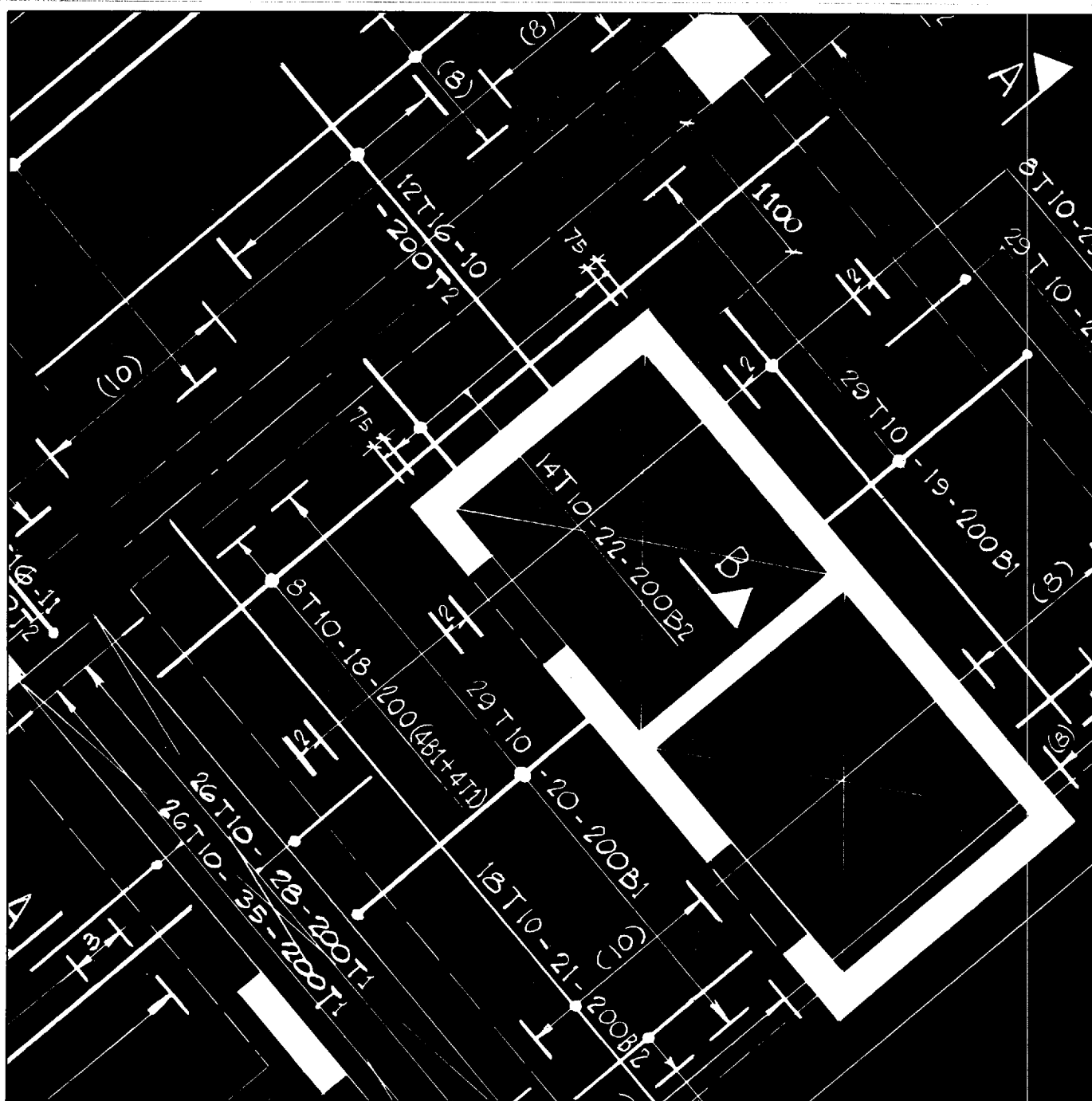


**J. B. Higgins and B. R. Rogers MA, CEng, MICE**



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# Designed and detailed (BS 8110 : 1997)

J. B. Higgins and B. R. Rogers MA, CEng, MICE

## Contents

- 2 Introduction
- 3 BS 8110 and limit state design
- 6 Design information
- 7 Structural summary sheet
- 8 Floor slab
- 10 First-floor main beam
- 16 Edge beam
- 18 Columns
- 22 Foundation
- 24 Shear wall
- 26 Staircase
- 28 Column design chart
- 29 Further information

## Foreword

This third edition of *Designed and detailed* has been revised to BS 8110 : Part 1: 1997, and the amendment dated 15 September 1998. Although there have been several amendments to the code since 1985, the latest and most significant change has been the reduction in the partial safety factor for reinforcement  $\gamma_m$  from 1.15 to 1.05. With higher stresses, less steel is required. However, the total saving may not be fully realised because there are other considerations such as choosing a practical arrangement of bars, and the deflection in the case of shallower members.

The calculations have also been revised for the loading requirements of BS 6399 : Part 1: 1996 and Part 2: 1995.

Design charts in BS 8110 : Part 3: 1985 may still be used to provide a conservative solution, and one of these charts has been included for the design of columns. Lap lengths for these members have also been taken from BS 8110, Table 3.27, but adjusted for the design stress of  $0.87f_y$ .

The tie reinforcement for robustness is designed at its characteristic strength. If the characteristic bond stress is used for calculating laps and anchorage lengths, then the values in Table 3.27 may be multiplied by  $1.05/1.4$ . This publication takes a conservative practical approach and uses directly the values given in Table 3.27.

Observant users of previous editions will appreciate the skill that is evident in the setting out of the calculations and the drawings. This is the work of the late Jim Higgins, whose care in the production of the original artwork was meticulous. Sadly, he never saw the second edition in print. I hope that my amendments to this third edition will not detract from his fine workmanship.

Special thanks are due to Tony Threlfall for his advice and suggestions for this edition.

**Railton Rogers**

# Introduction

The purpose of this publication is to apply the principles of limit state design given in BS 8110 by means of a simple worked example for a reinforced concrete building frame. The calculations and details are presented in a form suitable for design office purposes and are generally in accordance with the following publications.

BRITISH STANDARDS INSTITUTION. *Structural use of concrete. Part 1. Code of practice for design and construction*. Milton Keynes, BSI. 1997. 120 pp. BS 8110 : Part 1: 1997.

H M STATIONERY OFFICE. *Building and buildings. The Building Regulations 1991* (Amended 1994). HMSO, London. 21 pp. Statutory Instruments No. 2768.

BRITISH STANDARDS INSTITUTION. *Loading for buildings. Part 1. Code of practice for dead and imposed loads*. Milton Keynes, BSI. 1996. 10 pp. BS 6399 : Part 1: 1996.

BRITISH STANDARDS INSTITUTION. *Loading for buildings. Part 2. Code of practice for wind loads*. Milton Keynes, BSI. 1995. 82 pp. BS 6399 : Part 2: 1995.

BRITISH STANDARDS INSTITUTION. *Loading for buildings. Part 3. Code of practice for imposed roof loads*. Milton Keynes, BSI. 1988. 23 pp. BS 6399 : Part 3: 1988.

BRITISH STANDARDS INSTITUTION. *Specification for scheduling, dimensioning, bending and cutting steel reinforcement for concrete*. Milton Keynes, BSI. 1989. 20 pp. BS 4466 : 1989.

THE CONCRETE SOCIETY. *Model procedure for the presentation of calculations*. London (now Slough). 1981. Technical Report 5, second edition. 18 pp.

THE CONCRETE SOCIETY AND THE INSTITUTION OF STRUCTURAL ENGINEERS. *Standard method of detailing structural concrete*. London, The Institution, 1989. 138 pp.

