736)	1709 (855)				
uaudl = 400 kN/m	1623 (811)	2051 (1025)	2491 (1245)						
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	27 (179)	35 (227)	38 (212)	44 (215)	48 (211)	51 (207)	54 (199)	60 (201)	58 (161)
uaudl = 100 kN/m	39 (234)	47 (242)	50 (220)	56 (217)	60 (202)	60 (162)	62 (138)	63 (119)	
uaudl = 200 kN/m	51 (240)	59 (229)	62 (192)	64 (151)	65 (124)				
uaudl = 400 kN/m	68 (235)	66 (157)	67 (117)						
DESIGN NOTES									
uaudl = 50 kN/m		ac	c	b	bd	c	ac	b	d
uaudl = 100 kN/m	acd	b	K c	K ac	cd	cd	d	d	
uaudl = 200 kN/m	K c	K ac	bd	d	d				
uaudl = 400 kN/m	cd	d	d						

See Section 3.2.4 on p 47

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

2 hours fire	+5 mm
4 hours fire	+25 mm
Moderate exposure	+20 mm
Severe exposure (C40)	+25 mm

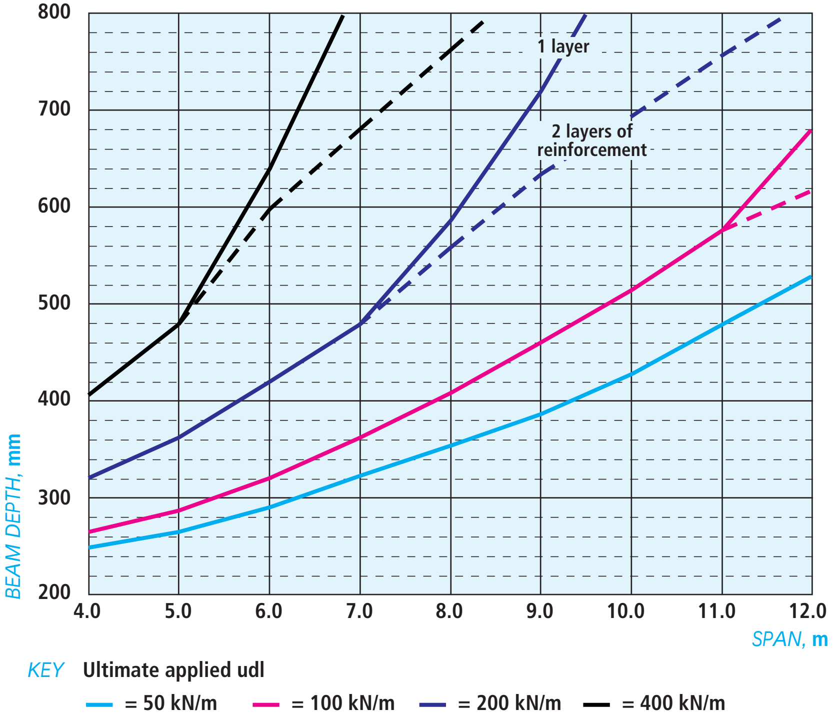
'T' beams

900 mm

wide web

multiple span

SPAN:DEPTH CHART



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	248	264	290	322	354	386	428	478	530
uaudl = 100 kN/m	264	286	320	362	408	460	514	576	682
uaudl = 200 kN/m	320	362	420	478	586	720	882		
uaudl = 400 kN/m	406	478	640	834					
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	206 (103)	260 (130)	316 (158)	376 (188)	437 (219)	501 (250)	570 (285)	646 (323)	716 (358)
uaudl = 100 kN/m	408 (204)	513 (257)	622 (311)	734 (367)	850 (425)	971 (485)	1094 (547)	1225 (613)	1375 (687)
uaudl = 200 kN/m	815 (407)	1024 (512)	1240 (620)	1459 (729)	1693 (847)	1942 (971)	2206 (1103)		
uaudl = 400 kN/m	1625 (812)	2042 (1021)	2480 (1240)	2934 (1467)					
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 50 kN/m	42 (186)	42 (179)	48 (185)	52 (179)	54 (169)	60 (174)	63 (161)	64 (146)	77 (165)
uaudl = 100 kN/m	50 (209)	59 (228)	66 (230)	70 (215)	77 (209)	86 (207)	92 (200)	97 (187)	98 (160)
uaudl = 200 kN/m	62 (215)	72 (222)	82 (218)	96 (224)	100 (190)	102 (158)	102 (129)		
uaudl = 400 kN/m	85 (233)	103 (240)	104 (180)	103 (138)					
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 50 kN/m	d					b			d
uaudl = 100 kN/m	ab	c	K c	K c	c	c	cd	bd	d
uaudl = 200 kN/m	cd	K c	K ac	K c	bd	d	d		
uaudl = 400 kN/m	cd	K c	bd	d					

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

2 hours fire	+5 mm
4 hours fire	+30 mm
Moderate exposure	+30 mm
Severe exposure (C40)	+30 mm

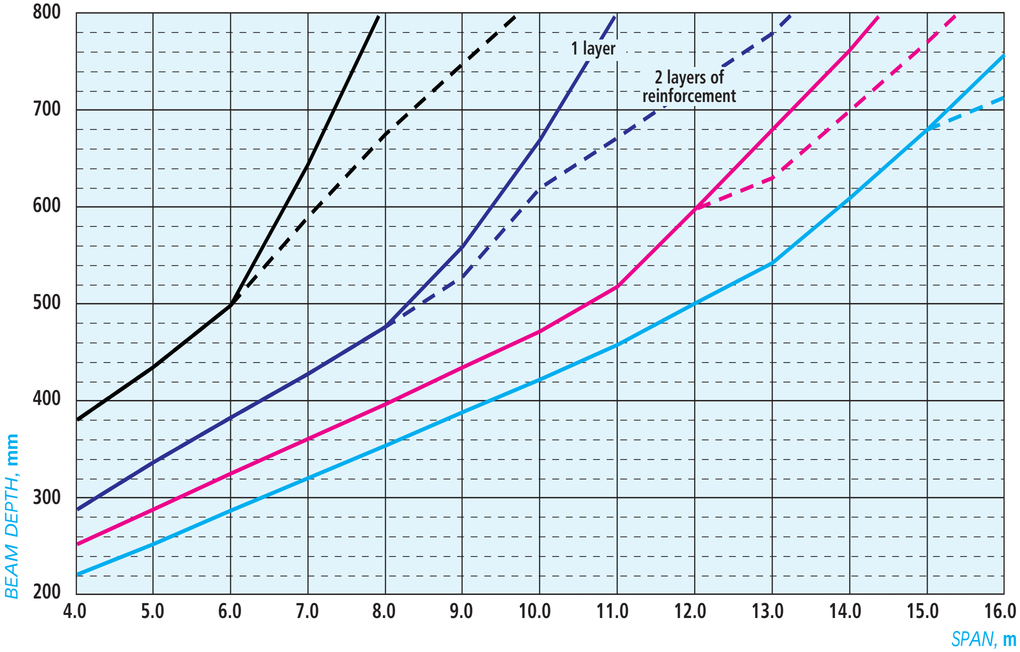
'T' beams

1200 mm

wide web

multiple span

SPAN:DEPTH CHART



KEY **Ultimate applied udl**

▬ = 50 kN/m ▬ = 100 kN/m ▬ = 200 kN/m ▬ = 400 kN/m

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	220	252	286	320	354	388	422	458	500
uaudl = 100 kN/m	252	288	324	360	396	434	472	518	598
uaudl = 200 kN/m	288	336	382	428	476	560	676	802	958
uaudl = 400 kN/m	380	434	498	644	812				
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	203 (102)	260 (130)	321 (160)	384 (192)	449 (225)	514 (257)	589 (294)	664 (332)	745 (373)
uaudl = 100 kN/m	408 (204)	517 (259)	629 (315)	743 (371)	865 (432)	983 (492)	1110 (555)	1241 (621)	1393 (696)
uaudl = 200 kN/m	814 (407)	1028 (514)	1244 (622)	1464 (732)	1689 (845)	1931 (965)	2190 (1095)	2467 (1233)	2767 (1383)
uaudl = 400 kN/m	1629 (815)	2046 (1023)	2472 (1236)	2925 (1463)	3397 (1699)				
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	54 (204)	52 (173)	53 (153)	56 (146)	59 (139)	68 (151)	70 (138)	77 (140)	88 (146)
uaudl = 100 kN/m	59 (195)	64 (187)	72 (188)	78 (185)	84 (174)	96 (187)	104 (184)	117 (188)	118 (164)
uaudl = 200 kN/m	76 (219)	85 (210)	99 (215)	111 (218)	123 (216)	135 (201)	138 (172)	137 (143)	138 (120)
uaudl = 400 kN/m	92 (202)	118 (231)	141 (236)	146 (189)	145 (149)				
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 50 kN/m									
uaudl = 100 kN/m			b	b	b	b	b	b	d
uaudl = 200 kN/m	c	c	c	K ac	K c	bd	d	d	d
uaudl = 400 kN/m	b	K ac	cd	bd	d				

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

2 hours fire	+5 mm
4 hours fire	+25 mm up to 10 m only
Moderate exposure	+20 mm
Severe exposure (C40)	+20 mm up to 10 m only

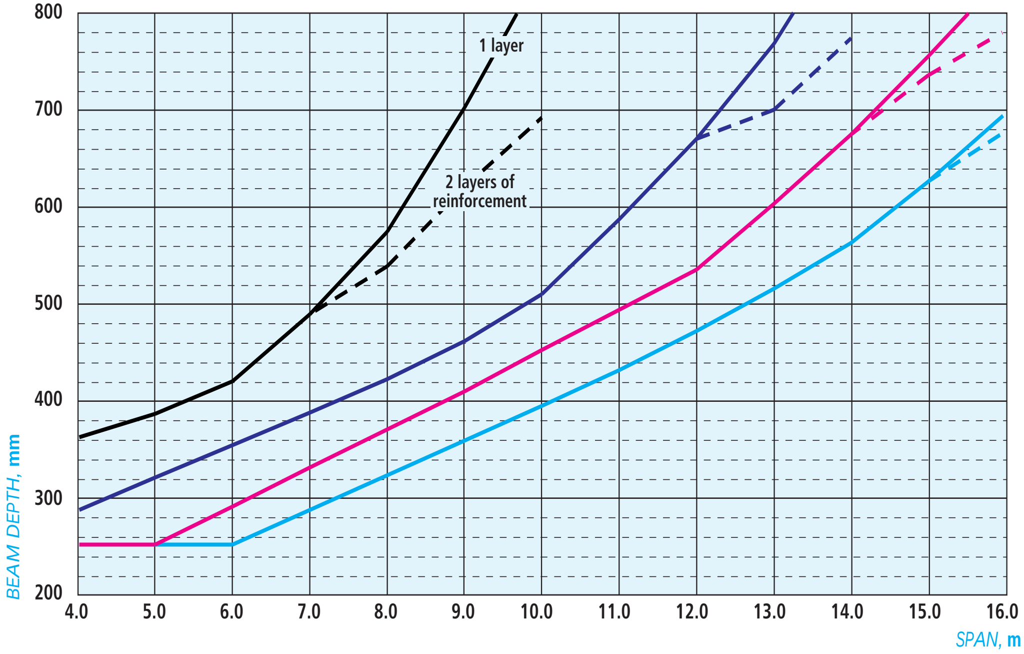
'T' beams

1800 mm

wide web

multiple span

SPAN:DEPTH CHART



KEY **Ultimate applied udl**

▬ = 50 kN/m ▬ = 100 kN/m ▬ = 200 kN/m ▬ = 400 kN/m

MULTIPLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
DEPTH, mm									
uaudl = 50 kN/m	250	286	322	358	394	432	472	564	698
uaudl = 100 kN/m	290	330	370	410	452	494	536	676	844
uaudl = 200 kN/m	354	388	422	462	510	588	672	898	
uaudl = 400 kN/m	420	490	576	704	848				
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	319 (159)	386 (193)	459 (230)	533 (266)	617 (309)	702 (351)	797 (399)	1008 (504)	1282 (641)
uaudl = 100 kN/m	632 (316)	755 (378)	883 (442)	1012 (506)	1152 (576)	1297 (648)	1444 (722)	1803 (902)	2223 (1112)
uaudl = 200 kN/m	1256 (628)	1479 (739)	1706 (853)	1940 (970)	2184 (1092)	2458 (1229)	2743 (1371)	3391 (1696)	
uaudl = 400 kN/m	2480 (1240)	2923 (1461)	3382 (1691)	3874 (1937)	4392 (2196)				
REINFORCEMENT, kg/m (kg/m³)									
uaudl = 50 kN/m	79 (174)	78 (151)	78 (135)	82 (130)	85 (119)	97 (126)	109 (129)	146 (143)	160 (127)
uaudl = 100 kN/m	93 (180)	96 (162)	101 (150)	114 (156)	118 (145)	132 (148)	162 (168)	175 (144)	185 (122)
uaudl = 200 kN/m	116 (183)	132 (190)	153 (203)	169 (205)	189 (208)	194 (183)	201 (166)	205 (127)	
uaudl = 400 kN/m	171 (227)	191 (217)	210 (203)	210 (165)	213 (139)				
DESIGN NOTES								*See Section 3.2.4 on p 47*	
uaudl = 50 kN/m								d	d
uaudl = 100 kN/m							d	d	d
uaudl = 200 kN/m	b	b	b	b	b	bd	d	d	
uaudl = 400 kN/m	K ac	c	bd	d	d				

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

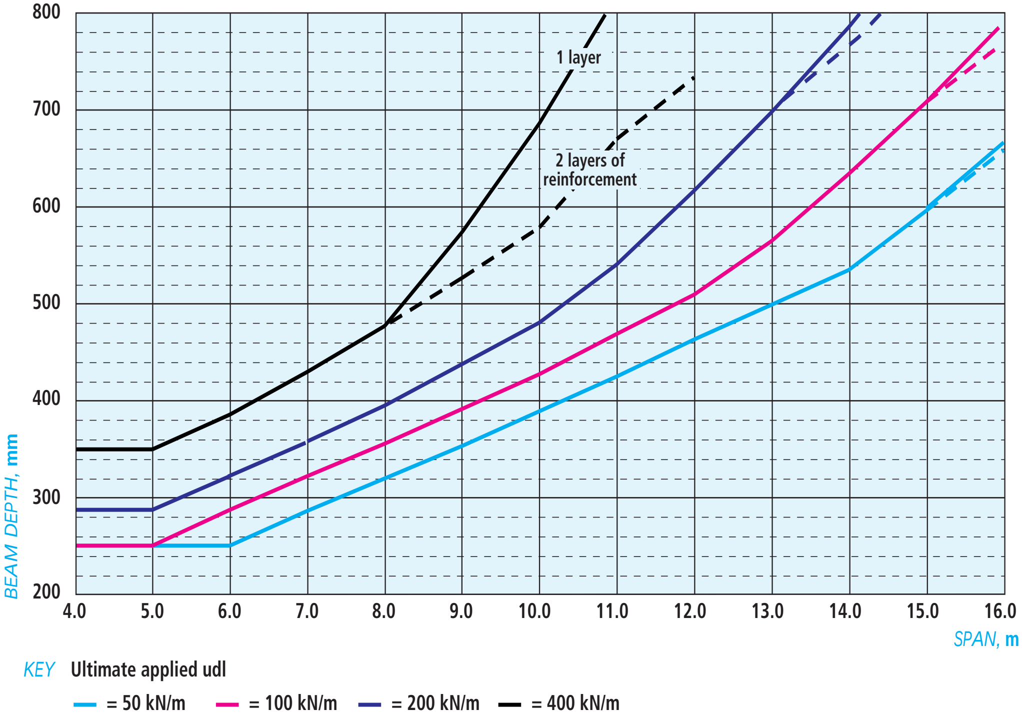
2 hours fire	+15 mm
4 hours fire	+35 mm
Moderate exposure	+25 mm
Severe exposure (C40)	+35 mm

'T' beams *multiple span*

2400 mm

wide web

SPAN:DEPTH CHART



MULTIPLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	14.0	16.0
DEPTH, mm									
uaudl = 50 kN/m	250	286	320	354	390	426	464	536	668
uaudl = 100 kN/m	288	322	356	392	428	470	510	636	792
uaudl = 200 kN/m	322	358	396	438	482	542	618	788	992
uaudl = 400 kN/m	386	430	478	576	688	820	972		
ULTIMATE LOAD TO SUPPORTS/COLUMNS INTERNAL (END), kN ult									
uaudl = 50 kN/m	325 (163)	400 (200)	477 (239)	560 (280)	653 (327)	745 (373)	855 (428)	1079 (540)	1404 (702)
uaudl = 100 kN/m	643 (321)	768 (384)	898 (449)	1039 (520)	1184 (592)	1341 (671)	1500 (750)	1892 (946)	2364 (1182)
uaudl = 200 kN/m	1259 (630)	1486 (743)	1726 (863)	1973 (986)	2227 (1114)	2503 (1252)	2804 (1402)	3464 (1732)	4222 (2111)
uaudl = 400 kN/m	2490 (1245)	2930 (1465)	3379 (1690)	3873 (1936)	4394 (2197)	4950 (2475)	5547 (2774)		
REINFORCEMENT, kg/m (kg/m^3)									
uaudl = 50 kN/m	95 (157)	91 (131)	91 (118)	93 (110)	98 (105)	111 (110)	125 (113)	181 (141)	206 (128)
uaudl = 100 kN/m	107 (154)	112 (146)	119 (140)	127 (135)	136 (132)	161 (142)	189 (154)	213 (139)	231 (122)
uaudl = 200 kN/m	142 (184)	162 (191)	174 (183)	192 (183)	209 (180)	234 (180)	240 (162)	253 (134)	268 (113)
uaudl = 400 kN/m	197 (213)	230 (222)	256 (223)	269 (194)	272 (165)	275 (140)	272 (117)		
DESIGN NOTES									*See Section 3.2.4 on p 47*
uaudl = 50 kN/m					d				d
uaudl = 100 kN/m								d	d
uaudl = 200 kN/m	b	b	b	b	b	bd	d	d	d
uaudl = 400 kN/m	c	ac	c	bd	d	d	d		

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 3.2.3 on p 46): implications on beam depths for 100 kN/m uaudl

2 hours fire	+5 mm
4 hours fire	+25 mm
Moderate exposure	+20 mm
Severe exposure (C40)	+25 mm

3.3 Columns

3.3.1 USING IN-SITU COLUMNS

In-situ columns offer strength, economy, versatility, mouldability, fire resistance, and robustness. They are often the most obvious and intrusive part of a structure and judgement is required to reconcile position, size, shape, spans of horizontal elements and economy. Generally the best economy comes from using regular square grids and constantly sized columns. Ideally, the same size of column should be used at all levels at all locations. If this is not possible, then keep the number of profiles to a minimum, eg. 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 of 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.

High-strength concrete columns can decrease the size of columns required. Smaller columns occupy less lettable space and should be considered on individual projects. However, up to about five storeys the size of perimeter columns is dominated by moment: concrete strengths greater than 35 N/mm^2 appear to make little difference to the size of perimeter column required. Rectangular columns can be less obtrusive than square columns.

3.3.2 USING THE CHARTS AND DATA

The column charts give square sizes against **total ultimate** axial load for a range of steel contents for internal, edge and corner **braced** columns. Further charts and tables allow bar arrangements to be judged and reinforcement densities estimated.

The column charts 'work' on **total ultimate** axial load in kN. The user should preferably calculate, otherwise estimate, this load for the lowest level of column under consideration (see Section 8.3).

Column design is dependant upon ultimate axial load **and** ultimate design moment. Design moments in columns are specific to that column and can only be generalized (but with unknown certainty) by using a fair amount of conservatism. The sizes given, particularly for perimeter columns, are, therefore, **estimates** only. 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 charts and data for internal columns assume nominal moments only: they assume that the slabs and beams supported have equal spans in each orthogonal direction (ie. $l_{x1} = l_{x2}$ and $l_{y1} = l_{y2}$). If spans differ by more than, say 15%, consider treating internal columns as edge columns.

In order to allow for moments, the charts for edge and corner columns give sizes according to axial load and the number of storeys supported. As explained in Section 7, the sizes should, generally, prove conservative, but will not be so if imposed floor loads greater than 5.0 kN/m^2, floor plates less stiff than solid flat slabs or unequal adjacent spans, are required. If spans parallel to the edge are unequal by more than, say 15%, then consider treating edge columns as corner columns.

Sizes derived from the charts and data should be checked for compatibility with slabs (eg. punching shear in flat slabs) and beams (eg. 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 (eg. by providing, effectively, a pin joint), it is suggested that this single storey load case should be checked at scheme design stage.

3.3.3 DESIGN ASSUMPTIONS

Reinforcement
Main bars: f_y = 460 N/mm^2. Links: f_y = 250 N/mm^2. Maximum bar size T40. Link size, maximum main bar size/4. Reinforcement weights assume 35 diameter laps and 3.6 m storey heights and links at 250 mm minimum centres. No allowance is made for wastage. With regard to reinforcement quantities, please refer to Section 2.2.4.

Concrete
C35, 24 kN/m^3, 20 mm aggregate.

Fire and durability
Fire resistance 1 hour; mild exposure.

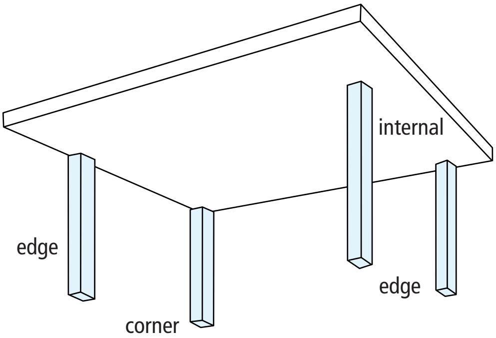
Other assumptions made are described and discussed in Section 7.

3.3.4 DESIGN NOTES

General
As described in Section 7, the charts and data are based on considering square braced columns supporting solid flat slabs, with panel aspect ratios of 1.00, 1.25, 1.5 and 1.75, carrying 5.0 kN/m^2 imposed load and 10 kN/m perimeter load. The charts and data correspond to the worst case, ie. largest size derived from considering the flat slabs described above. 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.

Main bars

Feasible bar arrangements for various square column sizes and reinforcement percentages are given on pages 75 and 80. These graphs have been prepared on the basis of maximum 300 mm centres of bars or minimum 30 mm gap at laps. For perimeter columns it is assumed that in 8 bar arrangements (3 bars per face), 6 bars are effective, and that in 12 bar arrangements (4 bars per face), 8 bars are effective.

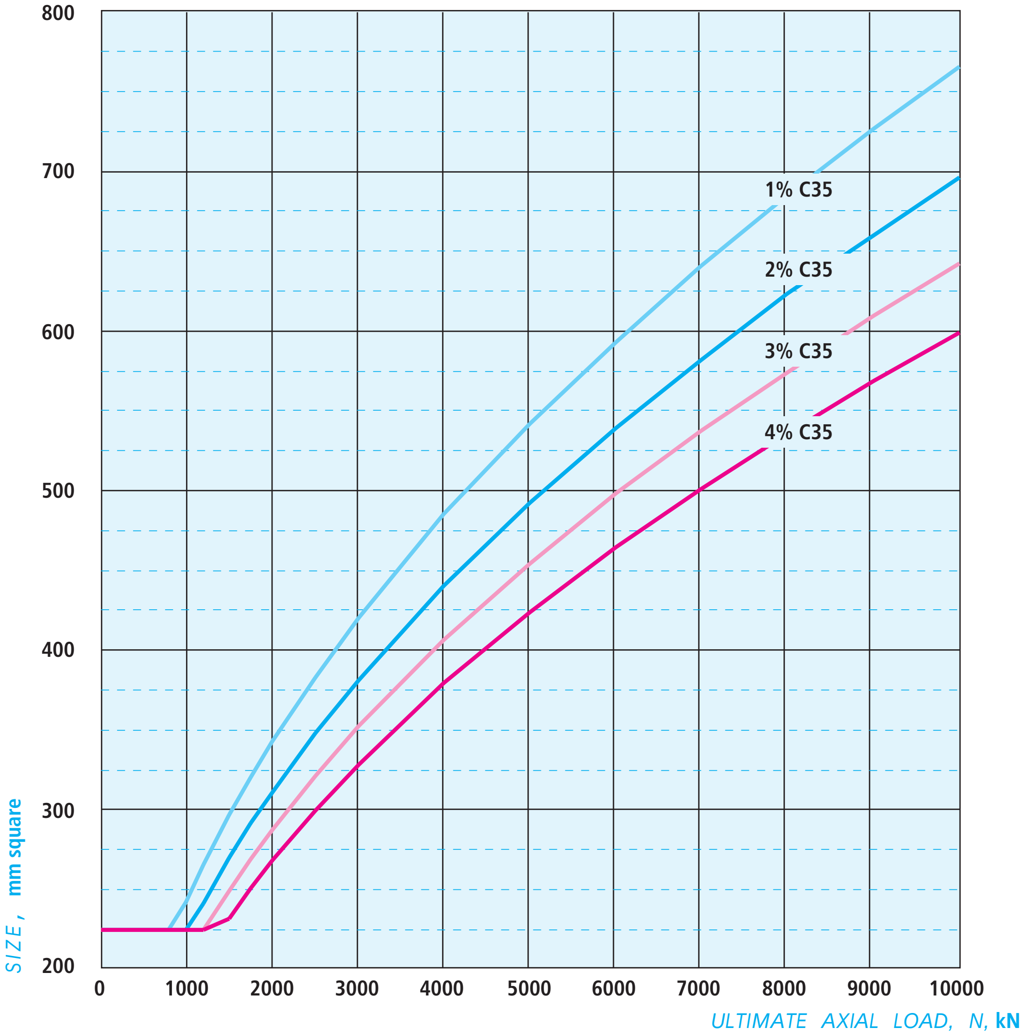


In-situ concrete columns

Size is not only dependent on load but also, especially in perimeter columns, on moment. In order to allow for moments in perimeter columns, the charts for edge and corner columns give sizes according to the number of storeys supported.

Internal columns

LOAD:SIZE CHART

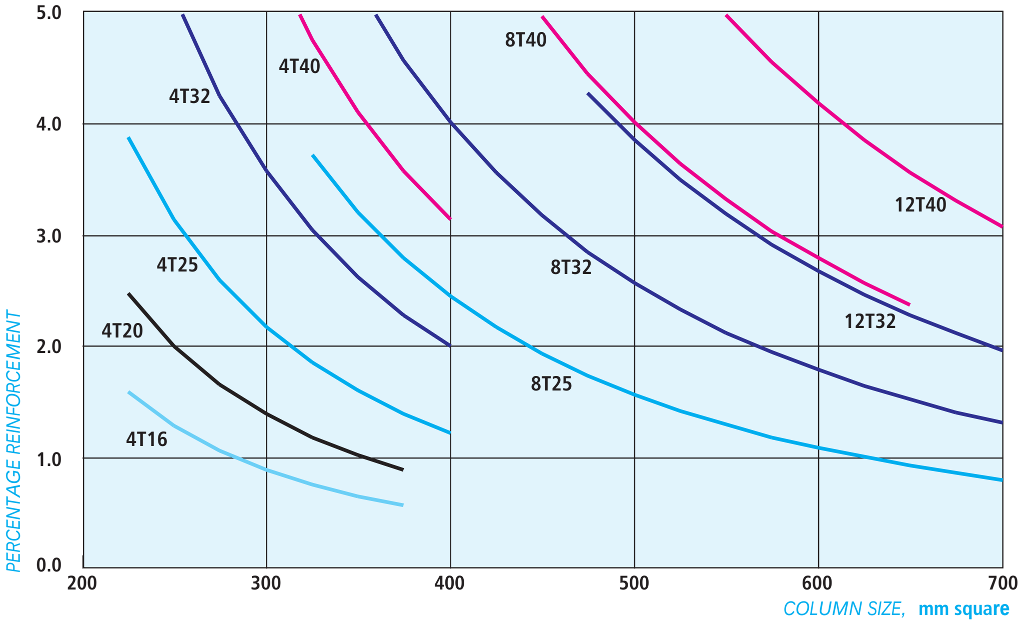


ULTIMATE AXIAL LOAD, kN	1000	1500	2000	3000	4000	5000	6000	8000	10000
SIZE, mm square									
1.0% C35	240	295	345	420	485	540	595	685	765
2.0% C35	225	270	310	380	440	490	540	620	695
3.0% C35	225	250	285	350	405	455	500	570	640
4.0% C35	225	230	270	330	380	425	465	535	595

VARIATIONS: implications of using different grades of concrete

Ultimate axial load									
2.5% C35	225	255	295	365	420	470	515	595	665
2.5% C40	225	245	285	350	405	450	495	570	640
2.5% C50	225	230	265	325	375	420	460	530	595
2.5% C60	225	225	245	305	350	395	430	495	555
2.5% C80	225	225	225	270	315	350	385	445	500

SIZE:PERCENTAGE REINFORCEMENT CHART, INTERNAL COLUMNS



Feasible bar arrangements for internal columns are given above. These are dependant on column sizes and required percentage of reinforcement. The graphs have been prepared on the basis of maximum 300 mm centres or minimum 30 mm gap at laps. All bars are assumed to be effective.

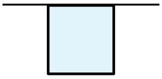
INTERNAL COLUMN SIZE, mm square

REINFORCEMENT, kg/m height (kg/m^3)

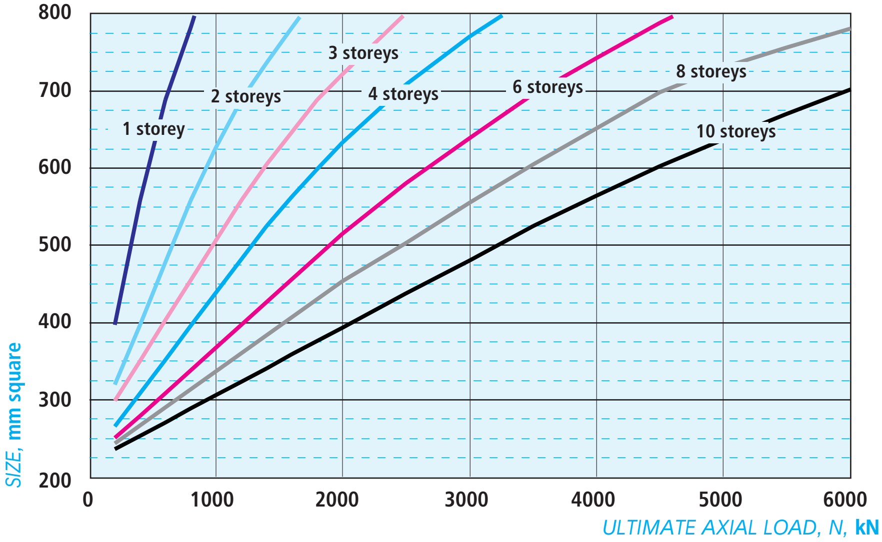
Including laps and links. See Section 2.2.4 on p 6

		250	300	350	400	450	500	550	600	650
4T16s,	**804 mm^2**	9 (152)	10 (111)							
4T20s,	**1256 mm^2**	14 (218)	14 (155)	14 (117)						
4T25s,	**1964 mm^2**	22 (350)	22 (249)	23 (187)	23 (146)					
4T32s,	**3216 mm^2**		37 (406)	37 (303)	38 (235)					
8T25s,	**3928 mm^2**			42 (347)	46 (285)	46 (229)	47 (189)	48 (158)	49 (135)	
4T32s + 4T20s,	**4472 mm^2**			56 (455)	57 (355)	58 (286)	59 (236)	65 (216)	67 (185)	68 (161)
4T32s + 4T25s,	**5180 mm^2**				64 (401)	65 (322)	66 (265)	73 (240)	74 (206)	76 (179)
8T32s,	**6432 mm^2**				74 (463)	75 (369)	76 (302)	76 (252)	77 (214)	78 (184)
12T32s,	**9648 mm^2**					109 (536)	109 (437)	114 (375)	115 (318)	116 (273)
8T40s,	**10048 mm^2**							125 (412)	126 (349)	127 (300)

Edge columns 1% *reinforcement*

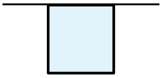


LOAD:SIZE CHART

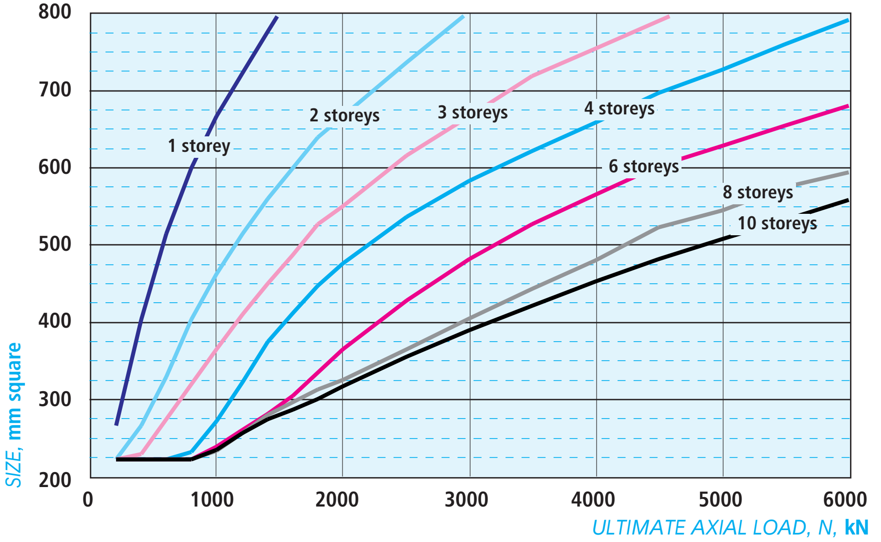


ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	3000	4000	5000	6000
SIZE, mm square									*C35 concrete*
2 storeys	395	560	690	780					
3 storeys	350	455	560	645	720				
4 storeys	310	395	480	565	635	770			
6 storeys	280	340	395	455	515	640	740		

Edge columns 2% *reinforcement*

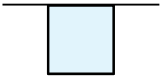


LOAD:SIZE CHART

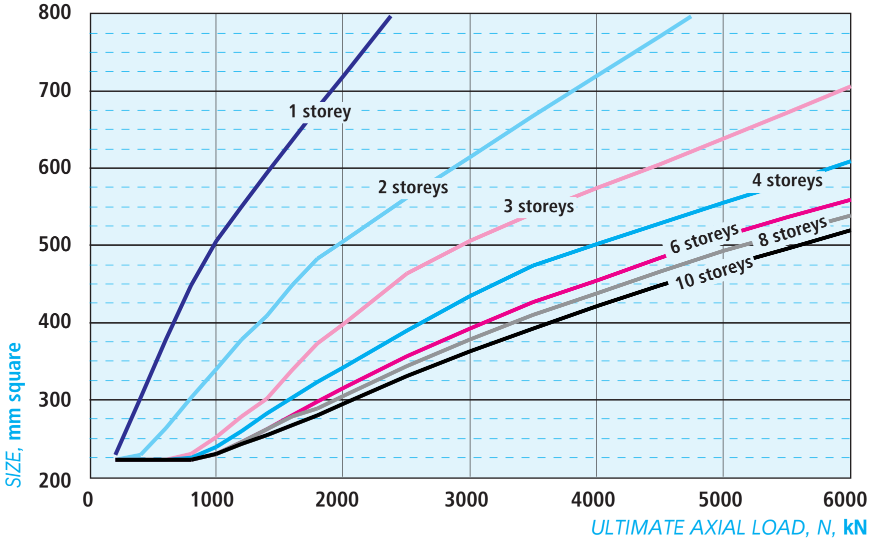


ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	3000	4000	5000	6000
SIZE, mm square									*C35 concrete*
2 storeys	270	405	515	600	670				
3 storeys	230	320	410	490	550	670			
4 storeys	225	235	325	415	480	585	660		
6 storeys	225	225	265	305	365	485	565	630	680

Edge columns 3% *reinforcement*

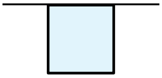


LOAD:SIZE CHART

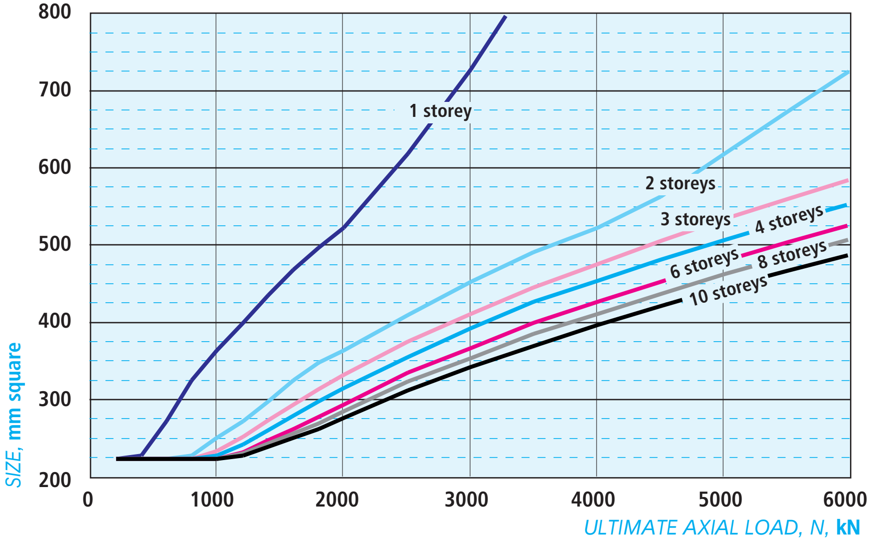


ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	3000	4000	5000	6000
SIZE, mm square									*C35 concrete*
2 storeys	230	305	380	450	505				
3 storeys	225	235	280	340	400	505	575		
4 storeys	225	225	260	305	345	435	505	555	
6 storeys	225	225	250	280	315	395	455	515	560

Edge columns 4% *reinforcement*



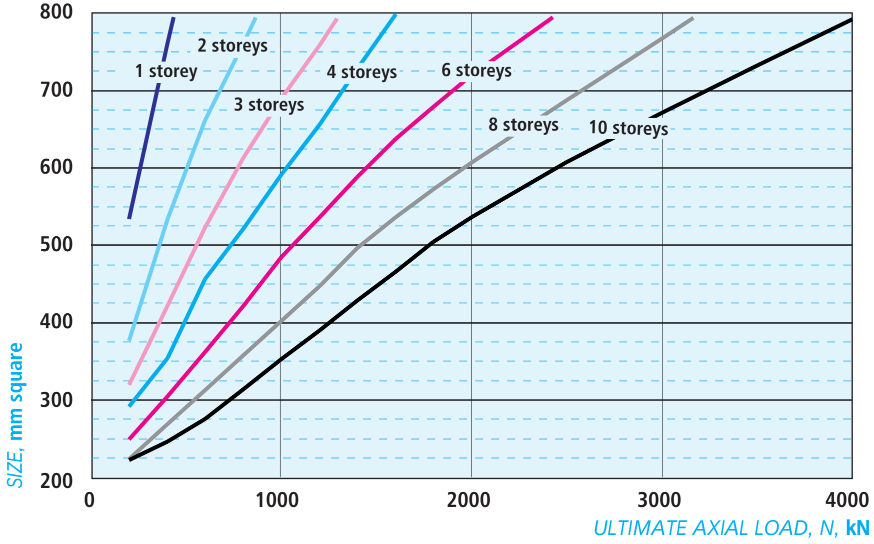
LOAD:SIZE CHART



ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	3000	4000	5000	6000
SIZE, mm square									*C35 concrete*
2 storeys	225	230	275	325	365	455	525		
3 storeys	225	225	255	295	335	410	475	535	
4 storeys	225	225	245	280	315	395	455	505	555
6 storeys	225	225	235	265	295	365	425	480	530

Corner columns 1% *reinforcement*

LOAD:SIZE CHART

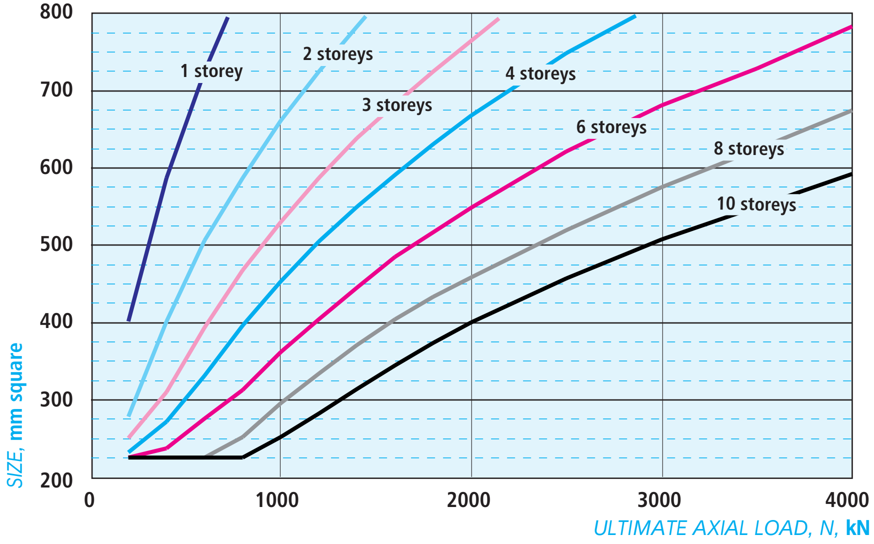


SIZE, mm square *C35 concrete*

ULTIMATE AXIAL LOAD, kN	200	400	600	800	1000	1200	1600	2000	3000
2 storeys	380	535	665	765	865				
3 storeys	320	425	525	615	690	760			
4 storeys	295	355	460	525	595	660	800		
6 storeys	250	305	365	425	485	540	640	720	

Corner columns 2% *reinforcement*

LOAD:SIZE CHART

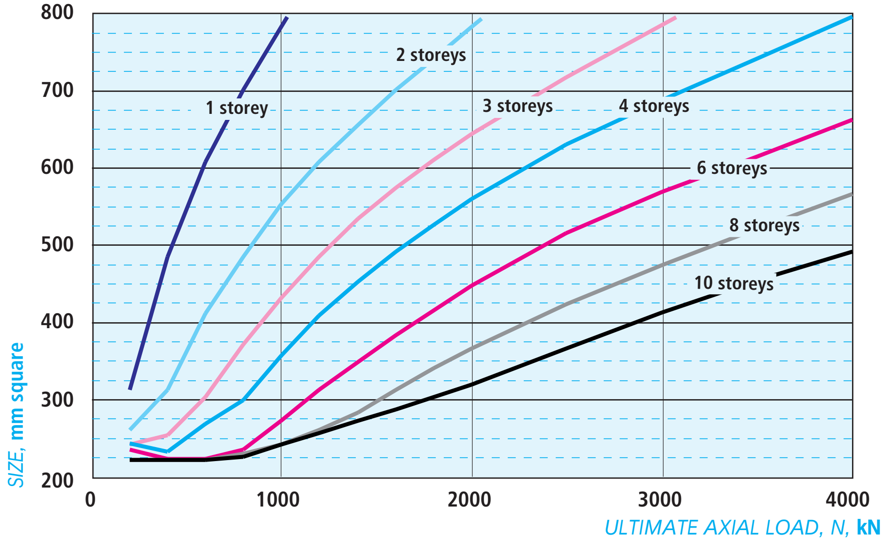


SIZE, mm square *C35 concrete*

ULTIMATE AXIAL LOAD, kN	200	400	600	800	1000	1200	1600	2000	3000
2 storeys	280	400	505	585	660				
3 storeys	250	310	395	465	530	585	683		
4 storeys	230	270	330	395	455	505	590	668	
6 storeys	225	235	275	315	360	405	485	550	682

Corner columns 3% *reinforcement*

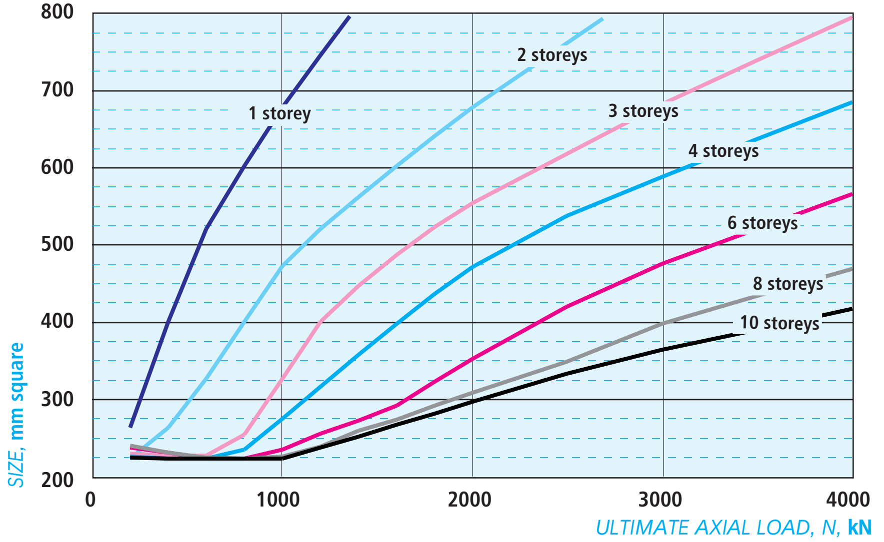
LOAD:SIZE CHART



ULTIMATE AXIAL LOAD, kN	200	400	600	800	1000	1200	1600	2000	3000
SIZE, mm square									*C35 concrete*
2 storeys	265	315	410	485	555				
3 storeys	245	255	305	375	435	485	574		
4 storeys	245	235	270	300	360	410	490	559	
6 storeys	240	225	225	240	275	315	385	450	569

Corner columns 4% *reinforcement*

LOAD:SIZE CHART

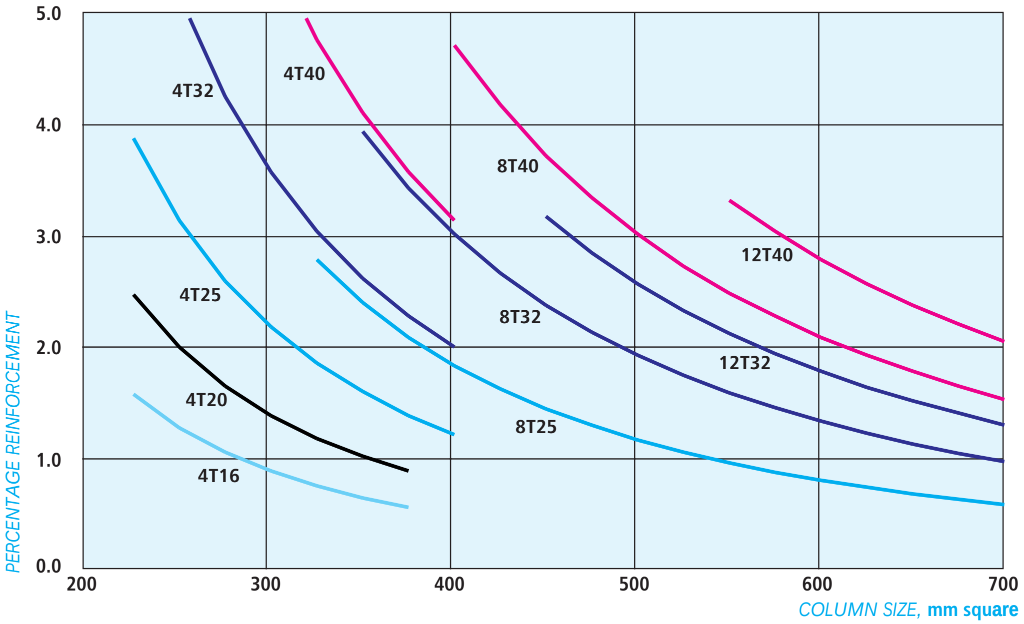


ULTIMATE AXIAL LOAD, kN	200	400	600	800	1000	1200	1600	2000	3000
SIZE, mm square									*C35 concrete*
2 storeys	225	265	330	400	475				
3 storeys	230	230	230	255	330	400	488		
4 storeys	230	225	225	235	275	320	400	472	
6 storeys	240	230	225	225	235	255	295	355	476

Perimeter columns

(Edge and corner columns)

SIZE:PERCENTAGE REINFORCEMENT CHART, PERIMETER COLUMNS



Feasible bar arrangements for perimeter columns are given above. These are dependant on column sizes and required percentage of reinforcement. The graphs assume maximum 300 mm centres or minimum 30 mm gaps at laps. As they are perimeter columns, ie. edge and corner columns, it is assumed that in 8 bar arrangements, 6 bars are effective and in 12 bar arrangements, 8 bars are effective. This makes the above chart slightly different from the one on p 75 which deals with internal columns; but for the same arrangement and size, reinforcement densities are the same for perimeter as internal columns.

PERIMETER COLUMN SIZE, mm square

REINFORCEMENT, kg/m height (kg/m³) — *Including laps and links. See Section 2.2.4 on p 6*

		250	**300**	**350**	**400**	**450**	**500**	**550**	**600**	**650**
4T16s,	**804 mm²**	9 (152)	10 (111)							
4T20s,	**1256 mm²**	14 (218)	14 (155)	14 (117)						
4T25s,	**1964 mm²**	22 (350)	22 (249)	23 (187)	23 (146)					
4T32s,	**3216 mm²**		37 (406)	37 (303)	38 (235)					
8T25s,	**3928 mm²**			42 (347)	46 (285)	46 (229)	47 (189)	48 (158)	49 (135)	
4T32s + 4T20s,	**4472 mm²**			56 (455)	57 (355)	58 (286)	59 (236)	65 (216)	67 (185)	68 (161)
4T32s + 4T25s,	**5180 mm²**				64 (401)	65 (322)	66 (265)	73 (240)	74 (206)	76 (179)
8T32s,	**6432 mm²**				74 (463)	75 (369)	76 (302)	76 (252)	77 (214)	78 (184)
12T32s,	**9648 mm²**					109 (536)	109 (437)	114 (375)	115 (318)	116 (273)
8T40s,	**10048 mm²**							125 (412)	126 (349)	127 (300)

4.1 Slabs

4.1.1 USING PRECAST AND COMPOSITE SLABS

Precast concrete flooring offers many advantages: speed of erection, small, medium and long spans, structural efficiency, economy, versatility, fire resistance, thermal capacity and sound insulation. It will readily accept fixings, floor and ceiling finishes, and small holes. Provision can be made for large holes. Handling and stacking is straightforward. Precast concrete flooring provides immediate safe working platforms and can eliminate formwork and propping.

The combination of precast concrete with in-situ concrete (or hybrid concrete construction(7)) 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 robustness, flexibility and versatility of in-situ construction.

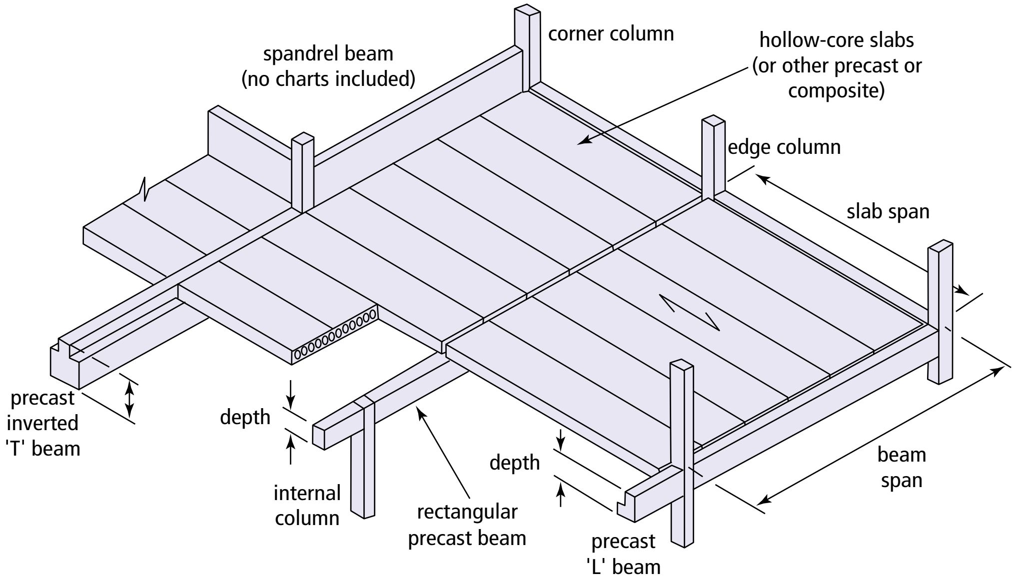
Each type has implications for overall costs, speed, self-weight, storey heights and flexibility in use; some guidance is given with the charts. The relative importance of these factors should be assessed for each particular case.

The units are designed to BS 8110, generally using grade C50 concrete and high tensile strand or wire prestressing steel to BS 5896 or high tensile steel to BS 4449. All prestressed precast concrete flooring systems exhibit a degree of upward camber and due allowance should be made. Minimum bearing of precast members is 40 mm plus allowances for spalling and construction inaccuracies (see BS 8110, Pt. 1, Cl 5.2.3 and Cl 5.2.4).

4.1.2 USING THE CHARTS AND DATA

The charts and data give overall depths against spans for a range of **characteristic** imposed loads assuming simply supported spans. An allowance of 1.5 kN/m^2 has been made for superimposed dead loads (finishes, services, etc). The range of many precast floors is considerably extended if this allowance is reduced.

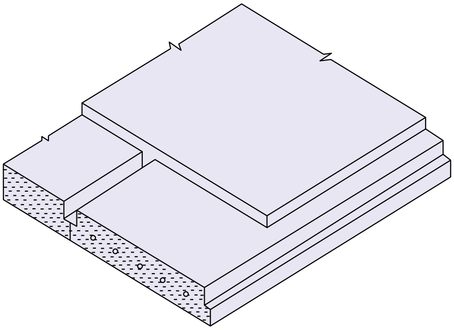
Actual span/load capacities and self-weights vary between manufacturers and are subject to development and change. The user should refer to manufacturers and their current literature. The sizes, spans and weights quoted in the charts and data are selected, whenever possible, from those offered in late 1996 by at least two manufacturers. Thicknesses are measured overall of structural toppings, etc.



Precast concrete construction

The diagram above shows typical components. *See Precast concrete framed structures - Design guide(8) and Multi-storey precast concrete framed structures(9)* for detailed guidance on procurement and design.

Composite solid prestressed soffit slabs



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

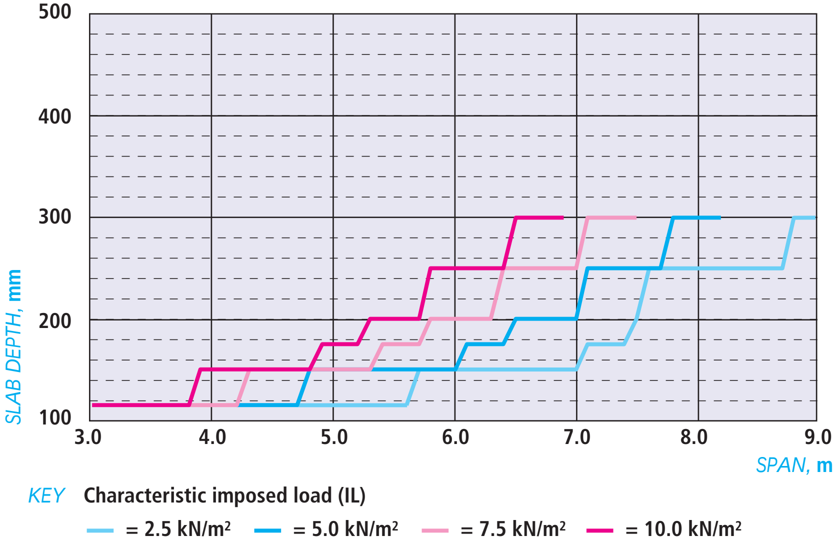
ADVANTAGES

- Speed
- Elimination of formwork
- Structural efficiency
- Robustness

DISADVANTAGES

- Limited spans and capacities
- Propping usually required

SPAN:DEPTH CHART

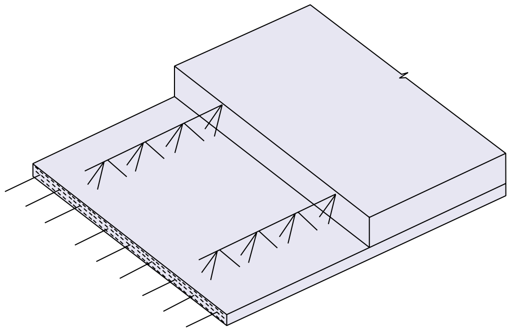


SINGLE SPAN, m	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**
THICKNESS, mm				*Including topping. Propping at 2.0 to 2.7 m maximum centres*					
IL = 2.5 kN/m²	115	115	115	150	150	250	300		
IL = 5.0 kN/m²	115	115	150	150	200	300			
IL = 7.5 kN/m²	115	115	150	200	250				
IL = 10.0 kN/m²	115	150	175	250					
ULTIMATE LOAD TO SUPPORTING BEAMS, internal (end), kN/m									
IL = 2.5 kN/m²	30 (15)	40 (20)	50 (25)	67 (33)	78 (39)	116 (58)	146 (73)		
IL = 5.0 kN/m²	42 (21)	56 (28)	76 (38)	91 (45)	118 (59)	161 (81)			
IL = 7.5 kN/m²	54 (27)	72 (36)	96 (48)	125 (62)	157 (79)				
IL = 10.0 kN/m²	66 (33)	93 (46)	120 (60)	159 (79)					

VARIATIONS TO DESIGN ASSUMPTIONS:

Thickness, mm	**Span, m**	**3.0**	**4.0**	**5.0**
	Unpropped, IL £ 10 kN/m²	115	150	150

Composite lattice girder soffit slabs



Precast plates act as permanent formwork and as precast soffits for robust, high-capacity, composite floor slabs.