# BS 8110 and limit state design

## Objective

To serve its purpose, a structure must be safe against collapse and be serviceable in use. Calculations alone do not produce safe, serviceable and durable structures. Equally important are the suitability of the materials, quality control and supervision of the workmanship.

Limit state design admits that a structure may become unsatisfactory through a number of ways which all have to be considered independently against defined limits of satisfactory behaviour. It admits that there is an inherent variability in loads, materials and methods of design and construction which makes it impossible to achieve complete safety against any possible shortcoming. By providing sufficient margins of safety, the aim of limit state design is to provide an acceptable probability that the structure will perform satisfactorily during its intended life.

Limit states can be classified into two main groups:

- (1) the ultimate limit state, which is concerned with the provision of adequate safety;
- (2) the serviceability limit states, which are essentially concerned with durability.

Generally, in practice, there are three limit states which are normally considered for reinforced concrete and these are given in the Table below.

	Ultimate limit state	Serviceability limit states	
		Deflection	Cracking
Objective	Provision of adequate safety	Structure should not deflect so as to impair use of structure	Cracking should not be such as to damage finishes or otherwise impair usage
Loading regime	Design ultimate loads	Design service load	
Performance limit	Structure should not fail	Deflection should not exceed specified limits	Crack width should not exceed 0.3 mm generally

## Characteristic values

For the testing of materials, a statistical approach can be applied to the variations within materials which occur in practice. A normal or Gaussian distribution curve is assumed to represent the results of the tests and a value known as the characteristic value can be chosen below which not more than 5% of the test results may be expected to lie.

The characteristic strength is given by the equation:

Characteristic strength = Mean or Average strength –  $1.64 \times$  Standard deviation

Ideally, a characteristic load should be similarly defined, as a load with a 5% probability of being exceeded during the lifetime of the structure. However, it is not yet possible to express loading in statistical terms, so the Code uses the loads defined in BS 6399: Parts 1, 2 and 3.

## Design loads

The design load is given by the equation:

$$\text{Design load} = \text{Characteristic load} \times \gamma_f$$

where  $\gamma_f$  is a partial safety factor for loading. This factor takes into account the possibility that the loads acting on the structure may be greater than the characteristic values. It also takes into account the assumptions made in the method of analysis, and the seriousness of failure to meet the design criteria for a particular limit state. The consequence of collapse is much more serious than exceeding the serviceability limits and so this is reflected in the higher values of the partial safety factors. Components of load have to be considered in their most unfavourable combinations, so sets of values of  $\gamma_f$  for minimum and maximum design loads are required. For example, the worst situation for a structure being checked for overturning under the action of wind load will be where the maximum wind load is combined with the minimum vertical dead load. Lower values of  $\gamma_f$  are used for the combination of wind, imposed and dead loads than for the combinations of wind and dead, and dead and imposed loads, as the probability of three independent design loads achieving their maximum value at the same time is less. The table below gives the partial load factors for the ultimate limit state.

Combination of loads	Partial safety factor to be applied to				wind load
	dead load		imposed load		
	when effect of load is				
	adverse	beneficial	adverse	beneficial	
1 Dead and imposed	1.4	1.0	1.6	0	–
2 Dead and wind	1.4	1.0	–	–	1.4
3 Dead and wind with imposed	1.2	1.2	1.2	1.2	1.2

## Design strengths

The design strength is given by the equation:

$$\text{Design strength} = \frac{\text{Characteristic strength}}{\gamma_m}$$

where  $\gamma_m$  is a partial safety factor on the material strength. This factor takes into account the variation in workmanship and quality control that may normally be expected to occur in the manufacture of the materials. The values of  $\gamma_m$  to be used for the two materials when designing for the ultimate limit state are given below:

Values of $\gamma_m$ for the ultimate limit state	
<i>Reinforcement</i>	1.05
<i>Concrete</i>	
Flexure or axial load	1.5
Shear strength without shear reinforcement	1.25
Bond strength	1.4
Others (e.g. bearing stress)	$\geq 1.5$

## Robustness

In addition to providing a structure that is capable of carrying the design loads, the layout should be such that damage to small areas of a structure or failure of single elements will not lead to a major collapse.

The Code requires that in all buildings the structural members should be linked together in the following manner:

- (a) by effectively continuous peripheral ties at each floor and roof level;

- (b) by internal ties in two directions approximately at right-angles, effectively continuous throughout their length and anchored to the peripheral ties at each end (unless continuing as horizontal ties to columns or walls);
- (c) by external column and wall ties anchored or tied horizontally into the structure at each floor and roof level;
- (d) by continuous vertical ties from foundation to the roof level in all columns and walls carrying vertical loads.

In the design of the ties, the reinforcement may be assumed to be acting at its characteristic strength with no other forces present but the tie forces. Reinforcement provided for other purposes can often be used to form part or the whole of these ties, so that in the design process, when the required reinforcement for the usual dead, imposed and wind loading has been found, a check can be made to see whether modifications or additions to the reinforcement are required to fulfil the tie requirements.

## Durability and fire resistance

At the commencement of the design, the following should be considered:

- the climate and environmental conditions to which the concrete will be exposed;
- the concrete quality;
- the cover to the reinforcement.

It should also be noted that the quality of the construction process and the first hours after casting of the concrete have a major influence upon the subsequent durability of the structure.

The cover for protection against corrosion may not be sufficient for fire protection, so this should be considered at the onset of the design, and also the dimensions of the members.

The Code gives maximum water/cement ratios, minimum cement contents and minimum characteristic strengths for concretes suitable for use in various environments with specified covers and using 20 mm nominal maximum size aggregate. The minimum grades will generally ensure that the limits on free water/cement ratio and cement content will be met without further checking.

## Application

Durability and fire resistance requirements are considered at the onset of the design process because this determines the grade of concrete, the cover, and the size of the members. Usually, for most structures, Part 1 of the Code will be used in which it is assumed that the ultimate limit state will be the most critical limit state. Design will therefore be carried out at this limit state, followed by checks to ensure that the serviceability limit states of deflection and cracking are not reached. In special circumstances, other limit states, such as vibration or the effects of fatigue, may require consideration. Should it be necessary to calculate deflections and crack widths, methods are given in Part 2 of the Code. The serviceability limit state of deflection may be the limiting requirement for floor slabs with large span/effective-depth ratios. This can be checked before the reinforcement is determined, although some engineers may prefer to follow the procedure where the check is made after the reinforcement has been found.