The units are cast with most, if not all, of the bottom reinforcement required. Top reinforcement is fixed in-situ. The lattice girders give the precast section strength during construction. The units, typically 50 mm to 100 mm thick and 1200 mm or 2400 mm wide, are usually propped during construction. The chart and data relate to 75 mm (up to 200 mm final thickness) and 100 mm thick units. Self-weight can be reduced by having the units supplied with polystyrene void-formers bonded to the upper surface.

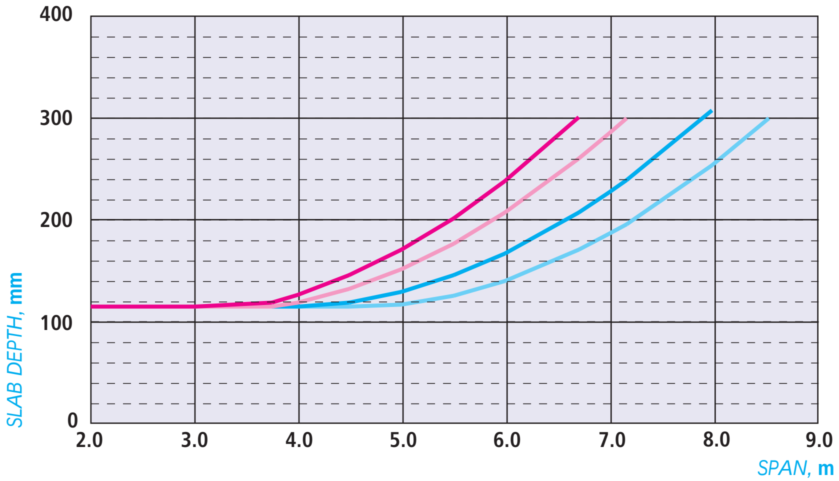
ADVANTAGES

- Speed
- Elimination of formwork
- Safe working platform
- Robust
- Quality soffit

DISADVANTAGES

- Propping usually required

SPAN:DEPTH CHART



KEY **Characteristic imposed load (IL)**

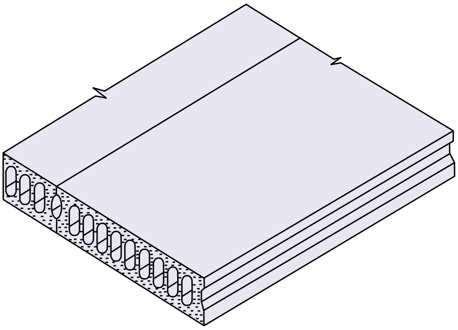
— = 2.5 kN/m² — = 5.0 kN/m² — = 7.5 kN/m² — = 10.0 kN/m²

SINGLE SPAN, m	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**8.5**
THICKNESS, mm						*Including topping. Propped at mid-point*		
IL = 2.5 kN/m²	115	115	115	116	140	186	254	300
IL = 5.0 kN/m²	115	115	115	130	168	226	308	
IL = 7.5 kN/m²	115	115	118	152	208	286		
IL = 10.0 kN/m²	115	115	126	170	238	330		
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), KN/M								
IL = 2.5 kN/m²	20 (10)	30 (15)	40 (20)	50 (25)	65 (32)	86 (43)	117 (58)	138 (69)
IL = 5.0 kN/m²	28 (14)	42 (21)	56 (28)	72 (36)	94 (47)	124 (62)	163 (82)	
IL = 7.5 kN/m²	36 (18)	54 (27)	72 (36)	96 (48)	126 (63)	166 (83)		
IL = 10.0 kN/m²	44 (22)	66 (33)	89 (45)	119 (60)	157 (78)	204 (102)		

VARIATIONS TO DESIGN ASSUMPTIONS: for a characteristic imposed load (IL) of 5.0 kN/m²

Unpropped	75 mm unit depth	115 to 200 mm deep max span 3.75 m
	100 mm unit depth	150 & 200 mm deep max span 5.00 m, 300 deep max span 4.71 m

Precast hollow-core slabs, *no topping*



Hollow-core floor slabs are precast prestressed concrete elements with continous voids provided to reduce self-weight and achieve structural efficiency. They are very popular, and economic across a wide range of spans and loadings. They are used in a wide range of buildings.

Depths range in increments from 110 mm to 450 mm; widths are generally 1200 mm. Span/load capacities may vary slightly between manufacturers. The soffit finish is suitable for exposure in car parks and industrial buildings, or for applied finishes. The top is designed to receive a levelling screed or appropriate flooring system.

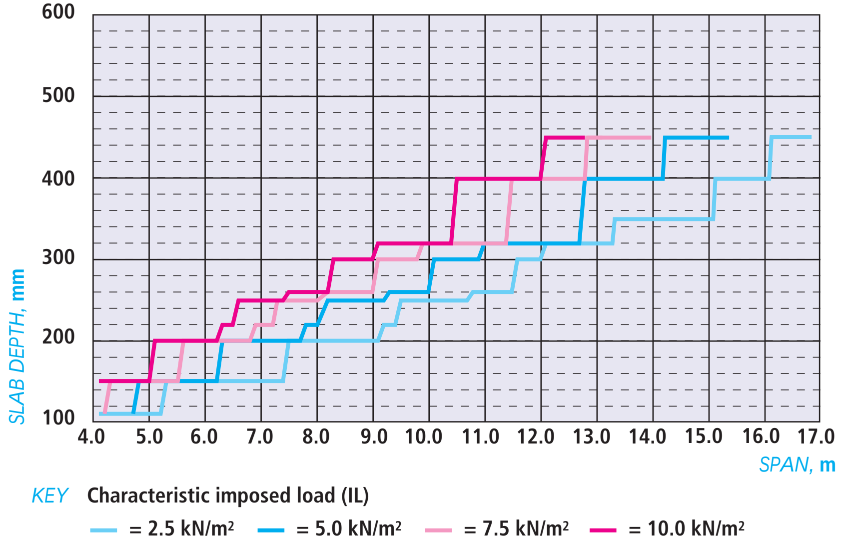
ADVANTAGES

- Speed
- Elimination of formwork
- Short, medium and long spans
- Elimination of propping
- High capacities
- Structural efficiency

DISADVANTAGES

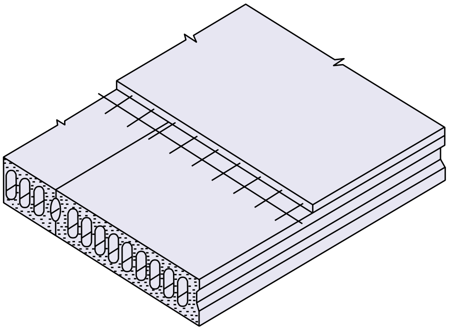
- Cranage may prove critical

SPAN:DEPTH CHART



SINGLE SPAN, m	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**12.0**	**14.0**	**16.0**
THICKNESS, mm									
IL = 2.5 kN/m²	110	150	150	200	200	250	300	350	400
IL = 5.0 kN/m²	150	150	200	220	250	260	320	400	
IL = 7.5 kN/m²	150	200	220	250	260	320	400	450	
IL = 10.0 kN/m²	150	200	250	260	300	320	400		
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m²	46 (23)	57 (29)	67 (34)	82 (41)	92 (46)	118 (59)	142 (71)	176 (88)	223 (111)
IL = 5.0 kN/m²	68 (34)	81 (41)	100 (50)	115 (58)	143 (71)	157 (78)	193 (96)	240 (120)	
IL = 7.5 kN/m²	88 (44)	109 (55)	129 (64)	159 (79)	177 (88)	201 (100)	248 (124)	308 (154)	
IL = 10.0 kN/m²	108 (54)	133 (67)	167 (83)	189 (95)	215 (107)	241 (120)	296 (148)		

Composite hollow-core slabs, *with topping*



Hollow-core floor slabs (see opposite) are used in conjunction with a structural topping where enhanced performance is required.

The units act compositely with the in-situ structural topping to creat a robust, high capacity composite floor. The structural topping overcomes possible differential camber between units, and is usually a grade C30 normal weight concrete, 50 mm thick, reinforced with a light mesh. Overall thicknesses are given.

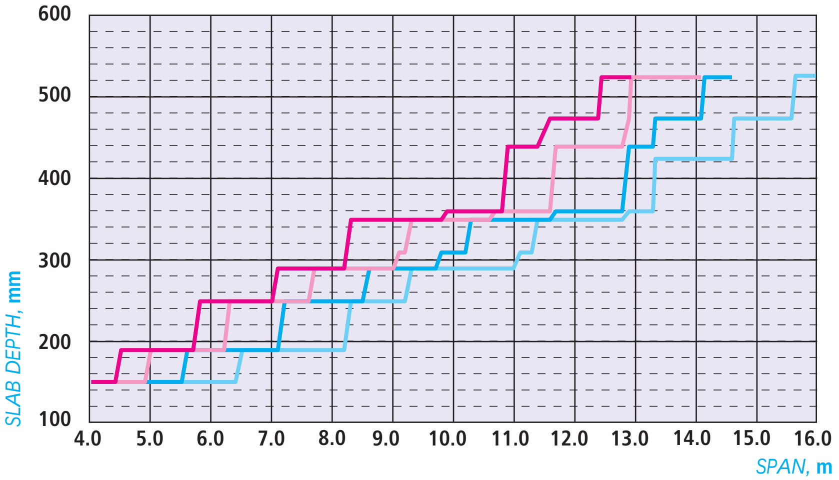
ADVANTAGES

- Speed
- High capacities
- Structural efficiency
- Short, medium and long spans
- Elimination of formwork
- Robustness
- Elimination of propping

DISADVANTAGES

- Cranage may prove critical

SPAN:DEPTH CHART

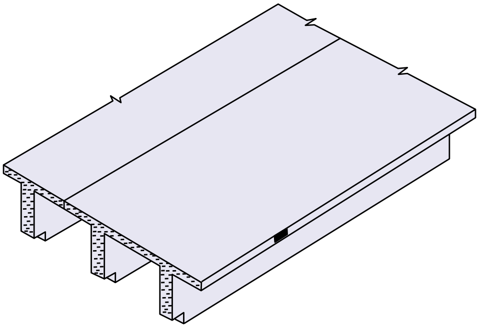


KEY **Characteristic imposed load (IL)**

— = 2.5 kN/m^2 — = 5.0 kN/m^2 — = 7.5 kN/m^2 — = 10.0 kN/m^2

SINGLE SPAN, m	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**12.0**	**14.0**	**16.0**
THICKNESS, mm								*Including topping*	
IL = 2.5 kN/m^2	150	150	190	190	250	290	350	425	525
IL = 5.0 kN/m^2	150	190	190	250	290	310	360	475	
IL = 7.5 kN/m^2	190	190	250	290	290	350	440		
IL = 10.0 kN/m^2	190	250	250	290	350	360	440		
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	53 (26)	63 (32)	78 (39)	89 (45)	107 (53)	123 (62)	162 (81)	207 (103)	290 (145)
IL = 5.0 kN/m^2	73 (36)	91 (45)	106 (53)	127 (63)	147 (73)	173 (87)	210 (105)	271 (135)	
IL = 7.5 kN/m^2	96 (48)	115 (57)	139 (69)	163 (81)	183 (91)	215 (108)	264 (132)		
IL = 10.0 kN/m^2	116 (58)	143 (72)	167 (83)	195 (97)	230 (115)	255 (128)	312 (156)		

Precast double 'T's, *no topping*



Double 'T's are used for long spans. They are relatively lightweight with a high load capacity. The units are prestressed and can be left exposed. TT2 units are intended for up to 2 hours fire resistance; TT4 for up to 4 hours. The top surface is designed to receive a levelling screed or appropriate flooring system.

Effective load sharing between units is achieved by welding cast-in plates together and brushing dry grout into the shaped longitudinal joints. Units are generally 2400 mm wide with ribs at approximately 1200 mm centres.

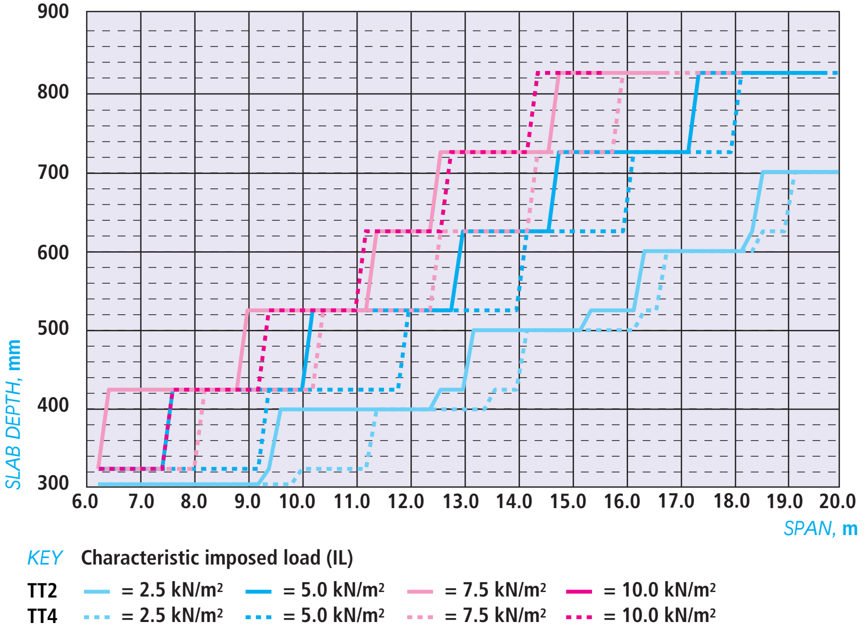
ADVANTAGES

- Quick
- Long spans
- Elimination of formwork and propping
- Efficient

DISADVANTAGES

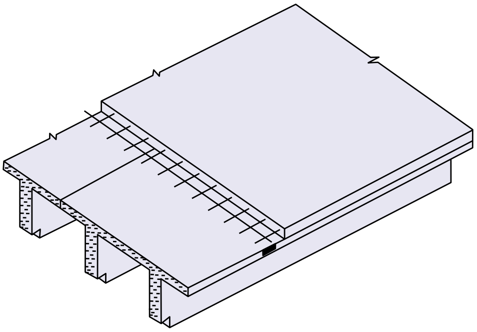
- Cranage may prove critical

SPAN:DEPTH CHART



SINGLE SPAN, m	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0
THICKNESS, mm TT2 (TT4)									
IL = 2.5 kN/m²	300 (300)	300 (300)	400 (325)	425 (425)	500 (425)	525 (500)	600 (600)	700 (700)	
IL = 5.0 kN/m²	325 (325)	425 (325)	425 (425)	525 (525)	625 (525)	725 (625)	825 (725)	825 (n/a)	
IL = 7.5 kN/m²	325 (325)	425 (325)	525 (425)	625 (525)	725 (625)	825 (825)	n/a (825)		
IL = 10.0 kN/m²	n/a (325)	n/a (425)	n/a (525)	n/a (625)	n/a (725)				
ULTIMATE LOAD TO END SUPPORTS, TT2 (TT4), kN/m									
IL = 2.5 kN/m²	27 (29)	36 (38)	47 (52)	61 (66)	69 (77)	86 (86)	94 (102)	109 (118)	
IL = 5.0 kN/m²	41 (43)	57 (57)	71 (75)	88 (93)	107 (109)	126 (129)	147 (150)	164 (n/a)	
IL = 7.5 kN/m²	53 (55)	73 (73)	93 (95)	115 (117)	138 (141)	163 (169)	n/a (190)		
IL = 10.0 kN/m²	n/a (67)	n/a (92)	n/a (118)	n/a (145)	n/a (172)				

Composite double 'T's, *with topping*



Double 'T's (see opposite) are used in conjunction with a structural topping where enhanced performance is required. Specifications vary between manufacturers

The units act compositely with the in-situ structural topping to create a robust composite floor. The structural topping overcomes possible differential camber between units and is usually a grade C30 normal-weight concrete, reinforced with a light mesh.

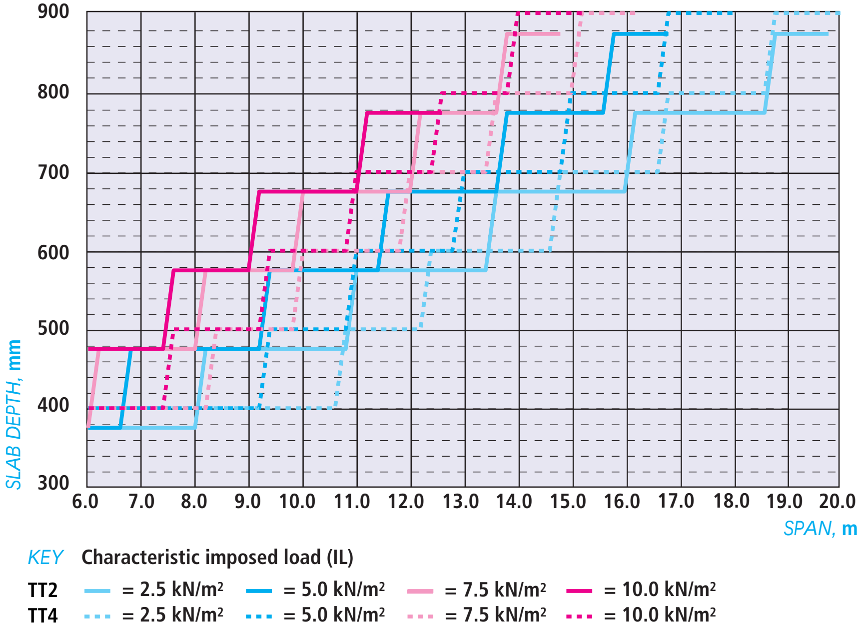
ADVANTAGES

- Quick
- Long spans
- Elimination of formwork and propping
- Robust
- Efficient

DISADVANTAGES

- Cranage may prove critical

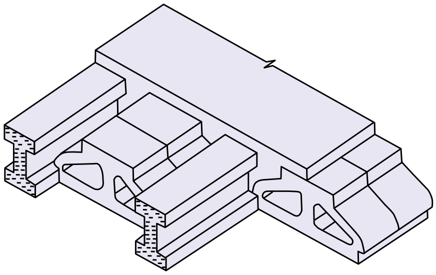
SPAN:DEPTH CHART



SINGLE SPAN, m	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0
THICKNESS, mm, TT2 + 75 mm topping (TT4 + 100 mm topping)									*Including topping*
IL = 2.5 kN/m^2	375 (400)	375 (400)	475 (400)	575 (500)	675 (600)	675 (700)	775 (800)	875 (900)	
IL = 5.0 kN/m^2	375 (400)	475 (400)	575 (500)	675 (600)	775 (700)	875 (800)	n/a (900)		
IL = 7.5 kN/m^2	375 (400)	475 (400)	675 (600)	675 (700)	875 (800)	n/a (900)			
IL = 10.0 kN/m^2	475 (400)	575 (500)	675 (600)	775 (700)	n/a (900)				
ULTIMATE LOAD TO END SUPPORTS, TT2 incl 75 mm topping (TT4 incl 100 mm topping), kN/m									
IL = 2.5 kN/m^2	34 (39)	46 (52)	59 (65)	74 (81)	90 (99)	103 (117)	121 (137)	141 (157)	
IL = 5.0 kN/m^2	46 (51)	64 (68)	82 (88)	102 (109)	122 (131)	144 (153)	n/a (177)		
IL = 7.5 kN/m^2	58 (63)	80 (84)	105 (111)	126 (136)	154 (162)	n/a (190)			
IL = 10.0 kN/m^2	72 (75)	97 (102)	125 (131)	153 (160)	n/a (194)				

Precast beam and block floors

(Beam and pot)



These systems combine prestressed beams with either solid blocks or voided 'pots'. They are widely used in the domestic market but can be used for commercial loadings for spans up to 6.5 m. Diaphragm action can be achieved by using a structural topping. Units are man-handable and ideal where access is restricted.

Flush soffits can be achieved using 'pots'. Holes can be formed by omitting 'pots' and making good. Slip tiles facilitate service runs or solid sections of concrete.

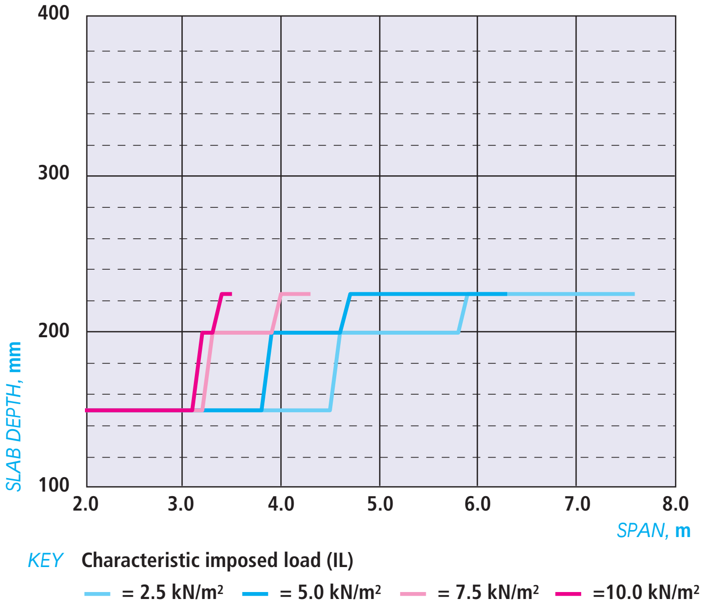
ADVANTAGES

- Ease of use
- Elimination of formwork
- Elimination of propping

DISADVANTAGES

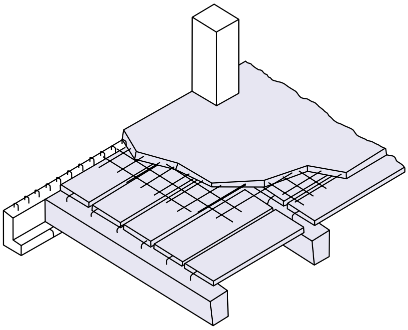
- Limited spans and capacities

SPAN:DEPTH CHART



SINGLE SPAN, m	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0
THICKNESS, mm									*Including any topping*
IL = 2.5 kN/m²	150	150	200	225	225				
IL = 5.0 kN/m²	150	200	225	225					
IL = 7.5 kN/m²	150	225							
IL = 10.0 kN/m²	150								
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m²	28 (14)	35 (18)	46 (23)	51 (26)	60 (30)				
IL = 5.0 kN/m²	40 (20)	52 (26)	63 (31)	80 (40)					
IL = 7.5 kN/m²	51 (25)	69 (35)							
IL = 10.0 kN/m²	63 (31)								

Composite prestressed rib floors



Precast, prestressed rib beams combine with precast soffit slabs or profiled metal decking and in-situ concrete to provide an economic, long-span ribbed floor. The ribs are manufactured in depths of 455 and 550 mm and used in slabs approximately 575 or 670 mm deep overall. Usually, they are at 2.0 to 2.4 m centres; closer centres increase load and span capacities. The extremes of the chart assume 0.9 m centres

The composite ribbed slab offers the advantages of a lightweight, yet efficient, floor construction, with the minimum of traditional formwork

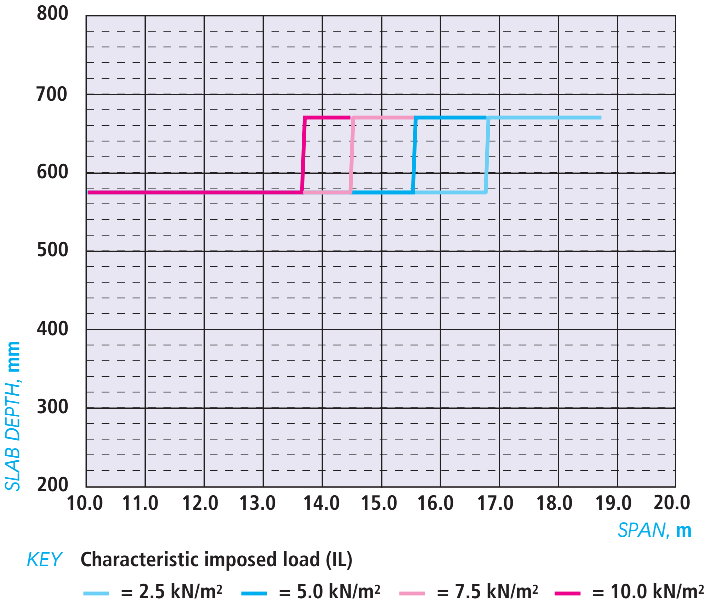
ADVANTAGES

- Speed
- Long spans
- Elimination of formwork
- Elimination of propping
- Structural efficiency
- Robustness

DISADVANTAGES

- Cranage may prove critical

SPAN:DEPTH CHART



SINGLE SPAN, m	10.0	11.0	12.0	14.0	16.0	18.0	20.0
THICKNESS, mm							*Including topping*
IL = 2.5 kN/m²	575	575	575	575	575	670	
IL = 5.0 kN/m²	575	575	575	575	670		
IL = 7.5 kN/m²	575	575	575	575	670		
IL = 10.0 kN/m²	575	575	575	670			
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m							
IL = 2.5 kN/m²	119 (60)	131 (66)	143 (72)	179 (89)	228 (114)	278 (139)	
IL = 5.0 kN/m²	159 (80)	175 (88)	195 (97)	248 (124)	303 (151)		
IL = 7.5 kN/m²	199 (100)	224 (112)	251 (125)	318 (159)	394 (197)		
IL = 10.0 kN/m²	243 (122)	276 (138)	307 (153)	377 (188)			

4.2 Beams

4.2.1 USING PRECAST AND COMPOSITE BEAMS

Factory-engineered precast concrete frames are used widely in offices, car parks, 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: a major cost item is the manufacture of the base moulds. Manufacturers should be consulted at the earliest opportunity (see Section 10.4).

4.2.2 USING THE CHARTS AND DATA

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, eg:
  isolated or upstand beams

'L' beams or single booted beams, eg:
  perimeter beams supporting hollow-core floor units

(Inverted) 'T' beams or double booted beams, eg:
  internal beams supporting hollow-core floor units

The charts assume that the beams are simply supported and non-composite, ie. no flange action or benefit from temporary propping is assumed. For 'L' and inverted 'T' beams, a ledge width of 125 mm has been assumed.

From the appropriate chart(s), use the maximum span and appropriate ultimate applied uniformly distributed loads to determine 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 his or her confidence in the design criteria used and normal modular sizing.

4.2.3 DESIGN ASSUMPTIONS

Reinforcement
Main bars: maximum T32T & B, minimum T20T & B at simply supported ends, links T10. Nominal T16T in mid-span. Minimum 50 mm between bars.

Concrete
C40, 24 kN/m^3, 20 mm aggregate. Fair-faced finish. Concrete grades up to C60 are commonly used to facilitate early removal from moulds. For severe exposure grade C50 concrete is assumed.

Fire and durability
Fire resistance 1 hour; mild exposure.

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

Span
For sizing precast beams, span can be taken as being centreline of support to centreline of support. For example, assuming 300 mm wide columns and, say, 100 mm from the end of beam to the centreline of support, beam span might be 500 mm less than centreline column to centreline column: however, for assessing loads to columns, the full centreline column to centreline column dimension should be used and is assumed in the charts and data.