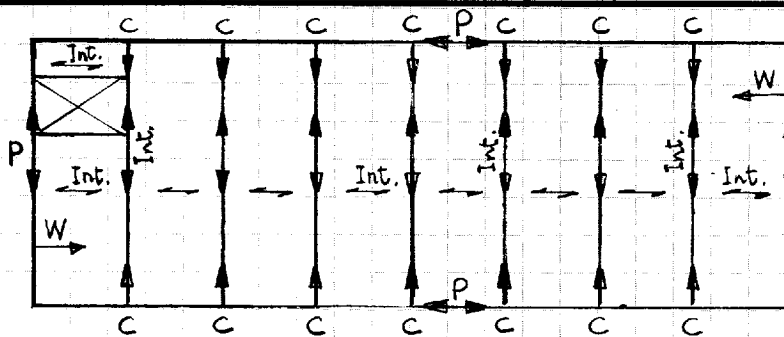
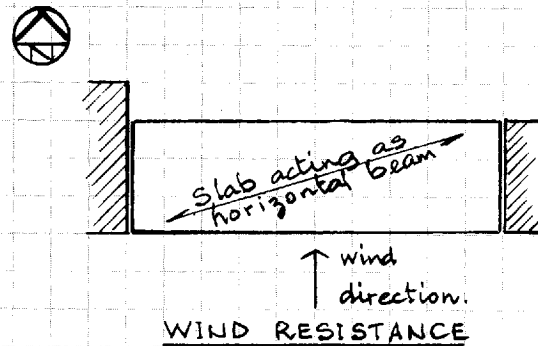
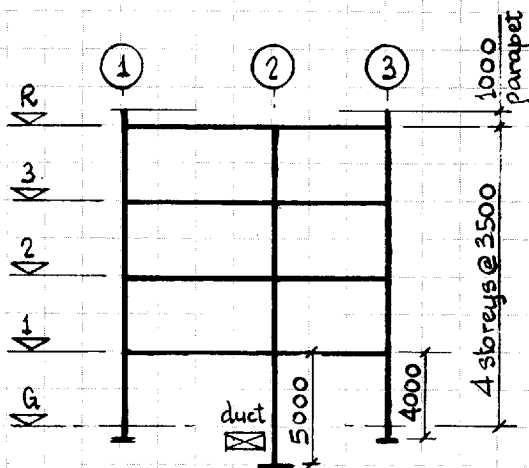
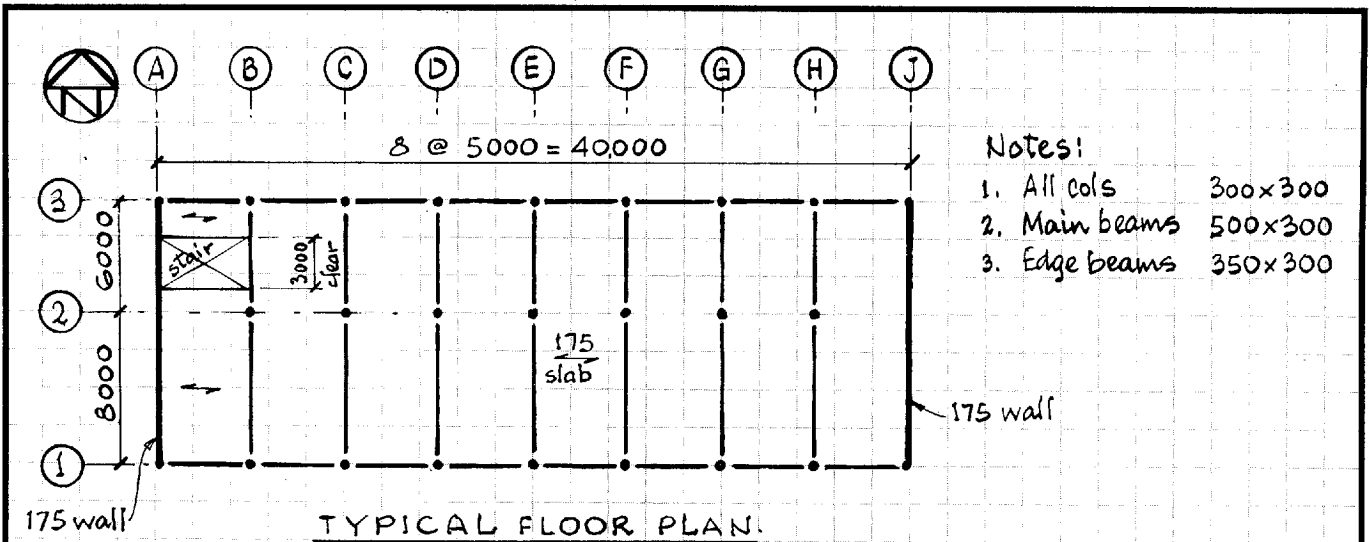
Simplified detailing requirements for the curtailment of the reinforcement may be used for beams and slabs which fulfil certain design conditions. However, for other situations, the curtailments should be taken from a bending moment envelope and be in accordance with the general recommendations of the Code.

# Design information

Client Architect	National Company plc	Engineer responsible BR Rogers
London	9/11/98	Building Regulation authority or other and Date of submission
1. The Building Regulations 1991 2. BS8110 The Structural Use of Concrete, Part 1 1997 3. BS6399 Part 1 1996, Part 2 1995, and Part 3 1988		Relevant Building Regulations and Design Codes
Laboratory and Office Block		Intended use of structure
1 hour for all elements		Fire resistance requirements
Roof - imposed = 1.5 kN/m <sup>2</sup> " - finishes = 1.5 kN/m <sup>2</sup> Floors - imposed (3.0) and partitions (1.0) = 4.0 kN/m <sup>2</sup> Stairs - imposed = 4.0 kN/m <sup>2</sup> " - finishes to Floors and Stairs = 0.5 kN/m <sup>2</sup> External Cladding = 5.0 kN/m <sup>2</sup>		General loading conditions
Speed Factors	21 m/sec. (basic) $S_a = 1.05$ $S_b = 1.71$ , $S_d = 1.0$ , $S_s = 1.0$ , $S_p = 1.0$ $C_a = 0.84$ , $C_{pe} = +0.8$ (front), $C_{pe} = -0.3$ (rear), $C_r = 0.025$	Wind loading conditions
	Severe (external) and Mild (internal) (BS8110 Table 3.2)	Exposure conditions
	Stiff clay - no sulphates Allowable bearing pressure = 200 kN/m <sup>2</sup>	Subsoil conditions
	RC footings to columns and walls	Foundation type
	Grade 40 with 20mm. max. aggregate (BS8110 Table 3.3)	Material data
	Characteristic strength - main bars, $f_y = 460 \text{ N/mm}^2$ " " - links = 250 N/mm <sup>2</sup>	
	Self weight of concrete = 24.0 kN/m <sup>3</sup> All dimensions shown on drawings are in millimetres (mm)	Other relevant information



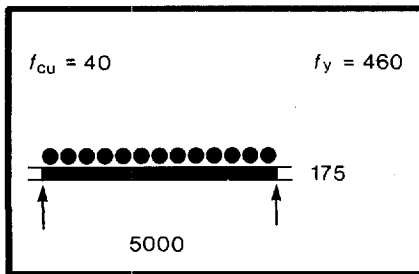
# Structural summary sheet



3.12.3

3.12.3.4.2

$$F_t = (20 + 4n_o) = 20 + 16 = 36 < 60 \text{ kN.}$$

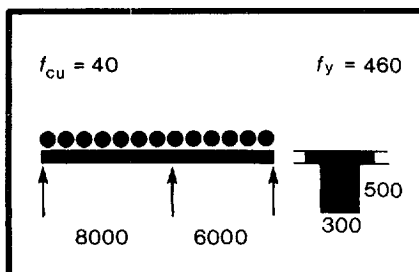


# Floor slab

interior-span solid slab

BS 8110 ref.	CALCULATIONS	OUTPUT
3.3 Table 3.3 " 3.4	<u>DURABILITY and FIRE RESISTANCE</u> Nominal cover for mild conditions of exposure = 20 mm. Max. fire resistance of 175 slab with 20 cover = 1 1/2 hr > 1 hr.	cover = 20 mm. ∴ 1 hr. fire resistance ok
3.5.2.4 Table 3.12	<u>LOADING</u> Self-weight $0.175 \times 24 = 4.2$ Finishes (page 6) = 0.5 Characteristic dead load = 4.7 kN/m <sup>2</sup> " imposed " (page 6) = 4.0 kN/m <sup>2</sup> Design load = $(1.4 \times 4.7 + 1.6 \times 4) \times 5.0 = 64.9$ kN/width	$g_k = 4.7$ kN/m <sup>2</sup> $q_k = 4.0$ kN/m <sup>2</sup> $F = 64.9$ kN/width
3.5.2.4 Table 3.12	<u>ULTIMATE B.M.'S</u> Interior mid-span and supports $0.063 Fl = 0.063 \times 64.9 \times 5.0 = 20.4$ kNm/m	
3.4.4.4 Table 3.8	<u>REINFORCEMENT</u> Interior mid-span and supports $K = \frac{M}{f_{cu} b d^2} = \frac{20.4 \times 10^6}{40 \times 10^3 \times 149^2} = 0.023$ $z = 149 (0.5 + \sqrt{0.25 - \frac{0.023}{0.9}}) = 145$ (but $> 0.95 \times 149 = 141.5$ ) $A_s = \frac{M}{0.95 f_y z} = \frac{20.4 \times 10^6}{0.95 \times 460 \times 141.5} = 330$ mm <sup>2</sup> /m Check for Shear: $v = \frac{0.5 \times 64.9 \times 10^3}{10^3 \times 149} = 0.22$ N/mm <sup>2</sup> < $v_c$	$d = 149$ mm.  Top & Bottom T12 @ 300 (377 mm <sup>2</sup> /m)  ∴ ok.
3.4.6 Table 3.9 Table 3.10	<u>DEFLECTION</u> basic span/eff. depth ratio = 26 max. $\frac{M}{b d^2} = \frac{20.4 \times 10^6}{10^3 \times 149^2} = 0.92$ . $f_s = \frac{2 \times 460 \times 330}{3 \times 377} = 268.4$ N/mm <sup>2</sup> Modification factor for tension reinf. = 1.5 ∴ Allowable span/eff. depth ratio = $26 \times 1.5 = 39$ ∴ Actual " " " = $\frac{5000}{149} = 33.56$	∴ $l/d$ ratio ok.
3.12.11.2.7	<u>CRACKING</u> $3d = 3 \times 149 = 447$ mm Spacing between bars = $300 - 12 = 288$ mm < $3d$ $h = 175 < 200$ mm ∴ no further checks required	∴ spacing ok. ok.
3.12.3.4 Table 3.27	<u>TIE PROVISION</u> East-West INTERNAL TIE $F_t = 36$ kN/m width Tie force = $F_t \left( \frac{g_k + q_k}{7.5} \right) \frac{l_r}{5} = 36 \left( \frac{4.7 + 4}{7.5} \right) \frac{5}{5} = 41.8$ kN/m.w. > $F_t$ ∴ Min. Area = $\frac{41.8 \times 10^3}{460} = 91$ mm <sup>2</sup> /m. LAP = $\frac{91 \times 35}{377} \phi = 8.5 \phi < 300$ mm.	Lap 300 mm @ supports  Bottom T12 @ 300 (377 mm <sup>2</sup> /m)