Ledge widths
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.

Loads
Ultimate loads to columns assume elastic reaction factors of 1.0 to internal columns and 0.5 to end columns.

4.2.4 DESIGN NOTES

Different design criteria can be critical across the range of beams described. The sizes given in the charts and data are critical on the following parameters:

a A_sB (area of steel, bottom) restricted by end support width or length.

d Sizes given are close to requiring two layers of steel. The use of two layers of reinforcement in precast beams is not uncommon.

e Compression steel required in top of span.

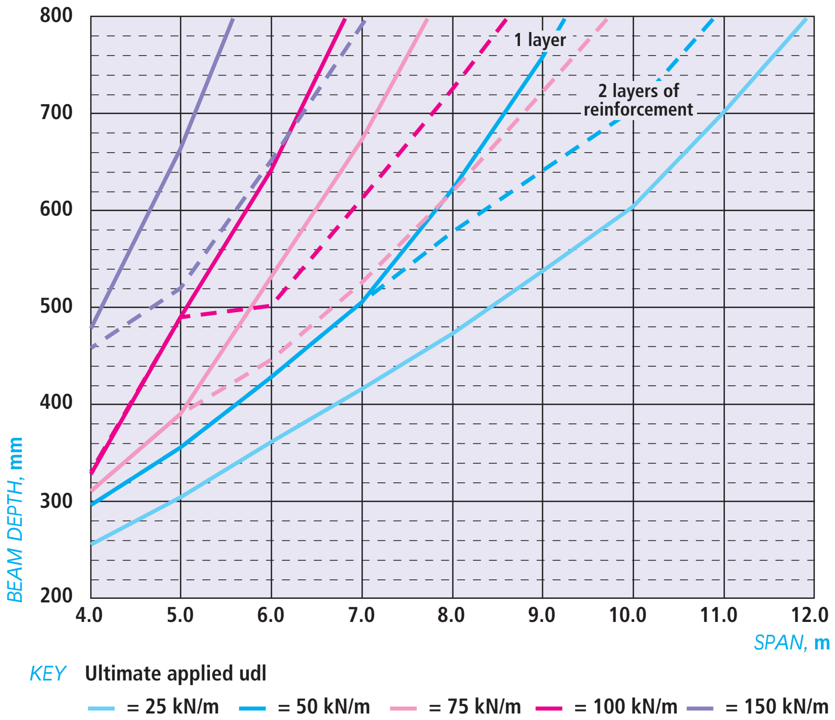
Rectangular precast beams

300 mm

wide

single span

SPAN:DEPTH CHART



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	258	306	364	418	476	540	606	704	810
uaudl = 50 kN/m	298	358	430	508	624	760			
uaudl = 75 kN/m	312	392	534	676	848				
uaudl = 100 kN/m	342	502	654	846					
uaudl = 150 kN/m	480	666	898						
ULTIMATE LOAD TO SUPPORTS/COLUMNS, INTERNAL (END), kN ult									
uaudl = 25 kN/m	110 (55)	140 (70)	172 (86)	204 (102)	238 (119)	274 (137)	312 (156)	354 (177)	398 (199)
uaudl = 50 kN/m	212 (106)	268 (134)	326 (163)	386 (193)	450 (225)	518 (259)			
uaudl = 75 kN/m	312 (156)	394 (197)	482 (241)	572 (286)	668 (334)				
uaudl = 100 kN/m	414 (207)	526 (263)	640 (320)	760 (380)					
uaudl = 150 kN/m	620 (310)	784 (392)	954 (477)						
DESIGN NOTES									*See Section 4.2.4 on p 90*
uaudl = 25 kN/m	a	a	a	a	a	a	ad	ad	ad
uaudl = 50 kN/m	ae	ae	ae	a	ad	ad			
uaudl = 75 kN/m	ae	aed	ad	ad	ad				
uaudl = 100 kN/m	ae	ad	ad	ad					
uaudl = 150 kN/m	ad	ad	ad						
VARIATIONS TO DESIGN ASSUMPTIONS (see Section 4.2.3 on p 90): implications on beam depths for 50 kN/m uaudl, mm									
2 hours fire	+ 5 mm								
4 hours fire	360	440	570	720	910				
Moderate exposure	300	400	520	680	870				
Severe exposure (C50)	330	410	540	690	880				

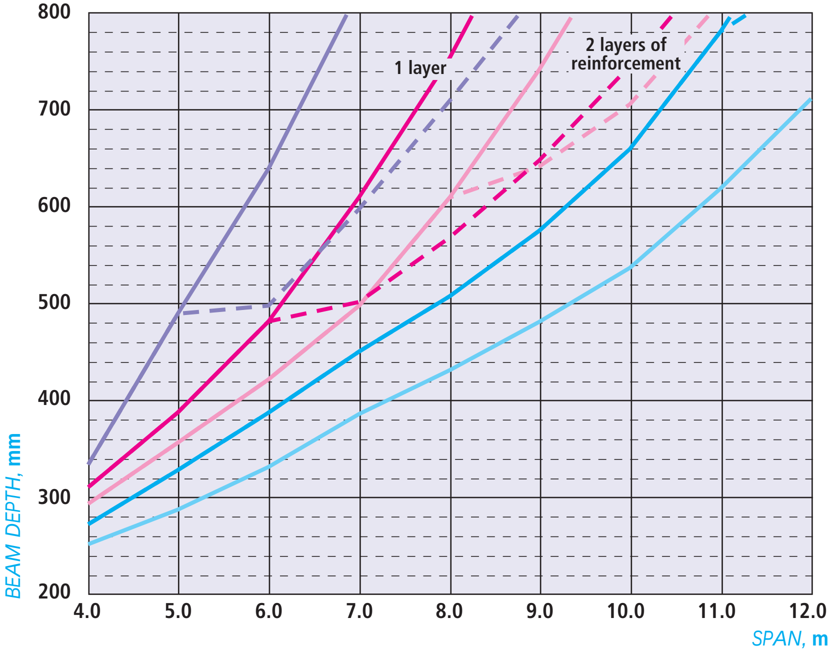
Rectangular precast beams

450 mm

wide

single span

SPAN:DEPTH CHART



KEY **Ultimate applied udl**

— = 25 kN/m — = 50 kN/m — = 75 kN/m — = 100 kN/m — = 150 kN/m

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	252	288	332	386	432	482	538	620	712
uaudl = 50 kN/m	272	328	388	450	508	576	660	782	
uaudl = 75 kN/m	294	356	422	498	610	742	896		
uaudl = 100 kN/m	310	388	482	610	754				
uaudl = 150 kN/m	334	490	640	824					
ULTIMATE LOAD TO SUPPORTS/COLUMNS, INTERNAL (END), kN ult									
uaudl = 25 kN/m	116 (58)	146 (73)	180 (90)	216 (108)	252 (126)	290 (145)	332 (166)	378 (189)	430 (215)
uaudl = 50 kN/m	216 (108)	274 (137)	336 (168)	398 (199)	462 (231)	528 (264)	600 (300)	680 (340)	
uaudl = 75 kN/m	318 (159)	402 (201)	488 (244)	578 (289)	674 (337)	776 (388)	886 (443)		
uaudl = 100 kN/m	418 (209)	526 (263)	644 (322)	764 (382)	892 (446)				
uaudl = 150 kN/m	620 (310)	788 (394)	958 (479)	1138 (569)					
DESIGN NOTES								*See Section 4.2.4 on p 90*	
uaudl = 25 kN/m							a	a	a
uaudl = 50 kN/m	a			a	a	a	ad	ad	
uaudl = 75 kN/m	ae	ae	ae	a	ad	ad	ad		
uaudl = 100 kN/m	aed	ae	aed	ad	ad				
uaudl = 150 kN/m	aed	ad	ad	ad					

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 4.2.3 on p 90): implications on beam depths for 50 kN/m uaudl, mm

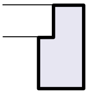
	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
2 hours fire	+ 5 mm								
4 hours fire	310	380	440	530	660	810			
Moderate exposure	280	340	400	470	530	620	750	900	
Severe exposure (C50)	290	350	410	500	630	780			

Precast 'L' beams

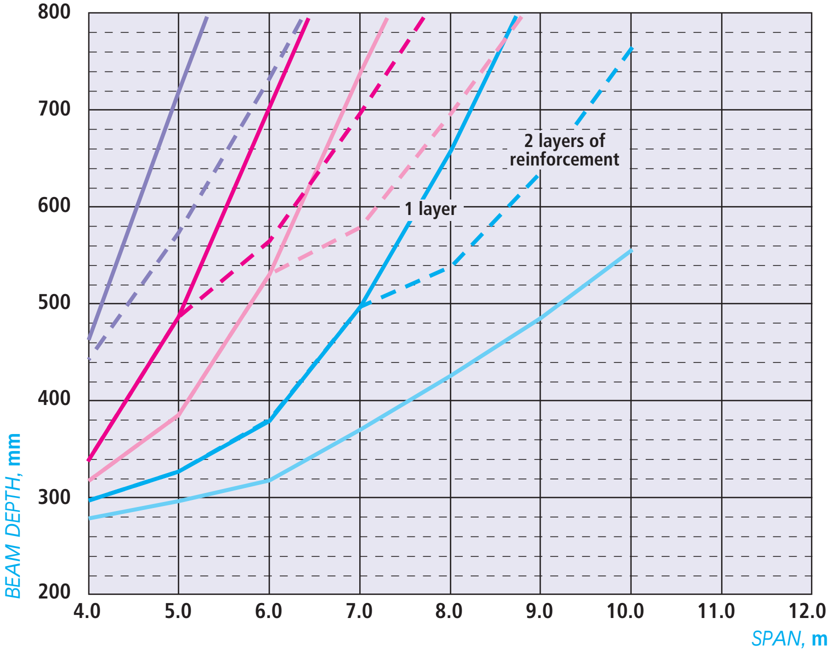
300 mm

wide overall

single span



SPAN:DEPTH CHART



KEY **Ultimate applied udl**

— = 25 kN/m — = 50 kN/m — = 75 kN/m — = 100 kN/m — = 150 kN/m

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	280	298	320	372	428	488	558		
uaudl = 50 kN/m	298	328	382	498	660	852			
uaudl = 75 kN/m	318	386	532	740					
uaudl = 100 kN/m	340	488	704						
uaudl = 150 kN/m	464	720							
ULTIMATE LOAD TO SUPPORTS/COLUMNS, INTERNAL (END), kN ult									
uaudl = 25 kN/m	110 (55)	140 (70)	170 (85)	202 (101)	234 (117)	270 (135)	306 (153)		
uaudl = 50 kN/m	212 (106)	266 (133)	324 (162)	386 (193)	454 (227)	528 (264)			
uaudl = 75 kN/m	312 (156)	394 (197)	482 (241)	578 (289)	676 (338)				
uaudl = 100 kN/m	414 (207)	524 (262)	642 (321)	764 (382)					
uaudl = 150 kN/m	618 (309)	786 (393)							
DESIGN NOTES							*See Section 4.2.4 on p 90*		
uaudl = 25 kN/m	ae	ae	ae	ae	aed	aed	aed		
uaudl = 50 kN/m	ae	ae	aed	aed	aed	ad			
uaudl = 75 kN/m	ae	aed	aed	aed					
uaudl = 100 kN/m	aed	aed	aed						
uaudl = 150 kN/m	aed	aed							

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 4.2.3 on p 90): implications on beam depths for 50 kN/m uaudl, mm

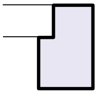
2 hours fire	+ 5 mm				
4 hours fire	400	480	610	770	960
Moderate exposure	+ 20 mm				
Severe exposure (C50)	330	430	560	710	910

Precast ‘L’ beams

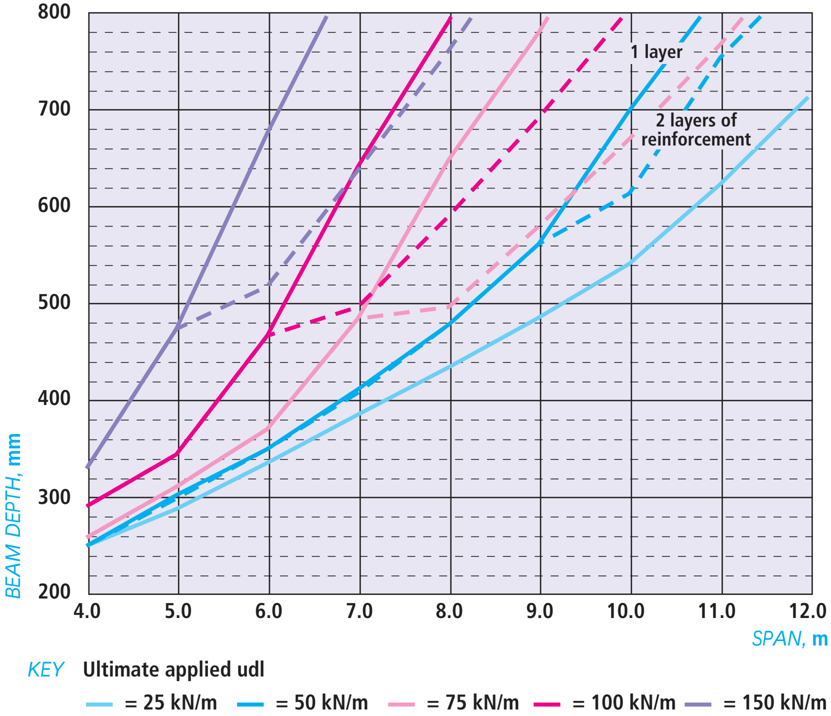
450 mm

wide overall

single span



SPAN:DEPTH CHART



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	252	290	338	388	436	488	544	626	718
uaudl = 50 kN/m	252	304	352	414	480	564	702	826	
uaudl = 75 kN/m	260	312	372	486	650	782			
uaudl = 100 kN/m	292	346	468	644	794				
uaudl = 150 kN/m	332	476	678	862					
ULTIMATE LOAD TO SUPPORTS/COLUMNS, INTERNAL (END), kN ult									
uaudl = 25 kN/m	116 (58)	146 (73)	180 (90)	216 (108)	252 (126)	292 (146)	332 (166)	380 (190)	430 (215)
uaudl = 50 kN/m	216 (108)	272 (136)	332 (166)	394 (197)	458 (229)	526 (263)	606 (303)	688 (344)	
uaudl = 75 kN/m	316 (158)	398 (199)	484 (242)	576 (288)	678 (339)	782 (391)			
uaudl = 100 kN/m	418 (209)	526 (263)	642 (321)	768 (384)	896 (448)				
uaudl = 150 kN/m	620 (310)	786 (393)	962 (481)	1142 (571)					
DESIGN NOTES									*See Section 4.2.4 on p 90*
uaudl = 25 kN/m	a						a	a	ad
uaudl = 50 kN/m	ae	ae	ae	aed	aed	aed	ad	ad	
uaudl = 75 kN/m	ae	aed	aed	aed	ad	ad			
uaudl = 100 kN/m	aed	aed	aed	aed	ad				
uaudl = 150 kN/m	aed	aed	ad	ad					

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 4.2.3 on p 90): implications on beam depths for 50 kN/m uaudl, mm

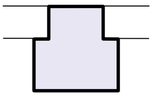
	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
2 hours fire	+ 5 mm								
4 hours fire	320	370	450	560	690	840			
Moderate exposure	260	310	380	450	540	660	790		
Severe exposure (C50)	290	350	420	520	650	800			

Precast inverted 'T' beams

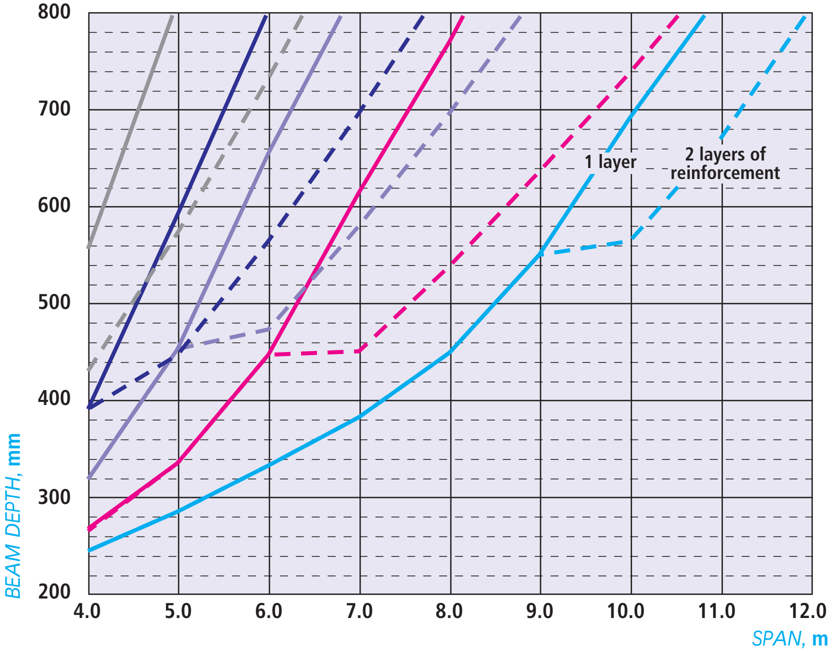
600 mm

wide overall

single span



SPAN:DEPTH CHART



KEY **Ultimate applied udl**

▬ = 50 kN/m ▬ = 100 kN/m ▬ = 150 kN/m ▬ = 200 kN/m ▬ = 300 kN/m

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	246	286	334	384	450	552	694	822	
uaudl = 100 kN/m	268	336	448	616	772				
uaudl = 150 kN/m	320	454	656	834					
uaudl = 200 kN/m	392	594	804						
uaudl = 300 kN/m	558	814							
ULTIMATE LOAD TO SUPPORTS/COLUMNS, INTERNAL (END), kN ult									
uaudl = 50 kN/m	218 (109)	278 (139)	340 (170)	402 (201)	472 (236)	550 (275)	640 (320)	732 (366)	
uaudl = 100 kN/m	422 (211)	534 (267)	654 (327)	786 (393)	924 (462)				
uaudl = 150 kN/m	626 (313)	796 (398)	980 (490)	1168 (584)					
uaudl = 200 kN/m	832 (416)	1060 (530)	1298 (649)						
uaudl = 300 kN/m	1244 (622)	1582 (791)							
DESIGN NOTES									*See Section 4.2.4 on p 90*
uaudl = 50 kN/m	ae	ae	ae	ae	aed	aed	ad	ad	
uaudl = 100 kN/m	ae	ae	aed	aed	ad				
uaudl = 150 kN/m	aed	aed	ad	ad					
uaudl = 200 kN/m	aed	aed	ad						
uaudl = 300 kN/m	aed	ad							

VARIATIONS TO DESIGN ASSUMPTIONS (see Section 4.2.3 on p 90): implications on beam depths for 100 kN/m uaudl

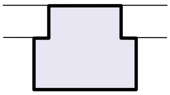
2 hours fire	+ 5 mm
4 hours fire	+ 50 mm
Moderate exposure	+ 20 mm
Severe exposure (C50)	+ 30 mm

Precast inverted ‘T’ beams

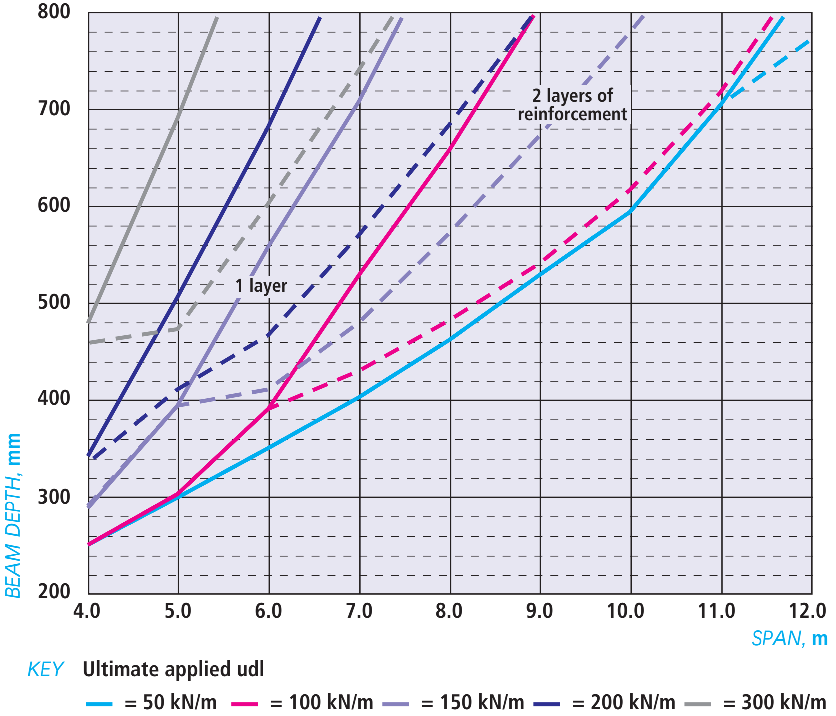
750 mm

wide overall

single span



SPAN:DEPTH CHART



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	252	300	352	404	464	530	596	706	838
uaudl = 100 kN/m	252	304	392	530	660	808			
uaudl = 150 kN/m	292	396	560	710	892				
uaudl = 200 kN/m	344	508	684	882					
uaudl = 300 kN/m	480	692							
ULTIMATE LOAD TO SUPPORTS/COLUMNS, INTERNAL (END), kN ult									
uaudl = 50 kN/m	226 (113)	288 (144)	354 (177)	422 (211)	492 (246)	570 (285)	650 (325)	746 (373)	854 (427)
uaudl = 100 kN/m	426 (213)	538 (269)	660 (330)	794 (397)	934 (467)	1084 (542)			
uaudl = 150 kN/m	630 (315)	800 (400)	984 (492)	1176 (588)	1380 (690)				
uaudl = 200 kN/m	834 (417)	1064 (532)	1304 (652)	1556 (778)					
uaudl = 300 kN/m	1248 (624)	1588 (794)							
DESIGN NOTES									*See Section 4.2.4 on p 90*
uaudl = 50 kN/m	ae	e	e	e	e	ad	ad	ad	ad
uaudl = 100 kN/m	ae	ae	aed	aed	ad	ad			
uaudl = 150 kN/m	ae	aed	ad	ad	ad				
uaudl = 200 kN/m	aed	aed	ad	ad					
uaudl = 300 kN/m	aed	ad	ad						

VARIATIONS TO DESIGN ASSUMPTIONS (see section 4.2.3 on p 90): implications on beam depths for 100 kN/m uaudl

2 hours fire	+ 5 mm
4 hours fire	+ 50 mm
Moderate exposure	+ 20 mm
Severe exposure (C50)	+ 30 mm

4.3 Columns

4.3.1 USING PRECAST COLUMNS

Precast columns 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.

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. 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, storey-height corbels are common in precast concrete car parks.

The economics of precast construction depend on repetition. As far as possible, the same section should be used throughout. Columns are often precast three or four storeys high.

4.3.2 USING THE CHARTS AND DATA

The column charts give square sizes against **ultimate** axial load for a range of steel contents for internal, edge and corner braced columns. Column design is dependant upon ultimate axial load and ultimate design moment. Design moments are specific to a project and cannot be generalized. 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. For similar reasons, reinforcement densities are not quoted.

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 his or her confidence in the design criteria used and normal modular sizing. The thickness of any specialist finishes required should be added to the sizes given.

The column charts 'work' on **total ultimate** axial load (N) in kN. Preferably, this load should be calculated from first principles for the lowest level of column under consideration (see Section 8.3). However, it may suffice to estimate the load in accordance with Section 2.7.

The charts for internal columns assume equal adjacent spans in each direction.

The charts for edge and corner columns give sizes according to the number of storeys in order to allow for the effects of moments generated by the eccentricity of the beam/column connection. As explained in Section 7, the sizes should generally prove conservative. As axial load predominates, so the design is less controlled by moment. Above about five storeys, perimeter columns can be sized by using the chart for internal columns. The sizes given may prove to be inadequate when unequal spans, eccentric loads or high imposed loads are envisaged.

4.3.3 DESIGN ASSUMPTIONS

Reinforcement
Main bars: fy = 460 N/mm^2, links fy = 250 N/mm^2. Link size, maximum main bar size/4. Maximum bar size T40.

Concrete
C50, 24 kN/m^3, 20 mm aggregate.

Fire and durability
Fire resistance 1 hour; mild exposure.

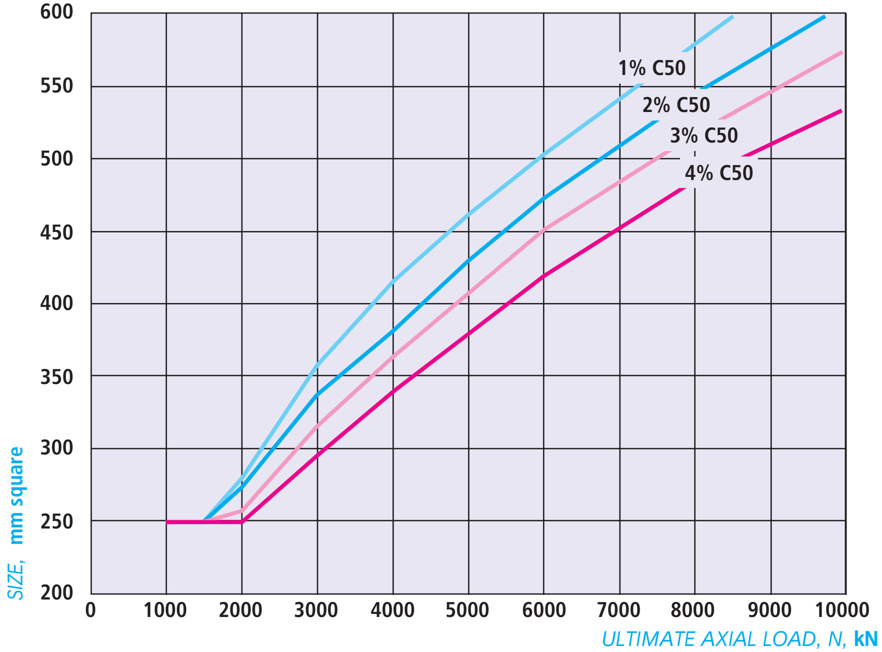
4.3.4 DESIGN NOTES

Internal columns
The charts and data for internal columns assume equal spans in each orthogonal direction (ie. $l_{x1} = l_{x2}$ and $l_{y1} = l_{y2}$). If spans are unequal by more than, say, 15%, then consider treating the column as an edge column.

Perimeter (edge and corner) columns
The charts and data for edge columns assume 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.