# First-floor main beam

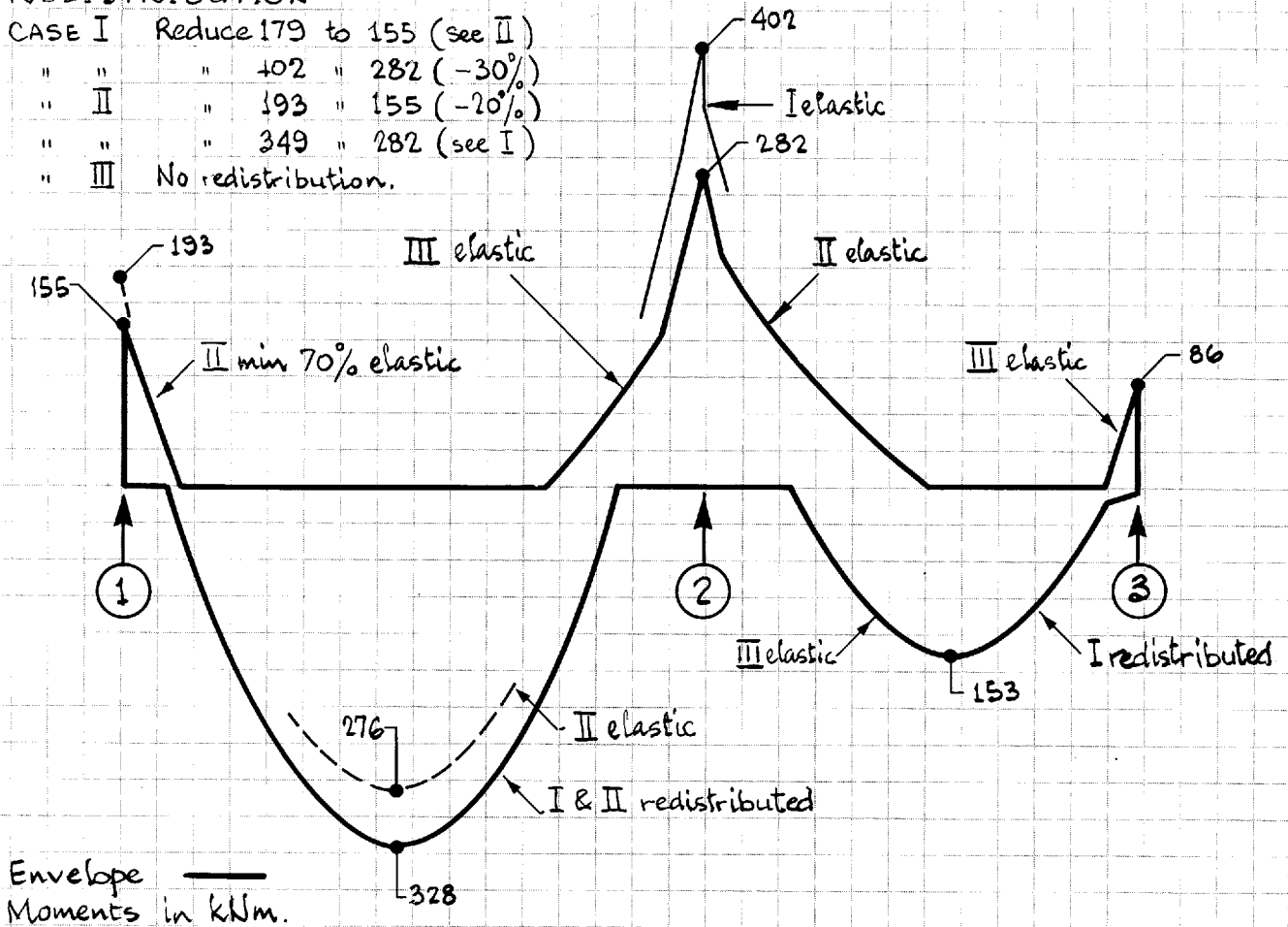
two-span flanged beam

BS 8110 ref.	CALCULATIONS	OUTPUT																																																																																																
3.2.1.2.1	<p><b>SUBFRAME ANALYSIS</b></p> <p>A linear elastic analysis either produced by hand or by computer program is used to obtain the moments and forces. For the first floor beams the columns above are assumed to be fixed and those below pinned at foundation level. The foundation will not provide rotational restraint. Lateral wind loads are taken by the end shear walls.</p>																																																																																																	
3.3 Tables 3.3, 3.4	<p><b>DURABILITY and FIRE RESISTANCE</b></p> <p>Nominal cover for mild conditions of exposure = 20mm</p> <p>Nominal cover for 300mm wide beam for 1hr. period = 20mm.</p>	Minimum cover to LINKS 20mm.																																																																																																
3.2.1.2.2	<p><b>LOADING</b></p> <p>Dead load from 175 slab (page 8) = <math>5 \times 4.7 = 23.5</math></p> <p>Self-weight <math>(0.5 - 0.175) 0.3 \times 24 = 2.3</math></p> <p><math>\therefore</math> Characteristic dead load on beam = <math>25.8 \text{ kN/m}</math>.</p> <p>" imposed " " " (p.8) = <math>5 \times 4 = 20.0 \text{ kN/m}</math>.</p> <p>Maximum design load = <math>1.4g_k + 1.6q_k = 36.12 + 32 = 68.12 \text{ kN/m}</math>.</p> <p>Minimum " " = <math>1.0g_k = 25.8 \text{ kN/m}</math>.</p>	$1.0g_k = 25.8 \text{ kN/m}$ $1.0q_k = 20.0 \text{ kN/m}$ $1.4g_k + 1.6q_k = 68.12 \text{ kN/m}$																																																																																																
	<p><b>BENDING MOMENTS (kNm)</b></p> <p>Sign convention used:</p> <p>Results from Analysis</p> <table><tr><th>I</th><th>8m.</th><th></th><th>②</th><th>6m.</th><th></th></tr><tr><td></td><td>68.12</td><td></td><td></td><td>68.12</td><td></td></tr><tr><td>- 179</td><td>260</td><td>+ 402</td><td>- 348</td><td>120</td><td>+ 60</td></tr><tr><td>+ 108</td><td></td><td></td><td>- 35</td><td></td><td>- 36</td></tr><tr><td>+ 71</td><td></td><td></td><td>- 19</td><td></td><td>- 24</td></tr><tr><td>245</td><td></td><td>300</td><td>252</td><td></td><td>156</td></tr></table> <p>← Spans</p> <p>CASE I loading Beam moments Upper Column mts. Lower " "</p> <table><tr><th>II</th><th>68.12</th><th></th><th>25.8</th><th></th><th></th></tr><tr><td>- 193</td><td>276</td><td>+ 349</td><td>- 250</td><td>28</td><td>- 4</td></tr><tr><td>+ 117</td><td></td><td></td><td>- 65</td><td></td><td>+ 2.5</td></tr><tr><td>+ 76</td><td></td><td></td><td>- 34</td><td></td><td>+ 1.5</td></tr><tr><td>253</td><td></td><td>292</td><td>120</td><td></td><td>35</td></tr></table> <p>CASE II loading Beam moments Upper Column mts. Lower " "</p> <table><tr><th>III</th><th>25.8</th><th></th><th>68.12</th><th></th><th></th></tr><tr><td>- 54</td><td>84</td><td>+ 204</td><td>- 230</td><td>153</td><td>+ 86</td></tr><tr><td>+ 33</td><td></td><td></td><td>+ 17</td><td></td><td>- 52</td></tr><tr><td>+ 21</td><td></td><td></td><td>+ 9</td><td></td><td>- 34</td></tr><tr><td>84</td><td></td><td>122</td><td>228</td><td></td><td>180</td></tr></table> <p>CASE III loading Beam moments Upper Column mts. Lower " "</p>	I	8m.		②	6m.			68.12			68.12		- 179	260	+ 402	- 348	120	+ 60	+ 108			- 35		- 36	+ 71			- 19		- 24	245		300	252		156	II	68.12		25.8			- 193	276	+ 349	- 250	28	- 4	+ 117			- 65		+ 2.5	+ 76			- 34		+ 1.5	253		292	120		35	III	25.8		68.12			- 54	84	+ 204	- 230	153	+ 86	+ 33			+ 17		- 52	+ 21			+ 9		- 34	84		122	228		180	
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# CALCULATIONS

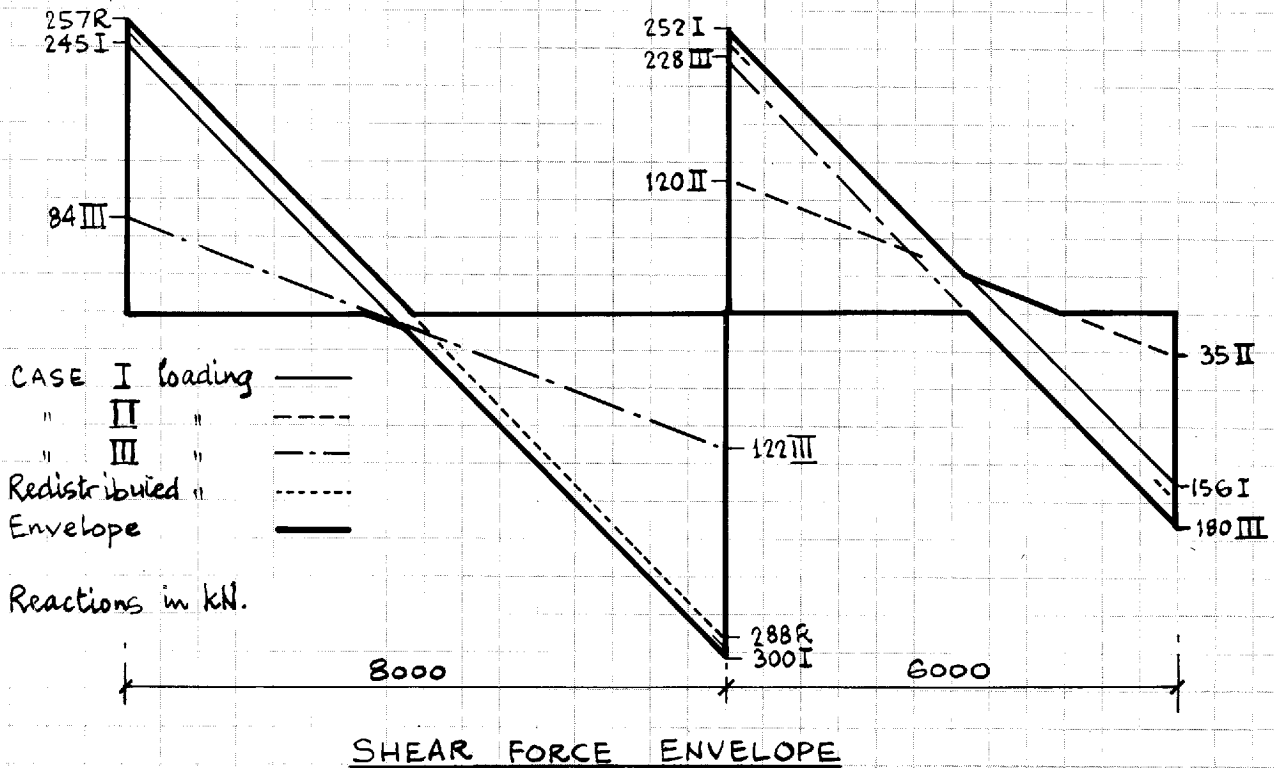
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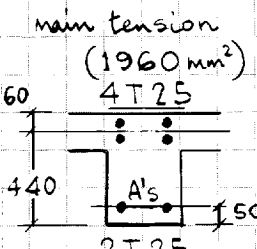
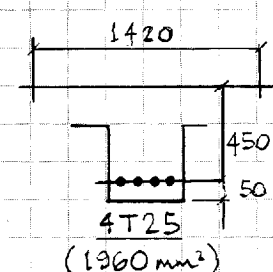
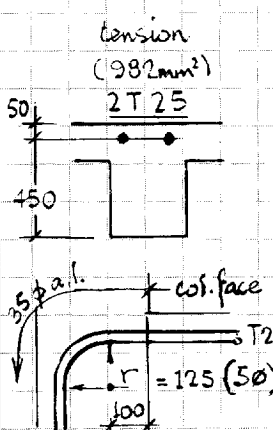
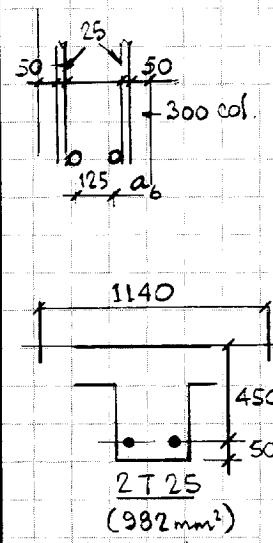
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" II	" 193 " 155 (-20%)
" "	" 349 " 282 (see I)
" III	No redistribution.