Precast internal columns

LOAD:SIZE CHART

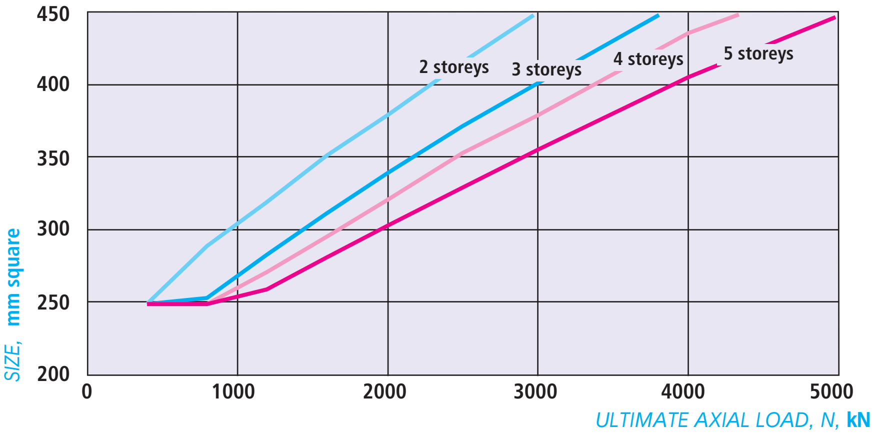


ULTIMATE AXIAL LOAD, kN

SIZE, mm square	1000	1500	2000	3000	4000	5000	6000	8000	10000
1.0% C50	250	250	280	358	416	462	504	580	656
2.0% C50	250	250	274	338	382	430	474	546	608
3.0% C50	250	250	258	316	364	408	452	518	576
4.0% C50	250	250	250	296	340	380	420	486	536

Precast edge columns 2% reinforcement

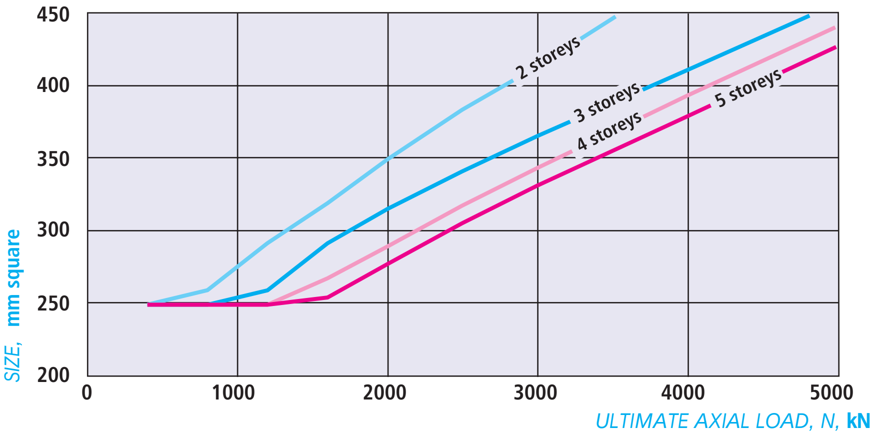
LOAD:SIZE CHART



ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	2500	3000	4000	5000
SIZE, mm square									*C50 concrete*
2 storeys	250	290	320	352	380	416	450		
3 storeys	250	254	284	312	340	372	402	460	
4 storeys	250	250	272	296	322	354	380	436	474
5 storeys	250	250	260	282	304	330	356	406	448

Precast edge columns 3% reinforcement

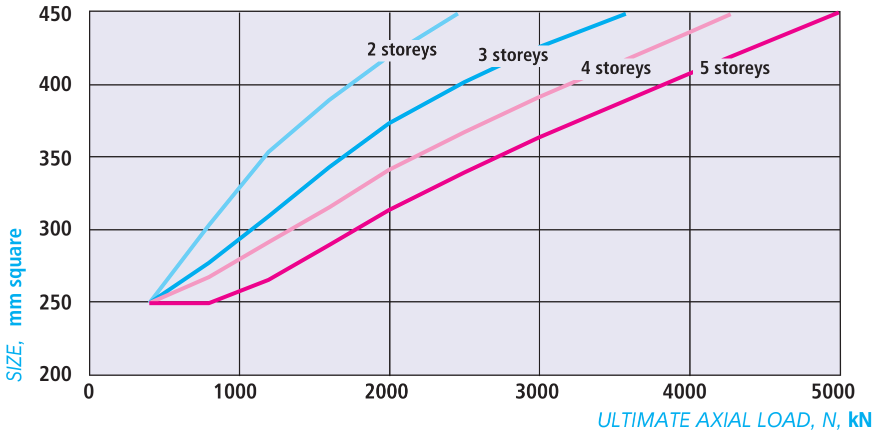
LOAD:SIZE CHART



ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	2500	3000	4000	5000
SIZE, mm square									*C50 concrete*
2 storeys	250	260	292	320	350	384	414	480	
3 storeys	250	250	260	292	316	342	366	412	458
4 storeys	250	250	250	268	290	318	344	394	442
5 storeys	250	250	250	255	278	306	332	380	428

Precast corner columns 2% *reinforcement*

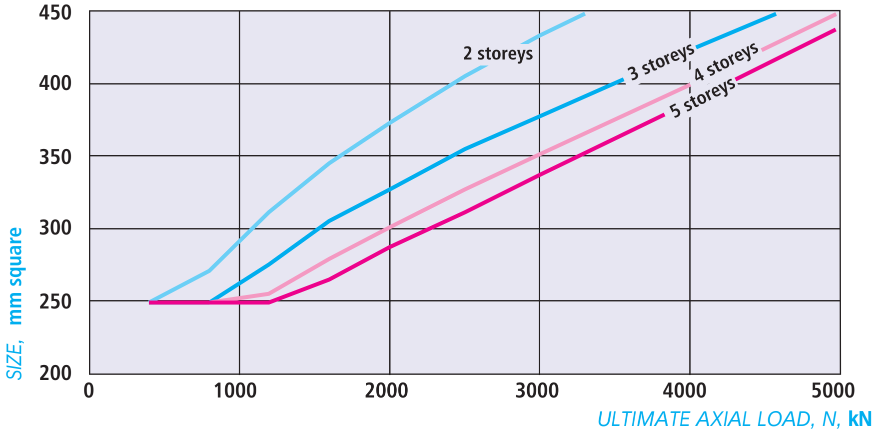
LOAD:SIZE CHART



ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	2500	3000	4000	5000
SIZE, mm square									*C50 concrete*
2 storeys	250	304	354	390	420	452	480		
3 storeys	250	278	310	344	374	402	426	466	514
4 storeys	250	268	292	316	342	368	392	436	482
5 storeys	250	250	266	290	314	340	364	408	450

Precast corner columns 3% *reinforcement*

LOAD:SIZE CHART



ULTIMATE AXIAL LOAD, kN	400	800	1200	1600	2000	2500	3000	4000	5000
SIZE, mm square									*C50 concrete*
2 storeys	250	272	312	346	374	406	434	484	
3 storeys	250	250	276	306	328	356	378	424	468
4 storeys	250	250	256	280	302	328	352	400	450
5 storeys	250	250	250	266	288	312	338	388	440

5.1 Notes

5.1.1 POST-TENSIONING

Compressing concrete, using tensioned high strength steel strands, reduces or even eliminates tensile stresses and cracks in the concrete. This gives rise to a range of benefits over normally reinforced sections: 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 strands before casting, (ie. pre-tensioning) tends to be used in the factory, eg, in precast floor units; and post-tensioning tends to be used on site.

In floors, where the level of prestress tends to be low, post-tensioning is usually achieved using monostrand **unbonded** tendons (typically 15.7 mm in diameter, covered in grease within a protective sheath) cast into the concrete. Once the concrete achieves sufficient strength, tendons are stressed using a simple hand-held jack and anchored off.

In beams, where the level of prestress tends to be higher and where tendon congestion is to be avoided (or in one-way slabs and beams, where large amounts of normal untensioned reinforcement are to be avoided), post-tensioning is generally achieved using multi-strand **bonded** tendons (eg. 3, 4, 5, or 9 no. 15.7 mm strands in round or flattened galvanised ducts). These too are cast into the concrete and tensioned once the concrete has gained sufficient strength. The strands are then anchored off and the ducts grouted.

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 No. 43, *Post-tensioned concrete floors - design handbook*(10) gives further details of design. *Post-tensioned floors for multi-storey buildings*(11) gives more general guidance. For specific applications, advice should be sought from specialist engineers and contractors.

5.1.2 USING THE CHARTS AND DATA

The charts and data for slabs cover one-way solid, ribbed and flat slabs, and assume the use of unbonded 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 first set of charts for post-tensioned beams assume 1000 mm wide rectangular beams with no flange action. Other web widths can be investigated on a pro-rata basis, ie. by determining the ultimate applied uniformly distributed load per metre width of web. 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 8.2). 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 his or her confidence in the design criteria used and normal modular sizing.

Please note that for any given load and span, there is a range of legitimate depths depending on the amount of prestress assumed. Indeed, in practice, many post-tensioned elements are designed to make a certain depth work (see Section 7.3).

5.1.3 DESIGN NOTES

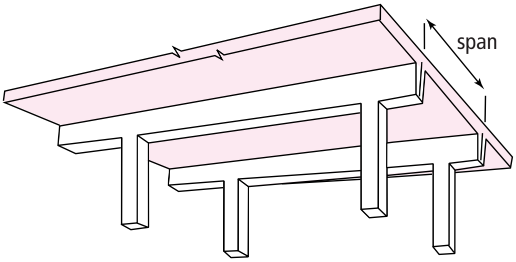
The charts and data assume the use of single-strand unbonded tendons. In longer spans, where single-strand unbonded tendons would become congested, consideration should be given to using bonded multi-strand tendons in flat or round ducts. In such cases, appropriate allowances should be made as several design assumptions made in the derivation of the charts become invalid (eg. cover, effective depth, wobble factor, etc.). Generally sections with bonded tendons need to be deeper than the theoretical sizes indicated for sections with unbonded tendons.

Design assumptions for the individual types of slab and beams are described in the relevant data. Other assumptions made are described and discussed in Section 7. Reinforcement and tendon quantities are approximate only (see Section 2.2.4).

For specific applications, advice should be sought from specialist engineers and contractors (see Section 10.4). For examples: CDM regulations oblige designers to consider demolition during initial design, and the effects of restraint need to be assessed. The use of detailed frame analysis can lead to significant economies in an overall package.

5.2 Post-tensioned slabs

One-way slabs



One-way in-situ solid slabs are the most basic form of slab. Post-tensioning can minimize slab thickness and control deflection and cracking. Generally used in office buildings and car parks. Economical in spans up to 10 m.

ADVANTAGES

- Simple
- Minimum thickness
- Controlled deflection and cracking

SINGLE SPAN, m	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
THICKNESS, mm									
IL = 2.5 kN/m²	206	206	214	244	278	312	350	390	434
IL = 5.0 kN/m²	206	216	246	280	310	344	382	430	498
IL = 7.5 kN/m²	206	238	272	310	342	376	416	486	
IL = 10.0 kN/m²	230	264	300	334	368	402	462		
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m²	n/a (39)	n/a (45)	n/a (53)	n/a (64)	n/a (77)	n/a (91)	n/a (107)	n/a (125)	n/a (145)
IL = 5.0 kN/m²	n/a (51)	n/a (61)	n/a (73)	n/a (88)	n/a (102)	n/a (116)	n/a (135)	n/a (156)	n/a (188)
IL = 7.5 kN/m²	n/a (63)	n/a (77)	n/a (93)	n/a (110)	n/a (128)	n/a (145)	n/a (167)	n/a (198)	
IL = 10.0 kN/m²	n/a (77)	n/a (94)	n/a (113)	n/a (132)	n/a (151)	n/a (173)	n/a (202)		
REINFORCEMENT (TENDONS), kg/m²									
IL = 2.5 kN/m²	11 (4)	10 (5)	10 (5)	10 (6)	10 (6)	11 (7)	11 (8)	12 (9)	14 (10)
IL = 5.0 kN/m²	11 (5)	10 (5)	10 (6)	11 (6)	12 (7)	13 (8)	14 (8)	15 (10)	18 (10)
IL = 7.5 kN/m²	11 (5)	11 (5)	11 (6)	11 (7)	12 (8)	14 (8)	15 (9)	17 (10)	
IL = 10.0 kN/m²	12 (5)	12 (5)	12 (6)	12 (7)	14 (8)	15 (9)	17 (10)		

DESIGN NOTES *o = limited by P/A of 2.5 N/mm²* *p = 8 > response factor > 4* *q = shrinkage >10 mm*
r = 15.7 mm diam tendons @ < 300 mm cc. (R @ < 200, min 150, mm cc.) *s = overall deflection > 20 mm (S > 30 mm)*

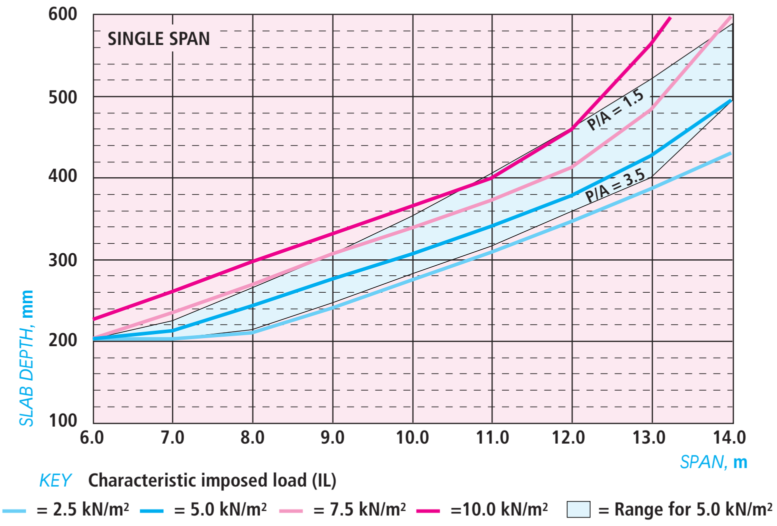
	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
IL = 2.5 kN/m²	p	opr	oprs	oprS	oprS	oRS	oRS	oRS	oRS
IL = 5.0 kN/m²	p	opr	opr	ors	oRs	oRS	oRS	oRS	RS
IL = 7.5 kN/m²	pr	opr	opr	oR	oRs	oRS	oRS	RS	
IL = 10.0 kN/m²	p	pr	r	R	oRs	oRs	Rs		

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

Serviceability	Class 1	222 mm @ 6m, 258 @ 7 m, 384 @ 8m	Class 2	+10 mm up to 9 m; 30 mm up to 12 m
Fire resistance	2 hours	+0 mm	4 hours	+25 mm up to 11 m
Exposure	Moderate	+0 mm	Severe	+15 mm

Thickness, mm	Span, m	8.0	9.0	10.0	11.0	12.0	13.0	14.0
	P/A 1.5 N/mm² max	268	310	356	408	464	524	
	P/A 3.5 N/mm² max	218	250	286	320	362	404	498

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	BEAMS. Refer to beam charts and data to estimate sizes and reinforcement.
DESIGN BASIS	To CS TR43[10]. Balanced load 100% DL + 25% IL . Maximum prestress (P/A) = 2.5 N/mm^2. See section 7. Class 3 and no restraint to movement assumed.
LOADS	A superimposed dead load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) is included. For mulltiple spans, ultimate loads result from moment distribution analysis for 3 span condition.
TENDONS	Unbonded 15.7 mm diam. Superstrand (A_{ps} 150 mm^2, f_{pu} 1770 N/mm^2). T2 and B2 . Max. 7 per m. 10% allowed for wastage and laps of bonded reinforcement and tendons (see Section 2.2.4).
REINFORCEMENT	f_y = 460 N/mm^2. Assumed T10 T1 and T2.
CONCRETE	C40, 24 kN/m^3, 20 mm aggregate. f_{ci} = 25 N/mm^2.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure (25 mm cover to all).

MULTIPLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
THICKNESS, mm									
IL = 2.5 kN/m^2	180	180	184	210	234	262	290	352	418
IL = 5.0 kN/m^2	180	188	216	244	272	302	334	402	520
IL = 7.5 kN/m^2	188	226	258	290	322	352	382	458	612
IL = 10.0 kN/m^2	238	274	312	346	382	416	452	524	694
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	84 (31)	98 (37)	114 (42)	137 (51)	161 (60)	189 (70)	219 (81)	290 (107)	372 (137)
IL = 5.0 kN/m^2	112 (42)	133 (50)	160 (60)	190 (71)	222 (83)	257 (96)	295 (110)	381 (142)	509 (189)
IL = 7.5 kN/m^2	142 (54)	175 (67)	210 (80)	248 (94)	288 (108)	329 (124)	373 (140)	476 (179)	640 (238)
IL = 10.0 kN/m^2	181 (69)	221 (84)	264 (100)	309 (117)	357 (135)	407 (154)	461 (174)	577 (217)	765 (286)
REINFORCEMENT (TENDONS), kg/m^2									
IL = 2.5 kN/m^2	7 (3)	8 (4)	8 (4)	9 (5)	10 (5)	12 (6)	13 (7)	16 (8)	19 (10)
IL = 5.0 kN/m^2	7 (4)	8 (4)	9 (5)	10 (6)	11 (6)	12 (7)	13 (8)	16 (9)	19 (10)
IL = 7.5 kN/m^2	7 (4)	8 (4)	9 (5)	10 (5)	11 (6)	12 (7)	13 (8)	16 (10)	20 (10)
IL = 10.0 kN/m^2	8 (3)	9 (4)	10 (4)	10 (5)	12 (6)	13 (7)	14 (8)	17 (10)	21 (10)

DESIGN NOTES *o = limited by P/A of 2.5 N/mm^2* *p = 8 > response factor > 4* *q = shrinkage per span >10 mm*
r = 15.7 mm diam tendons @ < 300 mm cc. (R @ < 200, min 150, mm cc.) *s = overall deflection > 20 mm (S > 30 mm)*

MULTIPLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
IL = 2.5 kN/m^2	p	op	op	opr	ors	ors	ors	oRs	oqRS
IL = 5.0 kN/m^2	p	o	or	or	or	oR	oR	oRs	Rs
IL = 7.5 kN/m^2	p		r	r	r	R	oR	oR	R
IL = 10.0 kN/m^2				r	r	r	R	R	R

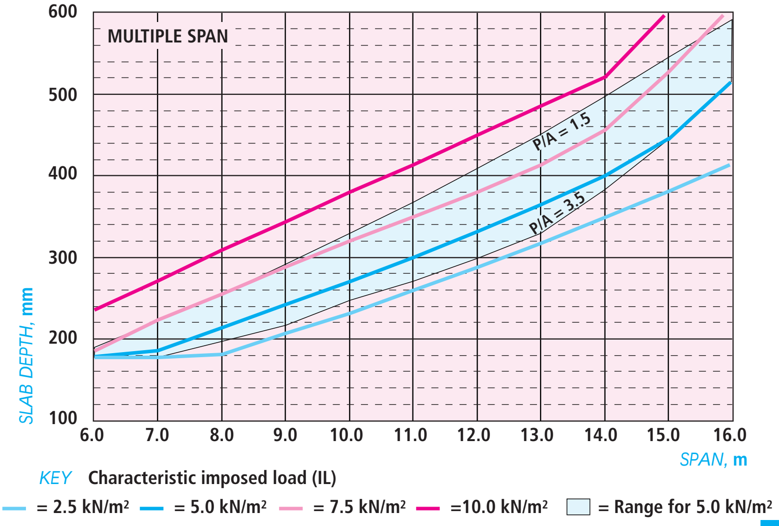
VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Fire resistance	2 hours	+5 mm	4 hours	+25 mm
Exposure	Moderate or severe	+5 mm	Class 2	+0 mm
Wide beam	2.4 m wide beam	- 0 mm	100% sustained load	+0 mm
Two span	Two span + 0 mm up to 12 m, +50 mm @ 16 m			

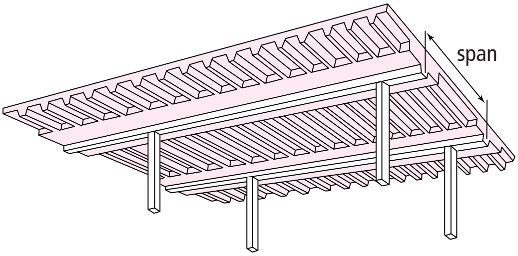
Thickness, mm

Spans, m	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
P/A 1.5 N/mm^2 max	258	294	332	370	412	500	596
P/A 3.5 N/mm^2 max	200	220	250	274	302	386	520

SPAN:DEPTH CHART



Ribbed slabs



Post-tensioning can minimize slab thickness and control deflection and cracking. Generally employed in office buildings and car parks. Economical in spans from 8 to 18 m. Charts are based on 300 mm wide ribs, spaced at 1200 mm centres.

ADVANTAGES

- Medium and long spans
- Lightweight
- Profile can be expressed architecturally
- Holes in topping cause few structural problems

SINGLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
THICKNESS, mm									
IL = 2.5 kN/m^2	278	298	298	298	316	364	416	532	668
IL = 5.0 kN/m^2	278	298	298	328	376	426	480	598	732
IL = 7.5 kN/m^2	278	312	358	402	446	496	554	690	
IL = 10.0 kN/m^2	330	386	442	502	562	626	692		
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m^2	n/a (33)	n/a (39)	n/a (44)	n/a (50)	n/a (56)	n/a (64)	n/a (73)	n/a (92)	n/a (114)
IL = 5.0 kN/m^2	n/a (45)	n/a (53)	n/a (60)	n/a (69)	n/a (79)	n/a (89)	n/a (100)	n/a (123)	n/a (150)
IL = 7.5 kN/m^2	n/a (57)	n/a (67)	n/a (78)	n/a (90)	n/a (102)	n/a (114)	n/a (128)	n/a (157)	
IL = 10.0 kN/m^2	n/a (70)	n/a (83)	n/a (97)	n/a (112)	n/a (127)	n/a (142)	n/a (159)		
REINFORCEMENT (TENDONS), kg/m^2									
IL = 2.5 kN/m^2	14 (2)	13 (2)	13 (3)	12 (3)	12 (4)	13 (4)	14 (4)	16 (5)	19 (6)
IL = 5.0 kN/m^2	14 (2)	13 (3)	13 (3)	13 (4)	14 (4)	15 (4)	16 (4)	18 (5)	21 (6)
IL = 7.5 kN/m^2	14 (2)	14 (3)	14 (3)	15 (4)	16 (4)	16 (5)	18 (5)	21 (6)	
IL = 10.0 kN/m^2	15 (2)	16 (2)	17 (3)	18 (3)	19 (4)	20 (4)	21 (5)		

DESIGN NOTES *n = shear links required in ribs* *o = limited by P/A of 2.5 N/mm^2* *p = 8 > response factor > 4*
q = shrinkage >10 mm *r = no. tendons req'd. per 300 mm rib > 4 (R > 5)* *s = o/a deflection > 20 mm (S > 30 mm)*

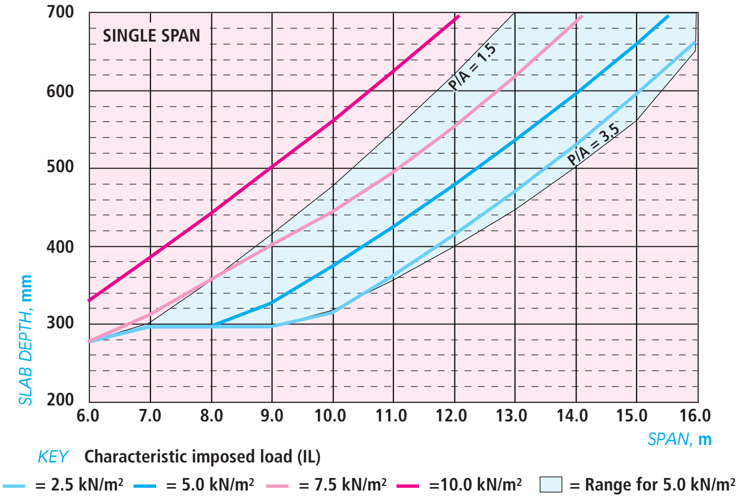
	6.0	7.0	8.0	9.0	10.0	11.0	12.0	14.0	16.0
IL = 2.5 kN/m^2	p	p	p	p	ps	ps	ps	rs	qrs
IL = 5.0 kN/m^2	p	p	p	p	p			rs	Rs
IL = 7.5 kN/m^2	p	p	p	p			r	r	
IL = 10.0 kN/m^2	p	p					r		

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m^2

Fire resistance	2 hours (115 topping)	+5 mm	4 hours (150 mm topping)	+15 mm
Exposure	Moderate	+0 mm	Severe	+10 mm
Serviceability	Class 1 @ 3.5 N/mm^2 & 180% DL	+0 mm	Class 2	see below

Thickness, mm	**Span, m**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
	P/A 1.5 N/mm^2 max	356	416	478	548	622	702	788
	P/A 3.5 N/mm^2 max	298	298	318	358	400	448	504
	Class 2	350	386	420	462	520	580	644
	ditto but 300 ribs @1500 cc		344	396	450	508	570	658

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	BEAMS. Refer to beam charts and data to estimate sizes and reinforcement.
DESIGN BASIS	Load balanced to 133% DL + 33% IL to maximum prestress (P/A) = of 2.5 N/mm². Class 3 and no restraint to movement assumed. See Section 7.
DIMENSIONS	300 mm ribs at 1200 mm cc. 100 mm topping. Solid area to span/9.6 from internal support
LOADS	A superimposed dead load (SDL) of 1.50 kN/m² (for finishes, services, etc.) is included. Self weight allows for slope on ribs and solid areas as indicated above (see Section 8.1.4 for range of values). For multiple spans, ultimate loads result from moment distribution analysis for three-span condition.
TENDONS	Unbonded 15.7 mm diam. Superstrand (A_{ps} 150 mm², f_{pu} 1770 N/mm²) B1 & T2. Max. 6 no. per rib
CONCRETE	C40, 24 kN/m³, 20 mm aggregate. f_{ci} = 25 N/mm².
REINFORCEMENT	fy = 460 N/mm². Assumed T10 T1 distribution reinforcement at supports and R8 links. Weight of mesh (A142, T2) not included.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure (25 mm cover to all).

MULTIPLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
THICKNESS, mm									
IL = 2.5 kN/m²				250	276	308	342	414	494
IL = 5.0 kN/m²		250	260	296	334	374	414	502	598
IL = 7.5 kN/m²	250	288	326	362	398	436	478	580	692
IL = 10.0 kN/m²	304	350	398	446	494	542	592	690	792
ULTIMATE LOAD TO SUPPORTING BEAMS, INTERNAL (END), kN/m									
IL = 2.5 kN/m²			94 (38)	108 (44)	123 (50)	139 (56)	155 (63)	191 (77)	231 (93)
IL = 5.0 kN/m²	96 (40)	117 (48)	133 (55)	153 (63)	174 (71)	196 (79)	218 (88)	267 (108)	320 (129)
IL = 7.5 kN/m²	127 (52)	150 (61)	175 (71)	200 (81)	225 (92)	252 (102)	280 (114)	341 (137)	407 (164)
IL = 10.0 kN/m²	157 (64)	187 (76)	217 (88)	249 (101)	281 (113)	314 (127)	348 (140)	420 (169)	495 (199)
REINFORCEMENT (tendons), kg/m²									
IL = 2.5 kN/m²			9 (3)	9 (3)	10 (3)	11 (3)	11 (4)	13 (4)	15 (5)
IL = 5.0 kN/m²		10 (3)	10 (3)	11 (3)	12 (4)	13 (4)	14 (4)	17 (5)	20 (5)
IL = 7.5 kN/m²	11 (2)	12 (2)	13 (3)	13 (3)	14 (4)	15 (4)	16 (4)	19 (5)	23 (6)
IL = 10.0 kN/m²	13 (2)	14 (2)	15 (3)	16 (3)	17 (3)	19 (4)	20 (4)	24 (5)	27 (6)

DESIGN NOTES *n = shear links required in ribs* *o = limited by P/A of 2.5 N/mm²* *p = 8 > response factor > 4*

q = shrinkage >10 mm *r = no. tendons req'd. per 300 mm rib > 4 (R > 5)* *s = o/a deflection > 20 mm (S > 30 mm)*

	6.0	7.0	8.0	9.0	10.0	11.0	12.0	14.0	16.0
IL = 2.5 kN/m²			op	op	op	op	o	os	oqs
IL = 5.0 kN/m²		op	o	o	o	o	o	or	oqr
IL = 7.5 kN/m²	p	p				o	o	or	noqr
IL = 10.0 kN/m²								nr	noqR

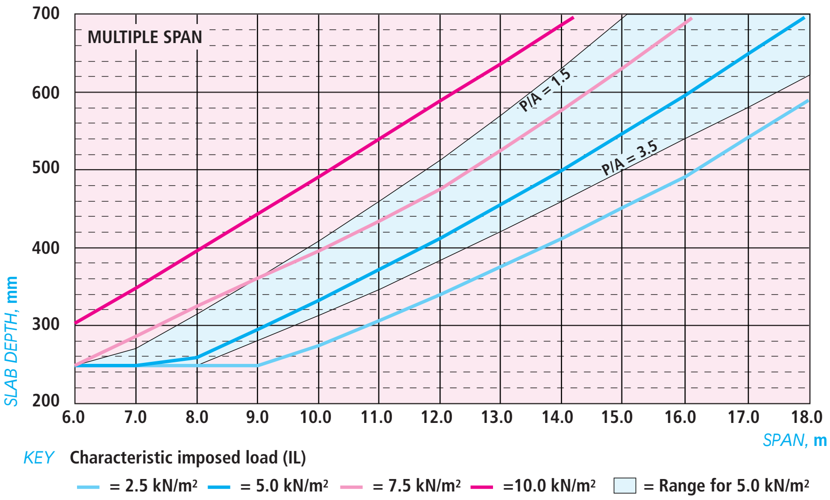
VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

Fire resistance	2 hours (115 topping)	+0 mm	4 hours (150 mm topping)	+15 mm
Exposure	Moderate	+5 mm	Severe	+10 mm
Serviceability	Class 1.	n/a	Class 2	see below

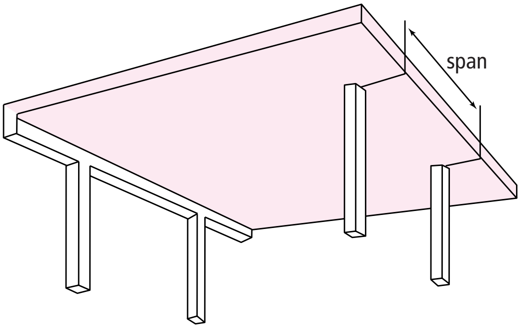
Thickness, mm

Spans, m	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**14.0**	**16.0**
P/A 1.5 N/mm² max	316	362	410	462	516	634	768
P/A 3.5 N/mm² max	250	282	314	348	386	462	542
Class 2	320	348	376	406	450	544	644
2-span	254	290	326	364	404	488	578
Max 4 tendons in 300 ribs @1500 cc		328	362	390	450	590	768

SPAN:DEPTH CHART



Flat slabs *with edge beams*



Popular overseas for apartment blocks, office buildings, hospitals, hotels etc, where spans are similar in both directions. Economical for spans of 7 to 12 m. Square panels are most economical.