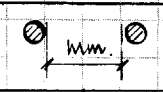



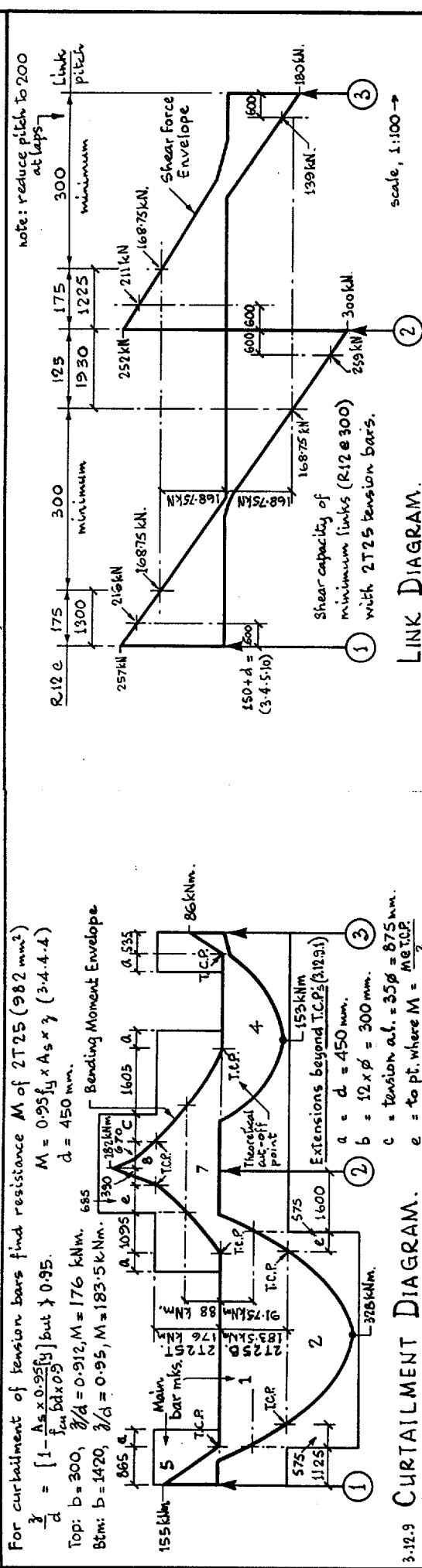
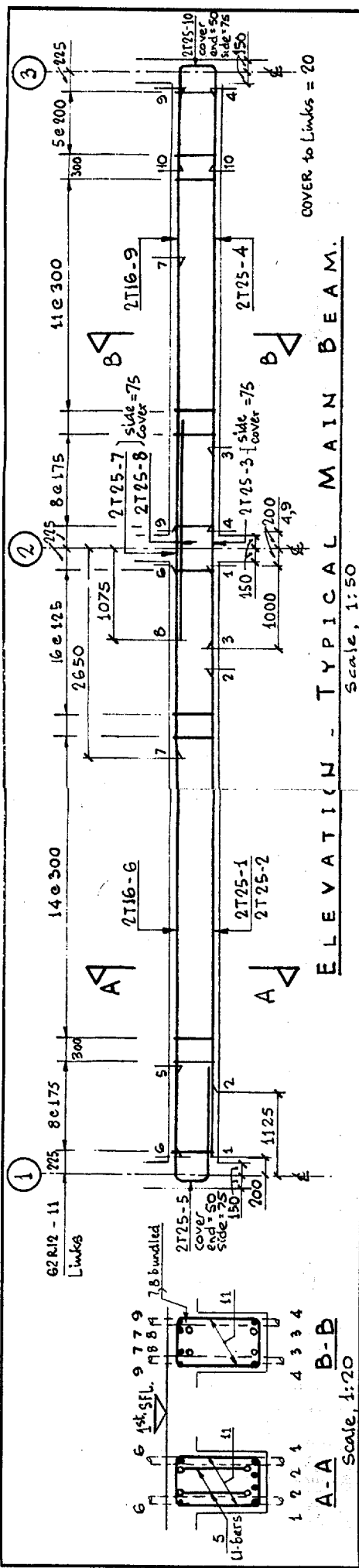
3.2.2

## BENDING MOMENT ENVELOPE



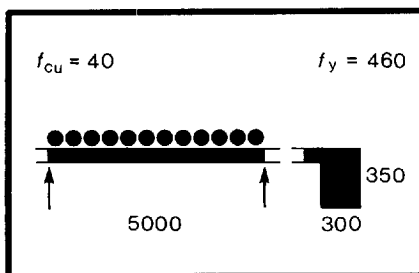
BS 8110 ref.	CALCULATIONS	OUTPUT
3.2.2.1 3.4.4.4	<p><u>MAIN BEAM, 1<sup>ST</sup>. FLOOR (continued)</u></p> <p><u>Internal Support:</u></p> <p>From B.M. envelope, page 11, <math>M = 282 \text{ kNm}</math></p> $\beta_b = \frac{282}{402} = 0.7, \quad \frac{x}{d} \leq 0.3$ $K' = 0.402(0.7 - 0.4) - 0.18(0.7 - 0.4)^2 = 0.104$ $K = \frac{M}{f_{cu} b d^2} = \frac{282 \times 10^6}{40 \times 300 \times 440^2} = 0.121 > 0.104$ <p>Comp. reinforcement reqd, <math>d' = 50</math>, <math>\frac{d'}{x} = 0.38 &gt; 0.37</math></p> $x = \frac{(d - z')}{0.45}, \therefore z' = 1 - 0.45 \times 0.3 = 0.865$ $A'_s = \frac{(0.121 - 0.104) \times 40 \times 300 \times 440^2}{0.95 \times 460 \times (440 - 50)} = 232 \text{ mm}^2$	<p>main tension (1960 mm<sup>2</sup>) 4T25</p>  <p>2T25 (982 mm<sup>2</sup>) compression</p>
Table 3.25	<p>Check minimum <math>A'_s = 0.2(300 \times 500)/100 = 300 \text{ mm}^2</math></p> $A_s = \frac{0.104 \times 40 \times 300 \times 440^2}{0.95 \times 460 \times 0.865 \times 440} + 232 = 1685 \text{ mm}^2$	
3.4.1.5 3.4.4.4	<p><u>8m mid-span</u></p> <p>From B.M. envelope, page 11, <math>M = 328 \text{ kNm}</math></p> <p>Eff. flange width = <math>b + (0.2 \times 0.7l) = 300 + 0.14 \times 8000 = 1420 \text{ mm}</math></p> $\frac{M}{f_{cu} b d^2} = \frac{328 \times 10^6}{40 \times 1420 \times 450^2} = 0.029$ $\frac{x}{d} = (0.5 + \sqrt{0.25 - \frac{0.029}{0.9}}) = 0.97 > 0.95 \therefore x = \frac{450(1 - 0.95)}{0.45} = 50 \text{ mm (N.A. in flange)}$ $A_s = \frac{328 \times 10^6}{0.95 \times 460 \times 0.95 \times 450} = 1756 \text{ mm}^2$	 <p>main tension (1960 mm<sup>2</sup>) 4T25</p>
3.2.2.1 3.4.4.4	<p><u>8m end-support</u></p> <p>From B.M. envelope, page 11, <math>M = 155 \text{ kNm}</math></p> $\beta_b = \frac{155}{193} = 0.8, \quad \frac{x}{d} \leq 0.4$ $\frac{M}{f_{cu} b d^2} = \frac{155 \times 10^6}{40 \times 300 \times 450^2} = 0.064$ $\frac{x}{d} = (0.5 + \sqrt{0.25 - \frac{0.064}{0.9}}) = 0.92 \therefore \frac{x}{d} = \left(\frac{1 - 0.92}{0.45}\right) = 0.18 \text{ (ok)}$ $A_s = \frac{155 \times 10^6}{0.95 \times 460 \times 0.92 \times 450} = 857 \text{ mm}^2$ <p>Check force in bar = <math>\frac{775 \times 0.95 \times 460 \times 491}{875 \times 10^3} = 190 \text{ kN}</math></p> <p><math>\therefore</math> Int. bend radius <math>r = \frac{190 \times 10^3 (1 + \frac{50}{125})}{2 \times 40 \times 25} = 133</math>, say 125 mm.</p>	<p>tension (982 mm<sup>2</sup>) 2T25</p> 
3.4.1.5 Table 3.25	<p><u>6m end-support</u></p> <p>From B.M. envelope, page 11, <math>M = 86 \text{ kNm}</math></p> $\frac{M}{f_{cu} b d^2} = 0.35, \quad \frac{x}{d} = 0.95$ $A_s = \frac{86 \times 10^6}{0.95 \times 460 \times 0.95 \times 450} = 460 \text{ mm}^2$ <p><u>6m mid-span</u></p> <p>From B.M. envelope, page 11, <math>M = 153 \text{ kNm}</math></p> <p>Eff. fig. width = <math>300 + 0.14 \times 6000 = 1140 \text{ mm}</math></p> $\frac{M}{f_{cu} b d^2} = 0.017, \quad \frac{x}{d} = 0.95$ $A_s = \frac{153 \times 10^6}{0.95 \times 460 \times 0.95 \times 450} = 819 \text{ mm}^2$ <p>Check for minimum reinforcement in flanged beam</p> <p>web in tension <math>0.0018 \times 300 \times 500 = 270 \text{ mm}^2 \text{ ok}</math></p> <p>flange in tension over supports, <math>0.0026 \times 300 \times 500 = 390 \text{ mm}^2 \text{ ok}</math></p>	 <p>main tension (982 mm<sup>2</sup>) 2T25</p>

BS 8110 ref.	CALCULATIONS							OUTPUT
3.4.5	<b>SHEAR REINFORCEMENT</b>							$d = 450 \text{ mm.}$
	Min. effective tension reinf. = 2T25 (982 mm <sup>2</sup> )							
3.4.5.4 Table 3-8	$\frac{100A_s}{bd} = \frac{100 \times 982}{300 \times 450} = 0.73, v_c = 0.57 \left( \frac{40}{25} \right)^{\frac{1}{3}} = 0.66 \text{ N/mm}^2$							
Table 3-7	Min. links = $\frac{A_{sv}}{S_v} = \frac{b \times 0.4}{0.95 f_{yv}} = \frac{300 \times 0.4}{0.95 \times 250} = 0.51$							
3.4.5.5	Max. $S_v = 0.75 \times 450 = 338$ , say 300 mm. Try R12 @ 300 as min.							
Table 3-7	$\frac{A_{sv}}{S_v} = \frac{226}{300} = 0.75 \therefore (v - v_c) = \frac{0.75 \times 0.4}{0.51} = 0.59 \text{ N/mm}^2$							
Table 3-7	Select R12 Links	$A_{sv}$ 226 mm <sup>2</sup>	$S_v$ $A_{sv}/S_v$	300 0.75	175 1.29	150 1.50	125 1.81	
Table 3-7 3.4.5.10	Location	V kN @ eff. d from support face.		$v = V/bd \text{ N/mm}^2$	$v - v_c$	$A_{sv}/S_v$	Links	
	8m. L.H. end	216		1.60	0.94	1.19	R12 @ 175	
	" R.H. "	259		1.92	1.26	1.59	R12 @ 125	
	6m. L.H. "	211		1.56	0.90	1.14	R12 @ 175	
	" R.H. "	139		1.03	0.37	0.51	R12 @ 300	
	minimum	$V = 168.75 \text{ kN}$		1.25	0.59	0.75	R12 @ 300	
3.4.6.1	<b>DEFLECTION</b> basic $l/d$ ratio = 20.8 max. ( $\frac{b_w}{b} < 0.3$ )							
Table 3-9	$\frac{M}{bd^2} = \frac{328 \times 10^6}{1420 \times 450^2} = 1.14, f_s = \frac{2 \times 460 \times 1756 \times 276}{3 \times 1960 \times 328} = 231.2 \text{ N/mm}^2$							
3.4.6.5	Modification factor = 1.55, $\therefore$ Allowable $l/d = 20.8 \times 1.55 = 32.24$							
Table 3.10	Actual $l/d = \frac{8000}{450} = 17.8$							$\therefore l/d$ ratio ok.
3.12.11.2	<b>CRACKING</b> limit crack widths by limiting bar spacing							
	Tension reinf.	M redistr. %						
			actual	allowed	actual	allowed		
Table 3-28	8m. external support T	-20	100 < 125		-	-	} $\therefore$ crack widths ok.	
3.12.11.2.3	" mid-span ( $f_s = 231.2$ ) B	+18	40 < 203		35	< 101.5		
3.12.11.2.4	Internal support T	-30	100 < 110		-	-		
3.12.11.2.5	6m. mid-span ( $f_s = 256$ ) B	0	180 < 184		35	< 92		
	" external support T	0	100 < 155		-	-		
3.12.3	<b>TIE PROVISION</b> $F_t = 36 \text{ kN/m width}$ (see page 7)							
	a. Internal (N $\rightarrow$ S) tie force for 8m. span							
3.12.3.4.2	$= F_t \left( \frac{g_k + q_k}{7.5} \right) \frac{l}{5} = 36 \left( \frac{4.7 + 4.0}{7.5} \right) \frac{8}{5} = 66.8 \text{ kN/m width}$							2T25 (982 mm <sup>2</sup> )
3.12.8.13b	$\therefore$ Min. area continuity reinf. = $66.8 \times 5 \times \frac{10^3}{460} = 726 \text{ mm}^2$							Use bottom bars already provided but made continuous using 1000 laps
	Cover to lapped bars = $35 < (2 \times 25) \text{ mm.}$							
Table 3-27	Spacing between adjacent laps = $100 < (6 \times 25) \text{ mm.}$							
	$\therefore$ Lap = $49 \times 25 \times \frac{726}{982} = 906 \text{ mm.}$							
3.12.3.6.1	b. External Column (N $\rightarrow$ S) tie force.							
	(i) $\frac{l}{2.5} F_t = \frac{3.325}{2.5} \times 36 = 47.9 < 2F_t = 72 \text{ kN}$							Use U-bar anchorage at external columns.
	(ii) 3% of total design ultimate load carried by column							
	$= 0.03 \times 1340 = 40.2 < 47.9 \text{ kN}$							
	$\therefore$ Min. area of continuity reinf. = $47.9 \times \frac{10^3}{460} = 104 \text{ mm}^2$							