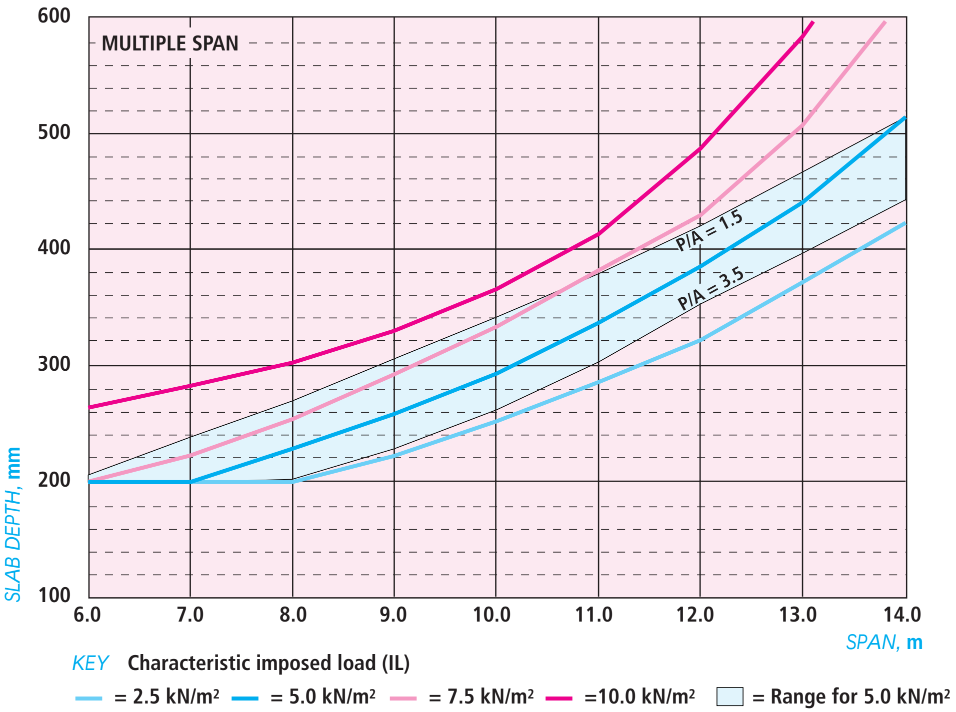
ADVANTAGES

- Simple, fast construction and formwork
- Architectural finish can be applied directly to the underside of the slab
- Minimum thickness and storey heights
- Controlled deflection and cracking
- Flexibility of partition location and horizontal service distribution

DISADVANTAGES

- Holes, especially large holes near columns, require planning
- Punching shear provision around columns may be considered to be a problem but can be offset by using larger columns, column heads, drop panels or proprietary systems. Post-tensioning improves shear capacity

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS internally and BEAMS around perimeter. Refer to charts and data to estimate sizes, etc.
DESIGN BASIS	To CS TR43. Balanced load 133% DL + 33% IL . Maximum prestress (P/A) = 2.5 N/mm². See Section 7. Effectively Class 2 assumed. No restraint to movement assumed.
DIMENSIONS	Square panels, assuming three spans by three bays. Outside edge flush with columns. Minimum column size as data. Edge beams should be at least 50% deeper than slab.
LOADS	SDL of 1.50 kN/m² (finishes) assumed. Perimeter load of 10 kN/m (14 kN/m ult) included in loads on edge beams. Ultimate loads to columns and beams are the result of moment distribution analysis.
TENDONS	Unbonded 15.7 mm diam. Superstrand (A_{ps} 150 mm², f_{pu} 1770 N/mm²), B1,T2, B2 & T3. Max 5 per m.
CONCRETE	C40, 24 kN/m³, 20 mm aggregate. f_{ci} = 25 N/mm².
REINFORCEMENT	Assumed min. T10@250T both ways at supports, min T12@500B both ways and T8 links. 10% allowed for wastage and laps.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure (25 mm cover to all).

MULTIPLE SPAN, m	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
THICKNESS, mm									
IL = 2.5 kN/m²	200	200	200	222	252	286	322	372	424
IL = 5.0 kN/m²	200	200	228	258	294	338	386	442	516
IL = 7.5 kN/m²	200	222	254	292	334	382	430	508	620
IL = 10.0 kN/m²	264	282	302	330	366	414	488	584	710
ULTIMATE LOAD TO SUPPORTING COLUMNS, MN, INTERNAL, PER STOREY.									
IL = 2.5 kN/m²	0.61	0.83	1.09	1.45	1.93	2.49	3.22	4.16	5.27
IL = 5.0 kN/m²	0.80	1.09	1.50	2.01	2.62	3.43	4.39	5.58	7.11
IL = 7.5 kN/m²	0.99	1.40	1.92	2.55	3.35	4.31	5.44	6.97	9.05
IL = 10.0 kN/m²	1.28	1.79	2.39	3.11	4.02	5.12	6.57	8.43	10.88
ULTIMATE LOADS ON EDGE BEAMS, kN/m									
IL = 2.5 kN/m²	48	54	59	68	78	89	102	119	137
IL = 5.0 kN/m²	59	66	77	89	101	118	135	155	181
IL = 7.5 kN/m²	70	81	94	109	126	144	164	191	227
IL = 10.0 kN/m²	86	100	114	130	148	169	196	229	270
REINFORCEMENT (TENDONS), kg/m²									
IL = 2.5 kN/m²	14 (8)	13 (9)	13 (9)	12 (10)	12 (11)	11 (12)	11 (13)	12 (13)	14 (13)
IL = 5.0 kN/m²	14 (8)	14 (9)	13 (10)	13 (11)	13 (13)	13 (13)	14 (13)	16 (13)	18 (13)
IL = 7.5 kN/m²	15 (8)	14 (10)	14 (11)	13 (13)	13 (13)	14 (13)	17 (13)	19 (13)	21 (13)
IL = 10.0 kN/m²	15 (7)	15 (9)	15 (12)	15 (13)	16 (13)	18 (13)	19 (13)	22 (13)	24 (13)
COLUMN SIZES ASSUMED, INTERNAL, mm square,									
IL = 2.5 kN/m²	280	330	380	440	500	570	650	740	830
IL = 5.0 kN/m²	320	370	440	510	580	660	750	840	950
IL = 7.5 kN/m²	350	410	480	560	640	730	820	920	1050
IL = 10.0 kN/m²	390	460	530	610	690	780	880	1000	1130
DESIGN NOTES									
IL = 2.5 kN/m²	p	o	os	ors	orS	orS	oRS	RS	RS
IL = 5.0 kN/m²	p	o	or	ors	ors	Rs	Rs	Rs	RS
IL = 7.5 kN/m²	p	or	or	or	R	Rs	Rs	Rs	Rs
IL = 10.0 kN/m²			r	r	R	Rs	R	R	R
LINKS, maximum number of perimeters (and percentage by weight of bonded reinforcement), no. (%)									
IL = 2.5 kN/m²	2 (0.6%)	3 (0.8%)	5 (1.4%)	6 (1.6%)	7 (1.8%)	7 (1.6%)	8 (1.9%)	8 (1.7%)	7 (1.1%)
IL = 5.0 kN/m²	3 (1.0%)	5 (1.7%)	6 (1.9%)	7 (2.0%)	7 (1.8%)	8 (2.1%)	8 (1.8%)	7 (1.1%)	6 (0.7%)
IL = 7.5 kN/m²	4 (1.5%)	6 (2.2%)	7 (2.4%)	7 (2.1%)	8 (2.3%)	8 (1.9%)	7 (1.2%)	6 (0.8%)	5 (0.5%)
IL = 10.0 kN/m²	3 (1.1%)	5 (1.7%)	6 (2.0%)	7 (2.1%)	7 (1.7%)	7 (1.4%)	6 (0.9%)	5 (0.6%)	4 (0.4%)

DESIGN NOTES *o = limited by P/A of 2.5 N/mm²* *p = 8 > response factor > 4* *q = shrinkage per span > 10 mm*
r = tendons @ < 300 mm cc. (R @ 200 mm cc.) *s = overall deflection, d_{xx} + d_{yy}, > 20 mm (S > 30 mm)*

VARIATIONS TO DESIGN ASSUMPTIONS: differences in slab thickness for a characteristic imposed load (IL) of 5.0 kN/m²

Fire resistance	2 hours	+0 mm	4 hours	+25 mm
Exposure	Moderate	+5 mm	Severe	+15 mm
Serviceability	Class 1	n/a	Column heads L/10 wide	-0 mm
Two spans	2 spans by 3 bays	see below	2 spans by 2 bays	see below
Rectangular bays	6.0 m wide bay	-15 mm @ 8 m and beyond	9.0 m wide bay	-15 mm @ 11.0 m and beyond

Thickness, mm	**Spans, m**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
	P/A 1.5 N/mm² max	270	306	342	380	422	468	516
	P/A 3.5 N/mm² max #	202	228	262	304	354	398	444
	2 spans by 3 bays	230	260	294	346	400	496	608
	2 spans by 2 bays	238	268	300	356	430	524	636
	T16@350B both ways	220	246	274	306	360	424	516

max 7 tendons/m

5.3 Post-tensioned beams

Rectangular 1000 mm wide

Prestressing beams can give great economic benefit for spans of 8 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 pro-rata.

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

ADVANTAGES

- Minimum thickness and storey heights
- Post-tensioning perceived to be a specialist operation

DISADVANTAGES

- Controlled deflection and cracking
- Tendon congestion

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	270	314	336	348	360	376	420	468	506
uaudl= 50 kN/m	274	318	366	416	468	524	584	644	708
uaudl= 100 kN/m	384	448	518	592	668	748	828	916	
uaudl= 200 kN/m	546	644	748	856					
ULTIMATE LOAD TO SUPPORTS, INTERNAL (END), PER METRE WEB WIDTH, kN/m									
uaudl= 25 kN/m	n/a (81)	n/a (101)	n/a (118)	n/a (135)	n/a (152)	n/a (170)	n/a (194)	n/a (221)	n/a (247)
uaudl= 50 kN/m	n/a (157)	n/a (189)	n/a (222)	n/a (258)	n/a (295)	n/a (335)	n/a (377)	n/a (422)	n/a (469)
uaudl= 100 kN/m	n/a (319)	n/a (379)	n/a (443)	n/a (509)	n/a (579)	n/a (651)	n/a (727)	n/a (806)	
uaudl= 200 kN/m	n/a (635)	n/a (752)	n/a (874)	n/a (999)					
REINFORCEMENT (TENDONS), kg/m^3									
uaudl= 25 kN/m	118 (34)	99 (33)	91 (35)	87 (35)	84 (35)	80 (35)	73 (35)	67 (35)	65 (35)
uaudl= 50 kN/m	115 (35)	98 (35)	86 (35)	77 (35)	71 (35)	66 (35)	84 (35)	77 (35)	71 (36)
uaudl= 100 kN/m	178 (35)	152 (35)	105 (35)	90 (35)	78 (35)	70 (35)	65 (35)	61 (35)	
uaudl= 200 kN/m	104 (35)	88 (35)	74 (35)	100 (35)					
DESIGN NOTES									
uaudl= 25 kN/m			o	o	o	o	ot8	ot9	oqt10
uaudl= 50 kN/m					t9	t10	t12	t13	t14
uaudl= 100 kN/m		t9	t10	t12	t13	nt15	nt17	nt18	
uaudl= 200 kN/m	t11	nt13	nt15	nt17					

n = designed shear links req'd *o = limited by prestress of 3.0 N/mm^2* *q = shrinkage >10 mm*

t-- = tendon congestion and no. of tendons required per m web width

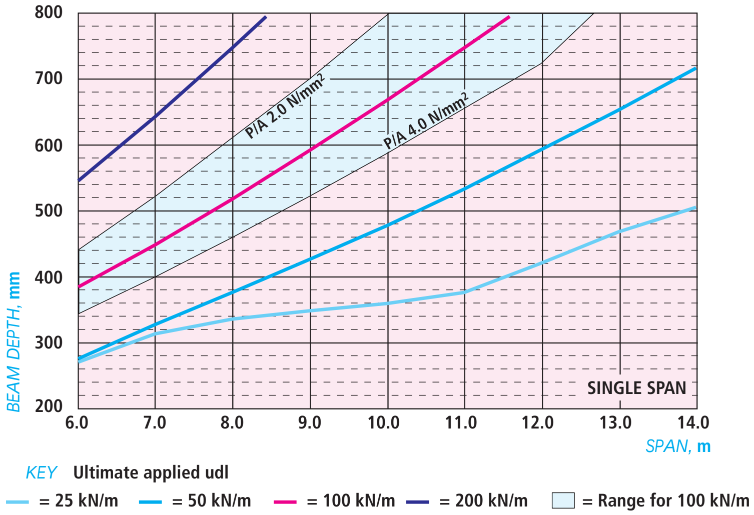
VARIATIONS TO DESIGN ASSUMPTIONS (see above): implications on beam depths for 100 kN/m uaudl

Fire resistance	2 hours	+10 mm	4 hours	+25 mm
Exposure	Moderate	+5 mm	Severe	+10 mm
Serviceability	Class 1& P/A = 4.0 N/mm^2	+20 mm up to 11 m	Class 2	+40 mm
	IL/DL = 1.25	+0 mm		

Depth, mm	**Span, m**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
	P/A 2.0 N/mm^2 max	612	700	796	900			
	P/A 4.0 N/mm^2 max	460	522	588	656	724	832	

Bonded tendons	Flat-4 multistrand, approx.	+80 mm up to 10 m	Round-7 multistrand	+80 mm up to 10 m
	Flat-4 multistrand & 4.0 N/mm^2	-10 mm up to 10 m		

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS
DESIGN BASIS	To CS TR43. Assuming $G_k = Q_k$ and balanced load of 133% DL + 33% LL. Maximum prestress (P/A) = 3.0 N/mm². Class 3 and no restraint assumed. See Section 7.
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. Ultimate loads for multiple spans are the result of moment distribution analysis for 3 spans.
TENDONS	Unbonded 15.7 mm diam Superstrand (A_{ps} 150 mm², f_{pu} 1770 N/mm²) B2 & T2. Bonded tendons (multiple strands in ducts) should be considered, and, indeed are necessary, where close centres are indicated. For the same level of prestress bonded tendons require more depth. See Section 7.
CONCRETE	C40, 24 kN/m³, 20 mm aggregate. f_{ci} = 25 N/mm².
REINFORCEMENT	f_y = 460 N/mm². Assumed 25 mm T1 for mesh, bars, etc., min T16@250B and T10 links.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure (25 mm cover to all).

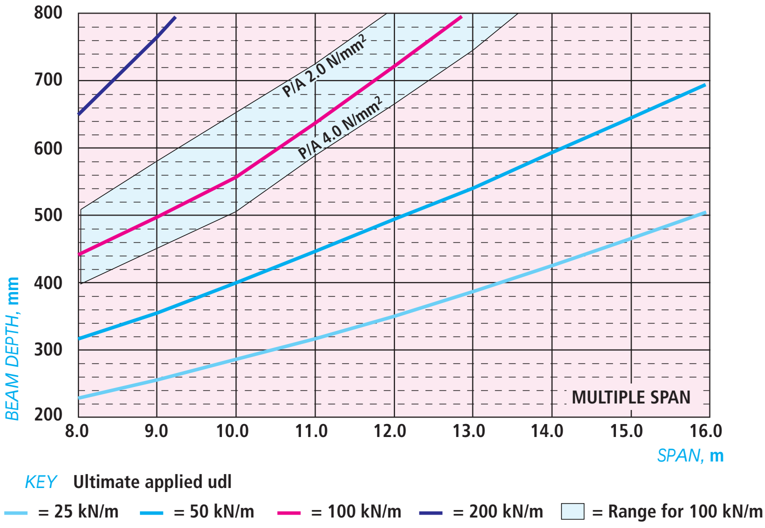
MULTIPLE SPAN, m	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**	**15.0**	**16.0**
DEPTH, mm									
uaudl= 25 kN/m	226	254	284	314	348	386	424	464	506
uaudl= 50 kN/m	314	354	398	444	492	540	592	644	696
uaudl= 100 kN/m	440	496	556	636	720	808	904		
uaudl= 200 kN/m	648	764	892						
ULTIMATE LOAD TO SUPPORTS, INTERNAL (END), PER METRE WEB WIDTH, kN/m									
uaudl= 25 kN/m	238 (92)	278 (107)	320 (123)	365 (140)	414 (158)	467 (179)	524 (200)	584 (223)	649 (247)
uaudl= 50 kN/m	496 (190)	571 (219)	652 (250)	736 (282)	826 (316)	918 (351)	1017 (389)	1120 (428)	1227 (468)
uaudl= 100 kN/m	995 (382)	1139 (437)	1288 (494)	1451 (556)	1622 (620)	1801 (688)	1992 (760)		
uaudl= 200 kN/m	1980 (759)	2268 (868)	2570 (983)						
REINFORCEMENT (TENDONS), kg/m²									
uaudl= 25 kN/m	220 (28)	174 (28)	150 (28)	125 (28)	113 (28)	105 (28)	98 (28)	92 (28)	87 (28)
uaudl= 50 kN/m	127 (28)	113 (28)	101 (28)	93 (28)	92 (28)	87 (28)	82 (28)	78 (28)	83 (28)
uaudl= 100 kN/m	143 (28)	110 (28)	102 (28)	93 (27)	85 (27)	80 (27)	77 (27)		
uaudl= 200 kN/m	103 (27)	94 (27)	86 (27)						
DESIGN NOTES									
uaudl= 25 kN/m	nop	nop	no	no	no	no	noqt8	noqt9	noqt10
uaudl= 50 kN/m	no	no	no	not9	not10	not11	noqt12	noqt13	noqt14
uaudl= 100 kN/m	not9	not10	not11	not13	not14	nt16	nt18		
uaudl= 200 kN/m	nt13	nt15	nt18						

n = designed shear links req'd *o = limited by prestress of 3.0 N/mm²* *q = shrinkage per span >10 mm*
t-- = tendon congestion and no. of tendons required per m web width

VARIATIONS TO DESIGN ASSUMPTIONS (see above): implications on beam depths for 100 kN/m uaudl

Fire resistance	2 hours		+10 mm			4 hours		+30 mm
Exposure	Moderate		+5 mm			Severe		+10 mm
Serviceability	Class 1		n/a			Class 2		+175 mm
Depth, mm	**Spans, m**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
	P/A 2.0 N/mm² max	508	580	652	724	800	880	964
	P/A 4.0 N/mm² max #	396	450	504	588	664	744	# IL/DL=0.8
	IL/DL = 1.25	440	514	592	672	760	852	956
	2 spans	470	528	592	656	720	788	856
Bonded tendons	Flat-4 multistrand		+30 mm approx		Round-7 multistrand			+50 mm approx

SPAN:DEPTH CHART



‘T’ beams 2400 mm wide web

Wide, shallow, post-tensioned multiple-span ‘T’ beams maximize the benefits of minimum construction depths, minimum deflections and less theoretical cracking. Economical for spans of 8 to 16 m.

The charts and data assume the use of single-strand unbonded tendons. However, in practice, bonded multi-strand tendons in flat or round ducts are more likely to be used. This will lead to increases in depth.

ADVANTAGES

- Minimum thickness and storey heights
- Controlled deflection and cracking

DISADVANTAGES

- Post-tensioning perceived to be a specialist operation
- Tendon congestion

SINGLE SPAN, m	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**	**15.0**	**16.0**
DEPTH, mm									
uaudl= 50 kN/m	220	240	268	304	346	386	426	470	516
uaudl= 100 kN/m	310	356	404	452	502	554	612	688	780
uaudl= 200 kN/m	444	502	560	624	716	864			
uaudl= 400 kN/m	636	760	948						
ULTIMATE LOAD TO SUPPORTS, INTERNAL (END), MN									
uaudl= 50 kN/m	n/a (0.21)	n/a (0.24)	n/a (0.28)	n/a (0.32)	n/a (0.37)	n/a (0.42)	n/a (0.48)	n/a (0.54)	n/a (0.60)
uaudl= 100 kN/m	n/a (0.43)	n/a (0.51)	n/a (0.58)	n/a (0.66)	n/a (0.75)	n/a (0.84)	n/a (0.93)	n/a (1.05)	n/a (1.15)
uaudl= 200 kN/m	n/a (0.88)	n/a (1.01)	n/a (1.14)	n/a (1.29)	n/a (1.45)	n/a (1.65)			
uaudl= 400 kN/m	n/a (1.74)	n/a (2.00)	n/a (2.30)						
REINFORCEMENT (TENDONS), kg/m^3									
uaudl= 50 kN/m	109 (46)	106 (46)	98 (46)	90 (46)	66 (46)	60 (46)	55 (45)	50 (45)	46 (45)
uaudl= 100 kN/m	91 (43)	66 (43)	59 (43)	53 (43)	49 (43)	56 (42)	50 (42)	44 (39)	45 (36)
uaudl= 200 kN/m	85 (41)	70 (41)	60 (41)	53 (41)	48 (37)	40 (31)			
uaudl= 400 kN/m	54 (39)	46 (35)	39 (28)						

DESIGN NOTES *n = designed shear links req’d o = limited by prestress of 3.0 N/mm^2 q = shrinkage >10 mm*

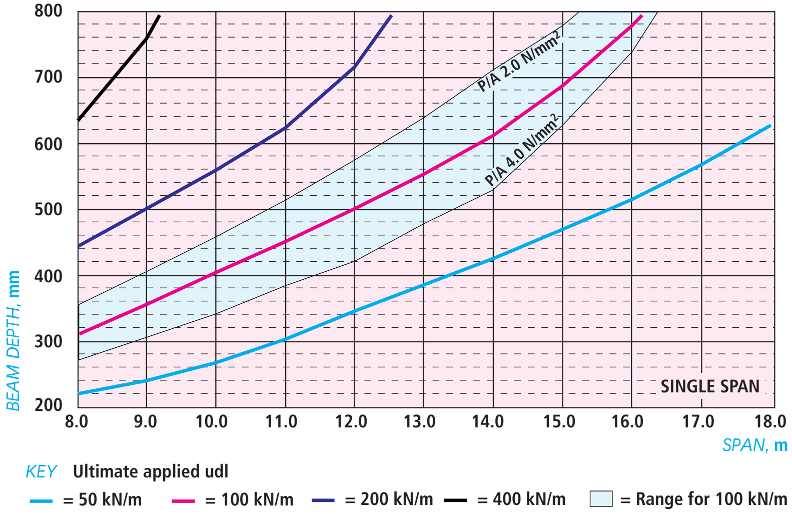
t-- = tendon congestion and no. of tendons required per m web width

	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
uaudl= 50 kN/m	o	op	o	o	ot21	ot24	oqt26	oqt29	oqt31
uaudl= 100 kN/m	o	ot21	ot23	ot26	ot29	ot32	oqt35	qt36	qt36
uaudl= 200 kN/m	t24	t28	t31	nt34	nt36	t36			
uaudl= 400 kN/m	nt34	nt36	nt36						

VARIATIONS TO DESIGN ASSUMPTIONS (see above): implications on beam depths for 100 kN/m uaudl

Fire resistance	2 hours		+10 mm			4 hours		+40 mm
Exposure	Moderate		+5 mm			Severe		+10 mm
Serviceability	Class 1& P/A = 4.0 N/mm^2		+15 mm			Class 2		+20 mm
Depth, mm	**Span, m**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
	P/A 2.0 N/mm^2 max	356	406	458	514	576	640	712
	P/A 4.0 N/mm^2 max	272	306	342	384	422	478	530
Bonded tendons	Flat-4 multistrand	+25 mm approx				Round-7 multistrand		+50 mm approx

SPAN:DEPTH CHART



DESIGN ASSUMPTIONS

SUPPORTED BY	COLUMNS
DESIGN BASIS	To CS TR43. Assuming $G_k = Q_k$ and balanced load of 133% DL + 33% LL. Maximum prestress (P/A) = 3.0 N/mm^2. Class 3 and no restraint assumed. See Section 7.
LOADS	Applied imposed load £ applied dead loads. Ultimate loads for multiple spans are the result of moment distribution analysis for 3 spans.
TENDONS	Unbonded 15.7 mm diam Superstrand (A_{ps} 150 mm^2, f_{pu} 1770 N/mm^2) B2 & T2. Bonded tendons (multiple strands in ducts) should be considered, and, indeed are necessary, where close centres are indicated. For the same level of prestress bonded tendons require more depth. See Section 7.
CONCRETE	C40, 24 kN/m^3, 20 mm aggregate. f_{ci} = 25 N/mm^2
REINFORCEMENT	f_y = 460 N/mm^2. Assumed 25 mm T1 for mesh, bars, etc., min T16@250B and T10 links.
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure (25 mm cover to all).

MULTIPLE SPAN, m	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**	**15.0**	**16.0**
DEPTH, mm									
uaudl= 50 kN/m	220	220	244	268	294	324	362	398	436
uaudl= 100 kN/m	276	310	350	390	434	478	524	570	620
uaudl= 200 kN/m	390	444	500	556	616	682	754	820	888
uaudl= 400 kN/m	554	626	706	796	886				
ULTIMATE LOAD TO SUPPORTS, INTERNAL (END), MN									
uaudl= 50 kN/m	0.5 (0.2)	0.5 (0.2)	0.6 (0.2)	0.7 (0.3)	0.8 (0.3)	0.9 (0.3)	1.0 (0.4)	1.1 (0.4)	1.3 (0.5)
uaudl= 100 kN/m	1.0 (0.4)	1.1 (0.4)	1.3 (0.5)	1.5 (0.6)	1.6 (0.6)	1.8 (0.7)	2.0 (0.8)	2.2 (0.9)	2.5 (0.9)
uaudl= 200 kN/m	2.0 (0.8)	2.3 (0.9)	2.6 (1.0)	2.9 (1.1)	3.2 (1.2)	3.6 (1.4)	3.9 (1.5)	4.3 (1.6)	4.7 (1.8)
uaudl= 400 kN/m	3.9 (1.5)	4.5 (1.7)	5.1 (1.9)	5.7 (2.2)	6.3 (2.4)				
REINFORCEMENT (TENDONS), kg/m^3									
uaudl= 50 kN/m	167 (32)	167 (32)	143 (32)	125 (32)	113 (32)	98 (32)	88 (32)	82 (32)	77 (32)
uaudl= 100 kN/m	125 (30)	108 (30)	88 (30)	81 (30)	75 (30)	69 (30)	67 (30)	66 (30)	64 (30)
uaudl= 200 kN/m	111 (29)	92 (29)	82 (29)	75 (29)	71 (29)	67 (29)	64 (29)	63 (28)	62 (26)
uaudl= 400 kN/m	88 (28)	83 (28)	79 (28)	74 (28)	70 (26)				

DESIGN NOTES n = designed shear links req'd o = limited by prestress of 3.0 N/mm^2 q = shrinkage per span >10 mm
v = reinforcement added, B, for ultimate load case t-- = tendon congestion and no. of tendons required per 2.4 m width

	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
uaudl= 50 kN/m	no	no	no	no	no	nov	noqvt20	noqvt22	noqvt24
uaudl= 100 kN/m	no	no	nov	novt21	novt23	novt25	noqvt27	noqvt30	noqvt32
uaudl= 200 kN/m	novt20	novt22	novt25	novt28	novt31	novt34	novt38	noqvt40	nqvt40
uaudl= 400 kN/m	not27	novt31	novt35	novt39	nvt40				

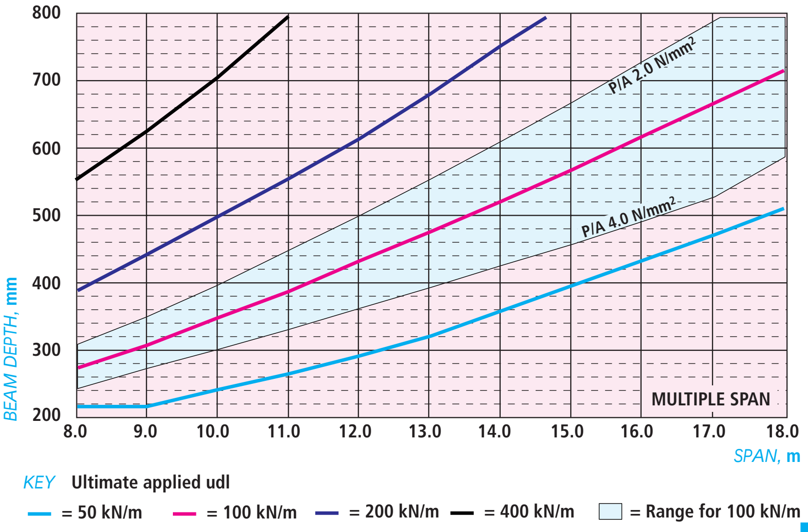
VARIATIONS TO DESIGN ASSUMPTIONS (see above): implications on beam depths for 100 kN/m uaudl

Fire resistance	2 hours	+10 mm	4 hours	+25 mm
Exposure	Moderate	+5 mm	Severe	+10 mm
Serviceability	Class 1	n/a	Class 2	+25 mm
ULS reinforcement	No additional	+0 mm to +40 mm @ 14 m	Unlimited	-0 mm to -30 mm @ 14 m
Two span	Two spans	+10 mm @ 8 m to -25 mm @ 14 m	IL/DL = 1.25	+0 mm to +20 mm @ 14 m

Depth, mm	**Spans, m**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**
	P/A 2.0 N/mm^2 max	312	352	398	450	502	556	612
	P/A 4.0 N/mm^2 max	246	276	304	334	364	396	428

Bonded tendons Flat-4 multistrand +15 mm @ 8 m reducing Round-7 multistrand +40 mm @ 8 m reducing

SPAN:DEPTH CHART



6.1 Walls

Reinforced concrete walls not only take vertical load, but they also very often provide lateral stability to a structure. Whilst this publication is not intended to cover stability, the design of such walls is considered here briefly.

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 (induced by adjoining floors, etc).

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.

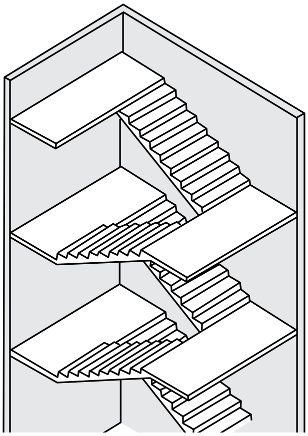
For an element to be considered as a wall, the breadth (b) must be at least four times the thickness (h). To be considered as being reinforced, a wall must have at least 0.004bh of high yield reinforcement in the vertical direction and 0.0025bh of high yield reinforcement horizontally.

Slender walls should be avoided, ie. the ratio of their effective height to thickness should be less than 15. From BS 8110 Pt 1 Cl 3.8.1.6, effective height factors for braced columns/walls are given as:

Condition 1 at both ends . . .
walls connected monolithically to slabs either side that are at least as deep as the wall, or connected to a foundation able to carry moment . . . 0.75

Condition 2 at both ends . . .
walls connected monolithically to slabs either side that are shallower but at least half as deep as the wall . . . 0.85

Condition 3 at both ends . . .
walls connected to members that provide no more than nominal restraint to rotation . . . 1.00



A factor of 0.85 is commonly used for conceptual design of in-situ walls. In practice these requirements usually result in the use of 200 mm thick cantilever walls in low-rise multi-storey buildings. The walls are dispersed around the plan and, as far as possible, located in cores and stair areas. The vertical load capacities of walls, with minimum quantities of reinforcement, are usually adequate in these 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.

With these caveats in mind the information in the table below is given for guidance only.

Thickness mm	Maximum height,m						Capacity		Reinforcement		
	Effective height factor						$A_{sreq'd}$	Capacity[a]	Typical arrangements		Densities[b]
	0.75	0.80	0.85	0.90	0.95	1.00	mm^2/m	kN/m	Vertical	Horizontal	kg/m^2(kg/m^3)
150	3.00	2.81	2.65	2.50	2.37	2.25	600	2020	T10@200 bs	T10@250 bs	13 (86)
175	3.50	3.28	3.09	2.92	2.76	2.63	700	2360	T10@200 bs	T10@200 bs	13 (76)
200#	4.00	3.75	3.53	3.33	3.16	3.00	800	2700	T12@200 bs	T10@250 bs	16 (80)

Notes: a capacities for $A_{sreq'd}$ assume nominal eccentricity only
b includes 20% for laps and wastage, etc.

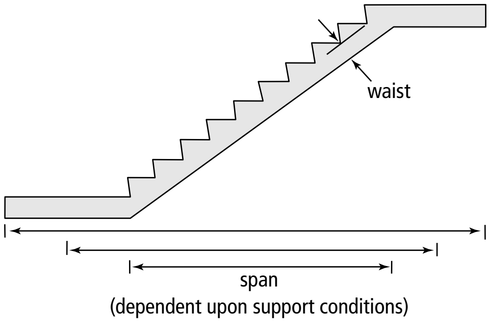
preferred thickness
bs both sides

6.2 Stairs

There are many possible configurations of stair flights, landings and supports. The charts and data consider parallel flights as illustrated opposite.

In-situ spans may be considered as being simply supported or continuous – depending upon the amount of continuity available. Precast flights are usually considered as simply supported. Landings are treated as solid slabs.

In-situ stairs provide robustness, mouldability and continuity of work for formworkers. Precast stairs provide quality, speed of construction and early access.



DESIGN ASSUMPTIONS

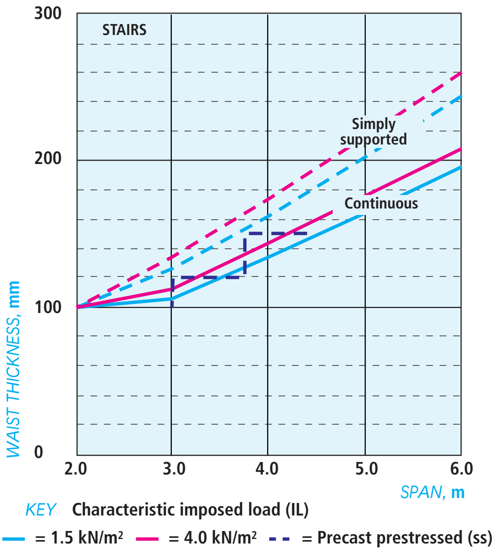
SUPPORTED BY	BEAMS, WALLS or LANDINGS.
REINFORCEMENT	T16. T10 @ 300 distribution. 10% allowed for wastage and laps.
DIMENSIONS	Flight assumed to be 60% of span. Going 250 mm, rise 180 mm.
LOADS	Superimposed load (SDL) of 1.50 kN/m^2 (for finishes, services, etc.) included. Ultimate loads assume elastic reaction factors of 0.5 to supports of single spans, 1.1 and 0.46 to supports of continuous spans.
IMPOSED LOADS	1.5 kN/m^2 - self-contained dwellings; 4.0 kN/m^2 - hotels, offices, institutional buildings, etc.
CONCRETE	C35, 24 kN/m^3, 20 mm aggregate
FIRE & DURABILITY	Fire resistance 1 hour; mild exposure.

	SINGLE SPANS, m				**MULTIPLE SPANS, m**			
SPANS, m	**2.0**	**3.0**	**4.0**	**5.0**	**2.0**	**3.0**	**4.0**	**5.0**
WAIST THICKNESS, mm								
IL = 1.5 kN/m^2	100	126	162	202	100	106	134	164
IL = 4.0 kN/m^2	100	134	174	216	100	112	144	176
ULTIMATE LOAD TO INTERNAL (END) SUPPORTS, kN/m					*(Equivalent to a ultimate applied udl to landing)*			
IL = 1.5 kN/m^2	n/a (10)	n/a (17)	n/a (25)	n/a (36)	21 (9)	34 (16)	51 (23)	70 (32)
IL = 4.0 kN/m^2	n/a (14)	n/a (23)	n/a (34)	n/a (47)	30 (13)	48 (22)	70 (32)	95 (43)
REINFORCEMENT, kg/m^2								
IL = 1.5 kN/m^2	16	20	24	27	9	12	15	18
IL = 4.0 kN/m^2	18	24	26	30	11	15	17	20

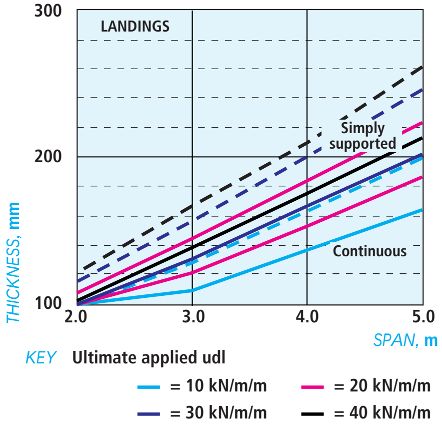
VARIATIONS TO DESIGN ASSUMPTIONS: differences in waist thickness for a characteristic imposed load (IL) of 4.0 kN/m^2

Fire resistance	2 hours, single span	+20 mm	2 hours, multiple span	+5 mm
Exposure	Moderate (ss and ms)	+12 mm	Severe (ss and ms), C40 concrete	+25 mm

Precast prestressed	**Span (ss), m**	**3.00**	**3.75**	**4.40**
	Waist mm	100	120	150



LANDINGS (chart only)



Reinforcement approximately 20 to 30 kg/m^2 extra over flight reinforcement.

7 DERIVATION OF CHARTS AND DATA

7.1 In-situ elements

7.1.1 GENERAL

For a given load and span, slabs (or beams) can be designed at different depths. Thinner slabs have proportionally more reinforcement, but require less concrete, less perimeter cladding and 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. There is a minimum overall cost which can be identified by designing an element at different depths and pricing the resulting quantities using budget rates and comparing totals. In order to derive the charts and data in this publication, this process was automated using computer spreadsheets.

For a particular span and load, elements were designed in accordance with BS 8110 Pt. 1 (up to and including Amendment 4)(2) and Pt 2 (up to and including Amendment 1)(3). Unit rates were applied to the required quantities of concrete, reinforcement and formwork. Allowances were made for perimeter cladding and supporting self-weight. The resulting budget costs were summed and the most economic valid depth identified, as illustrated by the chart below.

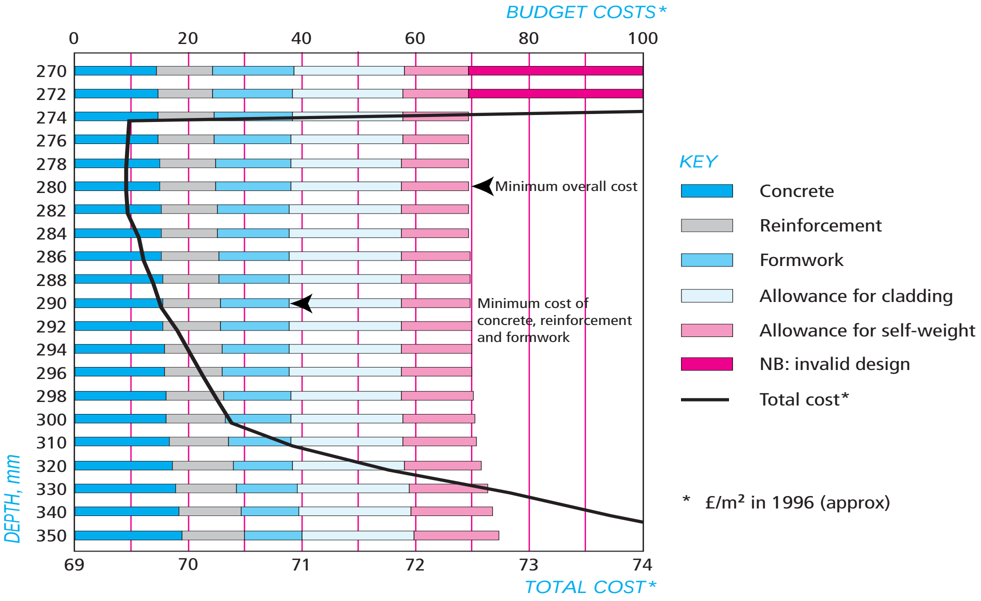
The example relates to the RCC's *Cost Model Study* (6) M4C3 building. This used solid flat slabs on a 7.5 m square grid, with 5.0 kN/m² imposed load, 1.5 kN/m² superimposed dead load and a 10 kN/m allowance for cladding. A thickness of 280 mm would appear to give best overall value. The data for a 280 mm 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. The charts and data therefore represent optimum depths over a range of common spans and loadings using the methods and assumptions described.

The budget rates used in the optimization were as follows:

Concrete C35	£54.00 /m³
Horizontal formwork (plain)	£18.60 /m²
Horizontal formwork (ribbed)	£22.50 /m²
Vertical formwork	£22.50 /m²
Cladding	£275.00 /m²
Main reinforcement	£400.00 /tonne
Links	£500.00 /tonne
Post-tensioning tendons	£2000.00 /tonne
Allowance for self-weight	£0.75 /kN

Origin of data: example showing how most economic sizes were identified



These rates, apart from post-tensioning tendons, are taken from the RCC's *Cost Model Study*, which was published in 1993. The rates have dated and will undoubtedly date further. However, the optimization process used in the derivation of the charts is not sensitive to actual rates and is not too sensitive to 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).

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 *Cost Model Study* buildings and is based on the difference in supporting three storeys rather than seven storeys in terms of £/kN. The foundations were 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.

Construction durations and differences attributable to different types of construction tend to be project specific and are difficult to model. Time costs, therefore, were not taken into account in the optimization process.

7.1.2 DESIGN ASSUMPTIONS

Unless noted otherwise, the charts assume:

The use of BS 8110(2) moment and shear factors (tables 3.6 and 3.13)

End spans are critical

The use of C35 concrete (f_{cu} = 35 N/mm^2) and high yield steel (f_y = 460 N/mm^2)

Mild exposure conditions and 1 hour fire resistance

Concrete density of 24 kN/m^3

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. Whenever appropriate, reference was made to relevant texts(12, 13, 14, 15).

Moments and shears factors given in BS 8110, Pt 1(2) tables 3.6 and 3.13 were used. More sophisticated analysis may be appropriate during more detailed design at a later stage of the design process.

The charts and data for multiple spans assume a minimum of three spans. Theoretically, to maintain a common 20% redistribution of support moments, two-span slab elements should be subject to greater support moment and shear coefficients than those given in table 3.13 of BS 8110. Nonetheless, the sizes given in the charts and data can be used for two-span slab elements unless support moment or shear is considered critical. In this case two-span slabs should be justified by analysis and design.

In many cases, particularly with slabs, deflection is critical to design. In such instances additional tension reinforcement was provided to reduce service stress, f_s, and increase the modification factor for tension reinforcement (see BS 8110, table 3.11). A modification factor allowing for small amounts of compression reinforcement was used in the determination of flat slab and beam depths.

As lightweight concretes are not always readily available, they were considered to be inappropriate for this publication. Nonetheless, they might be an ideal solution for a particular project.

7.1.3 SLAB CHARTS AND DATA

Slab charts give overall depths against spans for a range of **characteristic** imposed loads assuming end spans. An allowance of 1.5 kN/m^2 has been made for superimposed dead loads (finishes, services, etc). For two-way slab systems (ie. flat slabs, troughed slabs and waffle slabs designed as two-way slabs with integral beams), an allowance of 10 kN/m has been made around perimeters to allow for the self-weight of cladding.

As BS 8110, Pt 1, Cl 3.5.2.4, the charts and data are valid where:

In a one-way slab the area of a bay (one span x full width) exceeds 30 m^2

The ratio of characteristic imposed loads, q_k, to characteristic dead loads, g_k, does not exceed 1.25

The characteristic imposed load, q_k, does not exceed 5 kN/m^2, excluding partitions

Additionally, for flat slabs, there are at least three rows of panels of approximately equal span in the direction being considered.

If design parameters stray outside these limits, the sizes and data given should be used with caution.

In general, slabs were assumed to have simple end supports, ie. an ultimate bending moment factor of 0.086 was used. For flat slabs, continuous end supports were assumed, but the end support moment was restricted to M_{tmax} with possible consequential increase in span moments.

Reinforcement densities assume that the areas or volumes of slabs are measured gross, eg. slabs are measured through beams and the presence of voids in ribbed 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 8.2.1) and web widths. For multiple spans, 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. As BS 8110, Pt 1, Cl 3.4.3, the charts and data are valid where:

Characteristic imposed loads, Q_k, do not exceed characteristic dead loads, G_k

Loads are substantially uniformly distributed over three or more spans

Variations in span length do not exceed 15% of the longest span

Where the charts stray outside these limits, the sizes and data given should be used with caution.

In the optimisation process there were slight differences in the allowances for cladding and the self-weight of beams compared with slabs. The allowance for perimeter cladding was applied only to 'T' (ie. internal) beams greater than 500 mm deep: the assumption made is that shallower internal beams, perimeter inverted 'L' beams and rectangular beams would not affect storey heights. For the purposes of self-weight, the first 200 mm depth of beam was ignored: it was assumed that the applied load included the self-weight of a 200 mm solid slab.

Different design criteria can be critical across the range of beams described. The sizes given in the charts and tables are at least 20 mm deeper than for an invalid design using BS 8110 table 3.6 for analysis. The critical criteria are given under *Design notes* in Section 3.2.4.

Particular attention is drawn to the need to check that there is adequate room for reinforcement bearing at end supports. End support/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 the end support/column size is based on edge columns with 2.5% 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 3.12.9.4 of BS 8110).

Beam reinforcement densities relate to web width multiplied by overall depth.

7.1.5 COLUMN CHARTS

The column charts give square sizes against **ultimate** axial loads for a range of steel contents for **braced** internal, edge and corner columns. Column design is dependant on both ultimate axial load and ultimate design moments. In recognition of the different amounts of moment likely to be experienced by the columns, internal, edge and external corner columns are treated separately. Design moments depend on spans, loads and stiffnesses of members and are specific to a column or group of columns. Whilst the allowance made for moments is considered to be conservative, it is uncertain. The sizes given, particularly for perimeter columns, are, therefore, **estimates** only.

All data were derived from spreadsheets that designed square braced columns supporting solid flat slabs. Forces were derived in accordance with BS 8110, Pt 1, Cl 3.8.2.3; and applied moments in perimeter columns in accordance with Cl 3.2.1.2.3. Many different configurations were used: 2 to 10 storeys, panel aspect ratios (l_y/l_x) of 1.00, 1.25, 1.5 and 1.75 etc. In general, the slabs were assumed to carry 5.0 kN/m^2 imposed load, 1.0 kN/m^2 superimposed dead load, and 8.5 kN/m perimeter load (3.0 kN/m at roof level). Floor-to-floor height was set at 3.6 m and β for columns, 0.85. Checks were carried out over a limited range of aspect ratios assuming different imposed loads, different perimeter loads and different types of slab (troughed floors and one-way slab and beams).

Internal columns

Internal column sizes are based on 'an allowable stress', p_c, where:

$$p_c = 0.384 \times f_{cu} + 3.6 \times f_y \times (As/100)/460.$$

The extensive trials suggested an accuracy of ±12 mm in square column size. The charts and data will be less accurate if unequal adjacent spans and/or imposed loads higher than 5 kN/m^2 are used or if other than nominal moment is envisaged.

Perimeter columns

The charts were derived from the design of square braced columns as described above: the largest square column size from the range of panel aspect ratios is quoted. As relatively flexible flat slabs were used in the derivation, these sizes should, in general, prove conservative. However, they may not be so when less stiff floor plates or very lightweight cladding is used.

In order to model design moments simply, the charts and data are presented in terms of ultimate axial load and number of storeys supported.

Comparisons of the charts with the base data suggested that the square sizes given are reasonably accurate. They appear to be an average of 12 mm (sd 25 mm) greater than those required for the desired percentage of reinforcement for the worst panel aspect ratio. Suggested sizes are less accurate for one- and two-storey columns, floor or beam spans greater than 12 m, and floor panel aspect ratios greater than 1.50.

Concrete grade

The use of concrete strengths greater than the 35 N/mm^2 concrete assumed can decrease the sizes of column required. Smaller columns occupy less lettable space. However, this publication is aimed at low-rise buildings where buildability issues (eg. different mixes on site, punching shear and reinforcement curtailment requirements) minimize potential gains. Also, in the range considered, the use of column concrete strengths greater than 35 N/mm^2 appears to make little difference to the size of perimeter column required. Higher strength columns are therefore not covered in this publication, but should be considered, particularly on high-rise projects.

Reinforcement percentages

Reinforcement percentages assume 3.6 m storey heights and 37 diameters + 100 mm laps.

7.2 Precast and composite elements

7.2.1 SLABS

The charts and data for proprietary precast and composite elements are based on manufacturers' 1996 data. The sizes given are selected, wherever possible, from those offered in late 1996 by at least two manufacturers. The ultimate loads to supporting beams are derived from the maximum self-weight quoted for the relevant size.

The units are designed to BS 8110, generally using grade C50 concrete, high tensile strand or wire prestressing steel to BS 5896 or high tensile steel to BS 4449. For specific applications the reader should refer to manufacturers' current literature.

Precast and in-situ concrete can act together to give efficient, economical and quick composite sections. For slabs, these benefits are exploited in the range of composite floors available. The data have been abstracted from manufacturers' literature.

7.2.2 COMPOSITE BEAMS

For composite beams the position is not so clear cut. During the construction of a composite beam (precast downstands acting with an in-situ topping), the precast element will usually require temporary propping until the in-situ part has gained sufficient strength. The number of variables (construction stage loading, span, propped span, age at loading, etc.) 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.

7.2.3 PRECAST BEAMS

The charts and data in this publication therefore concentrate on unpropped non-composite beams. They cover a range of profiles, web widths and **ultimate** applied uniformly distributed loads (uaudl).

These charts were derived from spreadsheets using the same optimisation process as in-situ beams. The design of precast beams was based on ordinary reinforced concrete design principles as covered in BS 8110(2) and *Multi-storey precast concrete framed structures* (9). The single spans were measured from centreline of support to centreline of support. For 'L' and inverted 'T' beams, a ledge width of 125 mm was assumed. Upstanding concrete is therefore relatively wide and, for structural purposes, was considered part of the section. In-situ concrete infill was ignored. The depths of beams were minimized consistent with allowing suitable depth for precast floor elements.

The main complication with precast beams is the connections. The type of connection is usually specific to individual manufacturers and can affect the beams. The sizes of beams given should therefore be considered as indicative only. Other aspects, notably, connection design and details, other components, columns, floors, walls, stairs, stability, structural integrity and overall economy can influence final beam sizing.

Manufacturers produce a wide range of preferred cross-sections based on 50 mm increments. Designs with other cross-sections are easily accommodated. The economics of precast beams depend on repetition: a major cost is the manufacture of the base moulds. Reinforcement is usually part of an overall package and, therefore, densities are not quoted (but they tend to be high). For specific applications, the reader should refer to manufacturers and their current literature.

7.2.4 COLUMNS

These charts were derived from spreadsheets using the same optimization process as that described for in-situ columns. The design of precast columns is based on ordinary reinforced concrete design principles as covered in BS 8110. Column design is dependant upon axial load and design moment induced. The charts and data for internal columns assume equal spans in each direction (ie. $l_{x1} = l_{x2}$ and $l_{y1} = l_{y2}$) and, therefore, nominal moments.

The charts and data for edge and corner columns are presented in terms of ultimate axial load, and, in order to model design moments simply, number of storeys. They have been derived by assuming that the floor reaction acts at a nominal eccentricity of Œ column size + 150 mm.

Grade 50 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 allow much greater percentages of reinforcement to be used. This is acknowledged in BS 8110, which allows reinforcement areas of up to 8%. However, connection details can limit the amounts of reinforcement that can be used. The charts for perimeter columns, therefore, concentrate on relatively small amounts of reinforcement. 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 8110(2) and Concrete Society Technical Report No 43(10). Reference was made to other material (11,16) as required. The effects of columns and restraint were ignored in the analysis and design.

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 load balanced or prestress assumed can be varied to make many depths work.

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 levels of prestress (eg. 3.0, 4.0 and 5.0 N/mm^2) 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 become increasingly dominant. Theoretical economies have to be balanced against issues of buildability and serviceability. The charts and data in this publication are, therefore, based on more typical mid-range levels of prestress, 2.5 N/mm^2 for slabs and 3.0 N/mm^2 for beams. The charts give an indication of the range of depth for higher and lower levels of prestress. Higher levels of pre-stress may be appropriate in certain circumstances. 2.5 N/mm^2 might be considered high for flat slabs.

The shape of the lines for the span:depth charts for prestressed elements is the product of a number of slopes (in order of increasing slope - vibration limitations, load balanced, 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 predominate. At shorter spans and lower loads, it is the amount of load balanced that is critical. The amounts of load that were used to balance loads were:

Solid slabs
100% dead load 25% imposed load

Ribbed slabs, flat slabs and beams
133% dead load 33% imposed load

The charts and data assume the use of single-strand unbonded tendons. Where these become congested, consideration should be given to using bonded multi-strand tendons in flat or round ducts. The use of bonded tendons in ducts will alter assumptions made regarding cover, drapes, wobble factors, coefficient of friction, construction methods etc. and, without increasing assumed prestress, will increase depths. For beams, indications of increased depths using bonded flat-4 and round-7 multi-strand tendons are given.

The charts for multiple spans are based on a three-span condition. Normally, at the serviceability limit state for a multiple span, the two-span condition would be assumed to give the maximum moment (at support). However, preventing post-tensioned multi-span elements rising at internal supports causes secondary moments in the elements. These moments are usually beneficial to support moments and detrimental to span moments to the extent that ultimate three-span span moments (including ultimate secondary moments) are generally more critical than serviceability two-span support moments (or, indeed, ultimate or serviceability four-span span or support moments). The three-span case has therefore been used.

Special care must be taken, however, with one-way slabs over 12 m and flat slabs, where the two-span condition appears to be more critical than the three-span condition. The depths of highly loaded two-span rectangular beams may also need minor adjustment. Please refer to relevant data.

BS 8110 allows for three serviceability classes: class 1 allows no flexural tensile stresses, class 2 allows flexural tensile stresses but no visible cracking, and class 3 allows flexural tensile stresses with cracks limited to 0.2 mm (0.1 mm in severe environments). Most elements in buildings are assumed to be in an internal environment, and are designed to serviceability class 3. The charts are therefore based on class 3. (The allowable crack width in the design of untensioned bonded reinforcement is 0.3 mm.)

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 six 15.7 mm diameter tendons per rib. The weight of (untensioned) reinforcement allows for nominal links to support the tendons, but does not allow for mesh, eg. A142, in the topping. Where four or fewer tendons are used (and apart from 2 and 4 hours fire resistance and severe exposure), the sizes are 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 regarding allowable tensile stresses determined the use of serviceability class 2 design. The inclusion of untensioned bonded reinforcement 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.

7.3.4 BEAMS - RATIO OF DEAD LOAD TO LIVE LOAD

The charts and data ‘work’ on applied ultimate load. However, in multiple spans, the ratio of imposed load to dead load can alter span moments, and a ratio of 1.0 (ie. applied imposed load = applied dead load) was assumed.

Lower ratios, with dead loads predominating, make little difference to the sizes advocated. For a higher ratio of 1.25 (imposed:dead, eg. a 300 mm ribbed slab, average 4.5 kN/m^2, supporting 1.5 kN/m^2 SDL and 7.5 kN/m^2 IL), guidance is given. Still higher ratios can induce mid-span hogging and might be dealt with by assuming the beam depth tends towards being 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 Concrete Society Technical Report No 43, and used the load balancing method of design. Moments and shears were derived from moment distribution analysis. Both tensioned and untensioned reinforcement were designed and allowance was made for distribution steel and reinforcement around anchorages. Designs were subject to limiting amount of prestress and number of tendons. Generally, service moments were critical.

Deflection checks were based on uncracked concrete sections and limited to span/250 overall and span/500 or 20 mm after the application of finishes. Vibration was considered using the Concrete Society Technical Report 43 method of analysis assuming three bays 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).

The following data was used in the preparation of the charts:

Bonded reinforcement

f_y = 460 N/mm^2

Tendons

15.7 mm diameter unbonded tendons, A_p = 150 mm^2

f_{pu} = 1770 N/mm^2

Transfer losses = 10%

Service losses = 20%

Coefficient of friction, μ = 0.06

Wobble factor, ω = 0.019 rads/m

Relaxation = 2.5%

Relaxation factor = 1.5%

Young’s modulus, E_{ps} = 195 kN/mm^2

Sheath thickness = 1.5 mm

P_{Ap} =150 kN approx.

Inflection of tendon at 0.1 of span.

Wedge draw-in = 6mm

Whilst Superstrand tendons were used in the derivation of the charts and data, other tendons, eg. Dyform strand, may prove to be just as, or more, economic.

Concrete

Properties at transfer: characteristic compressive strength, $f_{ci,}$ = 25.0 N/mm^2, Young’s modulus, E_{ci}, = 21.7 kN/mm^2.

Indoor exposure; Coefficient of drying shrinkage, e_{sh}, = 300 microstrain.

Creep coefficients, ϕ, for loads applied after 7 days, 2.0; after 1 month, 1.8 and after 6 months, 1.2.

8.1 Slabs

The slab charts and data give overall depths, etc. against span for a range of **characteristic** imposed loads assuming end spans and a superimposed dead load (finishes, services, etc) of 1.5 kN/m². In order to use the slab charts and data as intended, it is essential that the correct characteristic imposed load is used (if necessary modified to account for different superimposed dead loads).

8.1.1 IMPOSED LOADS, q_{ks}

The imposed load should be determined from the intended use of the building (see BS 6399 Pt 1(5)). 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.

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
4.0 kN/m²	Corridors, high-specification office loading, shop floors
5.0 kN/m²	High-specification office loading, file rooms, areas of assembly
7.5 kN/m²	Plant rooms
2.4 kN/m²/m	General storage per metre height
4.0 kN/m²/m	Stationery stores per metre height

The slab charts highlight:

2.5 kN/m²	General office loading, car parking
5.0 kN/m²	High-specification office loading, file rooms, areas of assembly
7.5 kN/m²	Plant room and storage loadings
10.0 kN/m²	Storage loadings

In addition, an allowance of 1.0 kN/m² should be considered for demountable partitions in office buildings. A common specification is '4 + 1', ie. 4.0 kN/m² imposed load plus 1.0 kN/m² for demountable partitions. No reductions in imposed load have been made (BS 6399 Pt 1 tables 2 and 3) nor are provisions for concentrated loads considered.

8.1.2 SUPERIMPOSED DEAD LOADS (SDL), g_{ksdl}

Superimposed dead loads allow for the weight of services, finishes, etc. The IStructE/ICE publication, *Manual for the design of reinforced concrete building structures*(12), recommends that allowances for dead loads on plan should be generous and not less than those shown in the opposite column.

Floor finish (screed)	1.8 kN/m²
Ceilings & services load	0.5 kN/m²
Demountable partitions	1.0 kN/m²
Blockwork partitions	2.5 kN/m²

Raised access flooring imparts loads of up to approximately 0.5 kN/m² and suspended ceilings weigh up to approximately 0.15 kN/m². BS 648(17) schedules the weight of building materials. It can be used to derive the following typical characteristic loads:

Carpet	0.03 kN/m²
Terrazzo tiles, 25.4mm	0.52 kN/m²
Screed, 1:3, 50mm	1.15 kN/m²
Gypsum plaster, 12.7 mm	0.21 kN/m²
Gypsum plasterboard, 12.7 mm	0.11 kN/m²

Examples of typical build-ups are given below:

Offices

Carpet	0.03 kN/m²
Screed, 1:3 (50 mm)	1.15 kN/m²
Gypsum plaster ceiling,12.7 mm	0.21 kN/m²
Services	0.11 kN/m²
	1.50 kN/m²

Speculative offices

Carpet tiles	0.03 kN/m²
Raised access floor	0.50 kN/m²
Suspended ceiling	0.15 kN/m²
Services	0.32 kN/m²
	1.00 kN/m²

Core areas

Terrazzo tiles, 25.4 mm	0.52 kN/m²
Screed, 1:3, 75 mm	1.75 kN/m²
Gypsum plaster, 12.7 mm	0.21 kN/m²
Blockwork partitions#	2.50 kN/m²
Services	0.22 kN/m²
	5.20 kN/m²

\# BS 6399 allows one to take „ of the line load from partitions as a uniformly distributed load. In this case, say, 3.25 m high 150 mm thick dense blockwork @ 1.90 kN/m² plus gypsum plaster 12.7 mm both sides @ 0.42 kN/m²

8.1.3 SUPERIMPOSED DEAD LOADS, g_{ksdl}: IMPOSED LOADS (IL) FOR USE WITH SLAB CHARTS AND DATA

The charts and data make an allowance of 1.50 kN/m² for superimposed dead loading (SDL). If the actual superimposed dead load differs from 1.50 kN/m², the characteristic imposed load used for interrogating the charts and data should be adjusted by adding 1.4/1.6 x (actual SDL - 1.50) kN/m². The equivalent characteristic imposed load can be estimated from the table opposite.

Equivalent imposed loads, kN/m²

Imposed load kN/m²	Superimposed dead load, kN/m² 0.0	1.0	2.0	3.0	4.0	5.0
2.5	1.2	2.1	2.9	3.8	4.7	5.6
5.0	3.7	4.6	5.4	6.3	7.2	8.1
7.5	6.2	7.1	7.9	8.8	9.7	10.6
10.0	8.7	9.6	10.4	11.3	12.2	n/a

8.1.4 SELF-WEIGHTS OF SLABS, g_{ks}

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, as in the case of post-tensioned beams, it may be necessary to know the proportion of dead load to imposed load. All slab charts and data include allowances for self-weight at a density of 24 kN/m²

The following self-weights are indicative. Values for ribbed and waffle slabs may differ, depending upon mould manufacture. Values for precast slabs also may differ between manufacturers.

Characteristic self-weight of slabs, g_{ks}, kN/m²

Slab thickness, mm	100	200	300	400	500	600
Solid slabs[1]	2.4	4.8	7.2	9.6	12.0	
Ribbed slabs[2]						
100% ribbed			3.5	4.1	4.8	5.5
75% ribbed, 25% solid			4.4	5.5	6.6	7.7
Waffle slabs[3]						
100% waffle			4.0	5.0	6.2	7.6
75% waffle, 25% solid			4.8	6.2	7.7	9.3
Slab thickness, mm	110	150	200	250	300	400
Hollow-core slabs without topping	2.2	2.4	2.9	3.7	4.1	4.7
Slab thickness, mm	150	190	240	290	340	440
Hollow-core slabs with 40 mm topping[4]	3.2	3.4	3.9	4.7	5.1	5.7
Slab thickness, mm	325	425	525	625	725	825
Double 'T's without topping[5]	2.6	2.9	3.3	3.7	4.1	4.5
Slab thickness, mm	400	500	600	700	800	900
Double 'T's with 75 mm topping[6]		4.4	4.7	5.1	5.5	6.3

Notes

1 including in-situ, precast and composite solid slabs
2 bespoke moulds, 150 mm ribs at 750 mm cc, 100 mm topping
3 bespoke moulds, 125 mm ribs at 900 mm cc, 100 mm topping
4 for slabs with 50 mm structural topping, add 0.2 kN/m²
5 for slabs 300, 400, 500 mm, etc. thick, deduct 0.6 kN/m²
6 for slabs with 100 mm topping, add 0.6 kN/m²

8.1.5 ULTIMATE SLAB LOAD, n_s

Ultimate loads are summations of characteristic loads multiplied by appropriate partial load factors, ie:

n_s = ultimate self-weight of slab, $g_{ks} \times \gamma_{fgk}$
+
ultimate superimposed dead loads, $g_{ksdl} \times \gamma_{fgk}$
+
ultimate imposed load, $q_{ks} \times \gamma_{fgk}$

where

g_{ks}, g_{ksdl} and q_{ks} are as explained above and measured in kN/m²

γ_{fgk} = load factor for dead loads = 1.4

$\gamma_{fqk=}$= load factor for dead loads = 1.6

Example

What is the ultimate load of a 300 mm solid slab supporting 1.5 kN/m² superimposed dead loads and 5.0 kN/m² imposed load?

n_s = 7.2 × 1.4 + 1.5 × 1.4 + 5.0 × 1.6
= 20.46 kN/m²

8.2 Beams

8.2.1 CALCULATING ULTIMATE APPLIED UNIFORMLY DISTRIBUTED LOADS (uaudl) TO BEAM, n_b

The beam charts give overall depths against span for a range of **ultimate** applied loads and web widths, assuming end spans. This load can be calculated as follows:

Ultimate applied udl to beam,

n_b = ultimate applied load from slabs, $n_s \times l_s \times$ erf
+
ultimate line loads, n_{ll}

8.2.2 ULTIMATE APPLIED LOAD FROM SLABS, $n_s \times l_s \times$ erf

Ultimate applied load from slabs should be calculated by multiplying the following terms:

$n_s \times l_s \times$ erf

where

ns ultimate slab load, kN/m2, as described above.

ls = slab span perpendicular to the beam, m. In the case of multiple-span slabs, take the average of the two spans perpendicular to the beam.

erf = elastic reaction factor =

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 interior supports of multiple-span continuous slabs (eg. in-situ slabs) or for all interior supports of discontinuous slabs (eg. precast slabs)

1.1 for the first interior supports of continuous slabs of three or more spans

1.2 for the internal support of continuous slabs of two spans

Adjustments for elastic reactions

The data for slabs include ultimate applied loads from slabs to beams. These figures may need to be adjusted to account for actual conditions, eg. for an in-situ slab of two spans rather than that for the three spans assumed, consider increasing loads to beams by 1.2/1.1, ie. approximately 10%. NB: data for post-tensioned slabs is the result of analysis and therefore includes elastic reactions.

8.2.3 ULTIMATE LINE LOADS, n_{ll}

Ultimate line load,

n_{ll} = ultimate cladding loads, $g_{kc} \times \gamma_{fgk} \times h$

+

other ultimate line loads, $g_{ko} \times \gamma_{fgk}$

+

adjustment for ultimate beam self-weight, $g_{kbm} \times \gamma_{fgk}$

where

g_{kc} = characteristic dead load of cladding, kN/m^2, see opposite

h = supported height of cladding

g_{ko} = characteristic dead load of other line loads, kN/m

g_{kbm} = characteristic dead load, kN/m. Beam self-weight is allowed for in the charts but the user may wish to make adjustments.

γ_{fgk} = partial safety factor for dead load, 1.4

Ultimate cladding loads, $g_{kc} \times h \times \gamma_{fgk}$

Ultimate cladding loads should be determined by multiplying characteristic cladding loads by the partial load factor and supported height. Cladding loads can be estimated from the following tables.

Ultimate applied load from cladding, $g_{kc} \times h \times \gamma_{fgk}$, kN/m

Char. cladding load, g_{kc},kN/m^2	Height supported (eg. floor to floor), 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	2	3	3	3
1.0	3	4	4	4	4	5	5	5	6
1.5	5	5	6	6	7	7	8	8	8
2.0	7	7	8	8	9	10	10	11	11
2.5	8	9	10	11	11	12	13	13	14
3.0	10	11	12	13	13	14	15	16	17
3.5	12	13	14	15	16	17	18	19	20
4.0	13	15	16	17	18	19	20	21	22
4.5	15	16	18	19	20	21	23	24	25
5.0	17	18	20	21	22	24	25	27	28

Typical characteristic cladding loads, g_{kc}

	kN/m^2
102.5 mm brickwork	
solid high-density clay	2.34
solid medium-density clay	2.17
15% voids high-density clay	1.95
concrete	2.30
150 mm solid blockwork	
stone aggregate	3.20
lightweight aggregate	1.90
aerated (560 kg/m^3)	0.85
aerated (800 kg/m^3)	1.13
150 mm cellular blockwork	
stone aggregate	2.35
lightweight aggregate	1.67
12 mm plaster	
gypsum, two coat	0.21
lightweight, 2-coat vermiculite	0.11
2 no. x 6 mm double glazing	
c/w aluminium framing	0.35
2 no. x 8 mm curtain wall glazing	
c/w aluminium framing	0.50
Precast concrete cladding	
average 100 mm thick	2.40
Profiled metal cladding	0.15
20 mm drylining on studwork	0.15
50 mm insulation	0.02

Example

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 = $\underline{0.21}$ kN/m^2

Subtotal = 3.70 kN/m^2

2 no. × 6 mm double glazing c/w framing = 0.35 kN/m^2

Assuming minimum 25% glazing, average =
75% × 3.70 + 25% × 0.35 = 2.86 kNm^2

Determine load/m

Assuming the height of cladding to be supported is 3.5 m then, the characteristic load per metre run =
2.86 × 3.5 = 10 kN/m^2

and the ultimate load per metre run =
10 × 1.4 = 14 kN/m

Ultimate line loads from other sources, $g_{ko} \times \gamma_{fgk}$

Any other applied loads on a particular beam must be determined. For example, characteristic partition loads:

150 mm blockwork, solid, stone aggregate = 3.20 kN/m^2
2 no. × 12 mm plaster, gypsum, two coat = 0.42 kN/m^2
Total = 3.62 kN/m^2

If the height of cladding to be supported is 3.0 m then ultimate cladding load, $g_{kp} \times h \times \gamma f_{gk} =$

$3.62 \times 3.0 \times 1.4 = 15$ kN/m

The ultimate applied load from partitions can be determined from characteristic loads and supported heights from the tables opposite.

Adjustment for self-weight of beam, $g_{kb} \times \gamma_{fgk}$

The beam charts assume that in-situ slab loads are imparted by a 200 mm thick solid slab. Where the slab is not 200 mm thick some adjustment can be made as follows:

Additional ultimate load per metre width of beam web, kN/m/m

Depth of slab, mm	Internal 'T' beams	Perimeter 'L' beams
100	3	2
200	0	0
300	-3	-2
400	-7	-3
500	-10	-5

Example

Determine the ultimate applied load to a 300 mm wide perimeter beam supporting a 250 mm one-way solid slab, IL 5.0 kN/m², SDL 1.5 kN/m², spanning 6.0 m, and 3.5 m of cladding, average 3.0 kN/m².

Ultimate slab load, kN/m^2.

$n_s = (6.0 + 1.5) \times 1.4 + 5.0 \times 1.6$
$= 18.5$ kN/m^2

Ultimate applied load from slabs, $n_s \times l_s \times \text{erf} =$
$18.5 \times 6.0 \times 0.5 = 55.5$ kN/m

Ultimate line load from cladding =
$3.5 \times 3.0 \times 1.4 = 14.7$ kN/m

Adjustment for self-weight of beam, =
$(0.25 - 0.20) \times 0.30/2 \times 24 \times 1.4 = -0.2$ kN/m

Total, ie. ultimate applied udl to beam, $n_b = 70.0$ kN/m

8.2.4 BEAMS SUPPORTING TWO-WAY SLABS

The loads outlined in the two-way slab data are derived in accordance with BS 8110 assuming square corner panels and assuming that these loads will be treated as uniformly distributed loads over 75% of the beam span. Treating the load as though it were applied to 100% of the beam span overestimates the moment by approximately 5%, making little practical difference for the purposes of sizing beams.

For non-square panels, it is suggested that the loads on the longer supporting beams should be determined from the loads for a square panel of the longer dimension. Using this load over 100% of the beam's span overestimates the span moment by an additional amount dependant on the slab panel aspect ratio:

Aspect ratio	1.00	1.25	1.33	1.50	2.00
Overestimate on moment	0%	6%	9%	15%	32%

Assuming that deflection is proportional to moment, these percentages can be used to modify the loads used in determining the beam sizes. The user may or may not choose to use this approximate method.

Example

What loads should be used in sizing the internal beams supporting bespoke waffle slabs designed as two-way slabs (SDL 1.5 kN/m², IL 5.0 kN/m²) on a 13.5 by 9.0 m grid?

For the 9.0 m span, from p 31 (bespoke moulds, multiple span, 9.0 m span, 5.0 kN/m^2) load to internal beam
= 108 kN/m

Allow 5% for overestimate of moment due to using load over 100% of length of beam 108/1.05
108/1.05 = 103 kN/m

For the 13.5 m span, from p 31 (bespoke moulds, multiple span, 13.5 m span, 5.0 kN/m^2) load to internal beam
= 197 kN/m

Allow 5% for overestimate of moment due to using load over 100% of length and 15% for overestimate of moment due to overestimating load for an aspect ratio of 1.5. Therefore, for the purposes of sizing beam only use:
197/(1.05×1.15) = 163 kN/m

8.2.5 POST-TENSIONED BEAMS

The first set of charts for post-tensioned beams assume 1000 mm wide rectangular beams. Other post-tensioned beam widths can be investigated on a pro-rata basis, ie. by determining the ultimate applied uniformly distributed load (uaudl) per metre width of web. The following table may help.

Equivalent uaudl per metre width of web, kN/m width/m run

		Beam width, mm						
		300	450	600	900	1200	1800	2400
Ultimate applied uniformly distributed load (uaudl) per metre run, kN/m	25	83	56	42				
	50	167	111	83	56	42		
	75	250	167	125	83	63	42	31
	100	333	222	167	111	83	56	42
	150		333	250	167	125	83	63
	200			333	222	167	111	83
	250				278	208	139	104
	300				333	250	167	125

8.3 Columns

8.3.1 CALCULATING ULTIMATE AXIAL LOAD, N

In design calculations, it is usual to determine the **characteristic** loads on a column on a floor-by-floor basis, assuming simple supports (see BS 8110, Pt 1, Cl 3.8.2.3) and keeping dead and imposed loads separate. Load factors, γ_f, are applied to the summation of these loads to obtain **ultimate** loads used in the design. BS 6399(5) allows some reduction in imposed load depending on usage, area supported and number of storeys.

Hence, the ultimate axial load can be expressed as

$$N = \Sigma\{g_{ks} \times l_x \, x \, l_y + g_{kbx} \times l_x + g_{kby} \times l_x + g_{kc}\} \times \gamma_{fgk} + \Sigma\{q_{ks} \times l_x \times l_y\} \times Y_{fqkx} \times \text{ilrf}$$

where

$\Sigma\{....\}$ = summation from highest to lowest level

g_{ks} = characteristic slab self-weight and superimposed dead loads

g_{kbx} = characteristic extra over beam, cladding loads and any other dead loads supported

g_{kc} = characteristic self-weight of column

q_{ks} = characteristic imposed load for the slab

l_x = supported span in the × direction, taken to be half of the sum of the two adjacent spans (but see Section 8.3.2, elastic reaction factors, below)

l_y = supported span in the y direction, taken to be half of the sum of the two adjacent spans (but see Section 8.3.2, elastic reaction factors, below)

γ_{fgk} = partial safety factor for dead load, 1.4

γ_{fqk} = partial safety factor for imposed load, 1.6

ilrf = imposed load reduction factor

Imposed load reduction factors

In accordance with BS 6399 table 2, imposed loads may be reduced in accordance with the number of floors, including roof, being supported. Generally, live load reduction is unwarranted in the pre-scheme design of low-rise structures: a factor of 1.00 may be used

Imposed load reduction factors

No. of floors carried by member	1	2	3	4	5-10	10+
Reduction in imposed load in member	0	10%	20%	30%	40%	50%

8.3.2 ELASTIC REACTION FACTORS

To allow for the effects of continuity when calculating column loads, many engineers use elastic reactions or summation of ultimate shears rather than simply supported (single span) reactions of beams or slabs. According to BS 8110, Pt 1, Cl 3.8.2.3, this precaution is unnecessary - simple supports may be assumed.

However, if required to avoid anomalies with more rigorous analysis or to reflect serviceability foundation loads more accurately, beam or slab loads to columns may be increased. The amount by which beam loads are increased depends on the circumstance (see Section 8.2.2 and BS 8110 tables 3.6 and 3.13) and engineering judgement. Often an increase of 10% (1.1/1.0) is used for penultimate columns supporting a beam of three or more spans. In the case of two-span beams an increase of 20% might be warranted. In the case of flat slabs, troughed slabs, etc. allowance might be made for each orthogonal direction.

8.3.3 ULTIMATE SELF-WEIGHT OF COLUMNS, kN

Ultimate self-weight of columns can be estimated from the following table

Ultimate self-weight of columns per storey, kN

		Height (eg. floor-to-floor),m								
		2.4	2.6	2.8	3.0	3.2	3.4	3.6	3.8	4.0
	250	5	5	6	6	7	7	8	#8	#8
Size, mm square	300	7	8	8	9	10	10	11	11	12
	400	13	14	15	16	17	18	19	20	22
	500	20	22	24	25	27	29	30	32	34
	600	29	31	34	36	39	41	44	46	48
	700	40	43	46	49	53	56	59	63	66
	800	52	56	60	65	69	73	77	82	86

slenderness may exceed 15, ie. may be a slender column in a braced frame.

8.3.4 ESTIMATING ULTIMATE AXIAL LOAD

See Section 2.7.

8.3.5 EXAMPLES

See Sections 2.11.4 and 2.11.5.

9 THE CASE FOR CONCRETE

9.1 General

Primarily, clients expect three things from building structures -

- low cost of construction
- short construction times
- excellent functional performance and quality.

Concrete frames fit the bill.

9.2 Costs

Construction costs

In comparison with steel frames, reinforced concrete can

- save up to 24% in frame costs
- save 5.5% in overall construction costs(6)

Finance costs

All other things being equal, concrete construction's 'pay as you pour' principle saves on finance costs. This could amount to saving 0.3% of overall construction cost compared with structural steel-framed buildings.

Thermal mass

Concrete's thermal mass tends to reduce excessive diurnal temperature fluctuations and causes a useful delay between peak external and peak internal temperatures. It can therefore, reduce cooling requirements in buildings, thereby reducing both initial and running costs of services. Concrete can be formed into appropriate shapes to aid the transfer of heat from circulating air to the structure.

Foundations

Foundations for concrete-framed buildings may cost up to 30% more than those for steel-framed buildings. However, this is more than compensated by up to 24% saving in superstructure costs(6). Superstructures cost 5 to 15 times as much as foundations.

Fees

The advent of fixed fees has tended to eliminate traditional additional engineers' fees for the detailing of reinforced concrete. Now however, reinforced concrete detailing is considered an additional service under the 1995 ACE *Conditions of engagement*. Fees for consultants are a small proportion of total costs, but their work has a great effect on buildability, functionality and value.

Specialist concrete contractors, notably members of *Construct*, are able to offer contractor detailing. Contractor detailing can offer many benefits. These include lower overall costs, faster construction, less adversarial relationships, increased buildability, more opportunity to innovate and to control safety within the requirements of the design.

9.3 Time

Speed

Overall, in-situ concrete-framed buildings generally take no longer to construct than steel-framed buildings: indeed they can be faster(6).

Perceptions about fast steel-frame construction must be balanced against the availability of suitable areas for follow-on trades. With no secondary application of fireproofing, and apart from propping of in-situ frames, concrete construction gives follow-on trades the opportunity of working on completed floors. Enlightened specifications and a willingness to adopt specialist contractors' methods, where appropriate, can have a remarkable effect on concrete construction programmes.

Buildability

The prerequisites for fast construction in any material are design discipline, repetition, integration, simplification and standardization of design details. Rationalising reinforcement, designing and detailing for prefabrication, precasting or part-precasting are some of the techniques that can help progress on site.

Many contractors appreciate the opportunity to discuss buildability and influence designs for construction.

Forms of contract

Construction management and design-and-build forms of contract are becoming more popular. Lack of lead-in times and concrete's ability to accommodate late information and variations are especially useful under these forms of contract (as the work can be let without finalising the design of following elements).

Weather

Cold and hot weather working need some preparation and planning. Precautions should be taken to ensure that progress is not impeded by rain or snow.

Striking times and propping

Striking times and propping are a part of traditional in-situ concrete construction. When critical to programme, contractors, with the co-operation of designers, can mitigate their effects.

Late changes

By its nature, concrete allows alteration at a very late stage. It is important that this attribute is not abused or productivity will suffer.

9.4 Performance

Quality

Quality requires proper motivation and committed management from the outset. Success is dependant on the use of skilled and motivated personnel and quality materials. Overspecification is both costly and wasteful.

Accuracy

Overall accuracy of concrete framed buildings is not markedly different from other forms of construction. BS 5606(18) 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.

Lettable areas

Concrete-framed buildings can give up to 1.5% more net lettable area than steel-framed buildings(6). This is due to the flexibility of concrete construction, the dual use of structural concrete walls as partitions (and not needing to allow for steel bracing zones) and fewer stair treads due to lower floor-to-floor heights.

Adaptability

Like no other construction material, concrete can deal with complex geometry. Concrete structures are amenable to many alteration techniques and adaptability can be designed in. Ribbed floor construction gives obvious soft spots for later holes with minimal disruption.

Service integration

Flat soffits allow simple, flexible service routes to access all parts of a floor. Forming openings for risers is relatively easy, although the size of openings adjoining columns in flat slabs may be restricted.

Deflections

Generally, deflections are not large.

Long spans

The chart on p 8 gives many examples of reinforced concrete floors and many options for spans greater than 12 m. Beyond about 7.5 m, prestressing or post-tensioning becomes economic, particularly if construction depth is critical. Traditional reservations about post-tensioning are very often misconceived.

Vibration

Except for extremely thin slabs, vibration is imperceptible.

Stability

In low- to medium-rise buildings, it is most economic to use the inherent moment-resisting frame action of the slab (and beams) and columns. Otherwise, discrete cantilever shear walls should be used around permanent openings such as lifts and stairs.

Corrosion

Corrosion is a problem only in concrete in external or damp environments. Provided that prescribed covers to reinforcement are achieved, and the concrete is of appropriate quality, concrete structures should have no corrosion problems.

Fire protection

Concrete provides inherent fire resistance.

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10.3 Abbreviations

1/rb	curvature at mid-span
A_{ps}	area of prestressing steel reinforcement
A_s	area of steel reinforcement
B1, B2	bottom layers, B1 = lowest layer (excluding links), B2 = second layer from bottom
C35	grade 35 concrete
cc	centres
DL	dead load (characteristic uno.), eg. for slabs self-weight + superimposed dead load
E	elastic modulus (Young's modulus)
erf	elastic reaction factor
f_{pu}	characteristic yield strength of prestressing steel reinforcement.
f_y	characteristic yield strength of reinforcement.
I	inertia
IL	imposed load (characteristic uno.)
m	metre
mm	millimetre
n	ultimate load per unit area or length
N	total ultimate load
n/a	not applicable
o/a	overall
P/A	load per unit area - a measure of prestress
R8, etc.	8 mm diameter, mild steel reinforcement, f_y = 250 N/mm^2, etc
SDL	superimposed dead load - allowance for services and finishes, or that part of dead loads that are not self-weight
T10 etc.	10 mm diameter, high yield reinforcement, fy = 460 N/mm2, etc
T1, T2	top layers, T1 = top layer (excluding links), T2 = second layer from top
uaudl	ultimate applied uniformly distributed load
uno.	unless noted otherwise

10.4 Organisations

Initials	Name	Telephone	Fax
BCA	British Cement Association	(01344) 762 676	(01344) 761 214
BPCF	British Precast Concrete Federation	(0116) 253 6161	(0116) 251 4568
Construct	Concrete Structures Group	(01344) 725 744	(01344) 761 214
CS	Concrete Society	(01753) 693 313	(01753) 692 333
PFF	Precast Flooring Federation	(0116) 253 6161	(0116) 251 4568
PTA	Post-tensioning Association	(0113) 270 1221	(0113) 276 0138
RCB	Ready-mixed Concrete Bureau	(01344) 725 732	(01344) 761 214
RCC	Reinforced Concrete Council	(01344) 725 733	(01344) 761 214
SPA	Structural Precast Association	(0116) 253 6161	(0116) 251 4568

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