3.12.9 CURTAILMENT DIAGRAM.	
BS 8110 ref	Bar marks
3.12.1	1
3.12.2	2
3.12.3	3
3.12.4	4
3.12.5	5
3.12.6	6
3.12.7	7
3.12.8	8
3.12.9	9
3.12.10	10
3.12.11	11
3.12.12	12
3.12.13	13
3.12.14	14
3.12.15	15
3.12.16	16
3.12.17	17
3.12.18	18
3.12.19	19
3.12.20	20
3.12.21	21
3.12.22	22
3.12.23	23
3.12.24	24
3.12.25	25
3.12.26	26
3.12.27	27
3.12.28	28
3.12.29	29
3.12.30	30
3.12.31	31
3.12.32	32
3.12.33	33
3.12.34	34
3.12.35	35
3.12.36	36
3.12.37	37
3.12.38	38
3.12.39	39
3.12.40	40
3.12.41	41
3.12.42	42
3.12.43	43
3.12.44	44
3.12.45	45
3.12.46	46
3.12.47	47
3.12.48	48
3.12.49	49
3.12.50	50
3.12.51	51
3.12.52	52
3.12.53	53
3.12.54	54
3.12.55	55
3.12.56	56
3.12.57	57
3.12.58	58
3.12.59	59
3.12.60	60
3.12.61	61
3.12.62	62
3.12.63	63
3.12.64	64
3.12.65	65
3.12.66	66
3.12.67	67
3.12.68	68
3.12.69	69
3.12.70	70
3.12.71	71
3.12.72	72
3.12.73	73
3.12.74	74
3.12.75	75
3.12.76	76
3.12.77	77
3.12.78	78
3.12.79	79
3.12.80	80
3.12.81	81
3.12.82	82
3.12.83	83
3.12.84	84
3.12.85	85
3.12.86	86
3.12.87	87
3.12.88	88
3.12.89	89
3.12.90	90
3.12.91	91
3.12.92	92
3.12.93	93
3.12.94	94
3.12.95	95
3.12.96	96
3.12.97	97
3.12.98	98
3.12.99	99
3.12.100	100

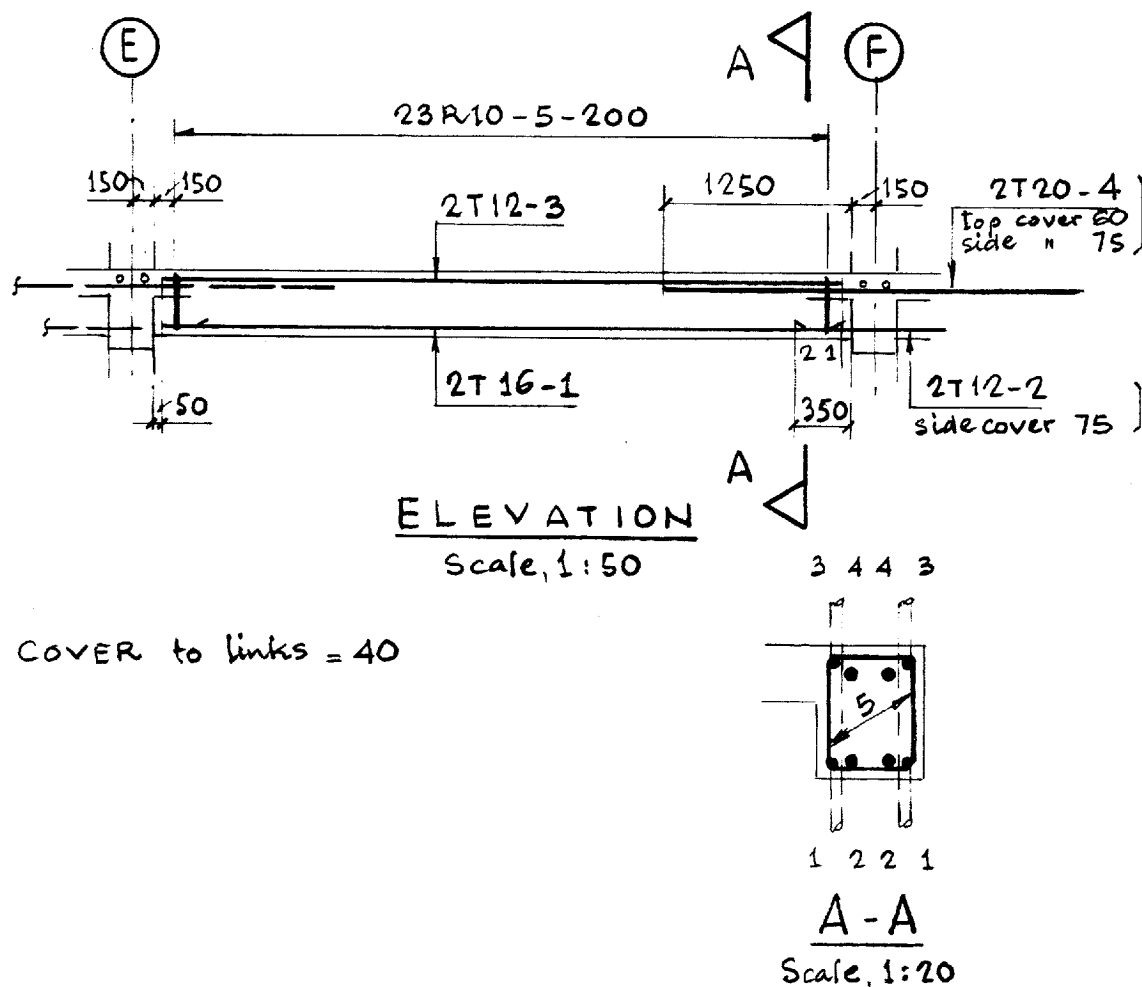




# Edge beam

interior-span flanged beam

BS 8110 ref.	CALCULATIONS	OUTPUT
3.3 Tables 3.3, 3.4	<b>DURABILITY and FIRE RESISTANCE</b> Nominal cover for severe conditions of exposure = 40mm. " " " 300 wide beam for 1hr period = 20mm.	Minimum cover to links = 40mm
3.4.3	<b>LOADING</b> Dead load from slab (1250 strip, p.10) = $23.5 \times 1.25 = 29.4$ Self weight $(0.35 - 0.175) \times 0.3 \times 24 \times 5 = 6.3$ Cladding @ 5kN/m (page 6) = $5 \times 5 = 25.0$ $\therefore$ Characteristic dead load = 60.7kN. " imposed " from slab (p.10) = $20 \times 1.25 = 25.0$ kN. Design load = $1.4 G_k + 1.6 Q_k = 85 + 40 = 125.0$ kN.	$G_k = 60.7$ kN $Q_k = 25.0$ kN $F = 125.0$ kN.
Table 3.5	<b>ULTIMATE B.M.'s</b> Interior supports: $M = 0.08 FL = 0.08 \times 125 \times 5 = 50.0$ kNm Mid-int. span: $M = 0.07 FL = 0.07 \times 125 \times 5 = 43.8$ kNm	
3.4.4.4 3.4.1.5 Table 3.5 3.4.5.10 Table 3.8 Table 3.7	<b>REINFORCEMENT</b> Interior supports: $\frac{M}{f_{cu} b d^2} = \frac{50 \times 10^6}{40 \times 300 \times 280^2} = 0.053$ $\frac{x}{d} = (0.5 + \sqrt{(0.25 - \frac{0.053}{0.9})}) = 0.937$ , $A_s = \frac{50 \times 10^6}{0.95 \times 460 \times 0.937 \times 280} = 436 \text{ mm}^2$ Mid-int. span: eff. flange width = $300 + \frac{2.7}{10} \times 5000 = 650 \text{ mm}$ . $\frac{M}{f_{cu} b d^2} = \frac{43.8 \times 10^6}{40 \times 650 \times 290^2} = 0.02$ , $\frac{x}{d} = 0.95$ , $A_s = \frac{43.8 \times 10^6}{0.95 \times 460 \times 0.95 \times 290} = 364 \text{ mm}^2$ Shear force = $0.55 F \times 125 = 68.75$ kN. Eff. tension reinf for shear is 2T20. Design shear force = $68.75 - (0.15 + 0.28) 25 = 58$ kN. $\frac{100 A_s}{b d} = \frac{100 \times 628}{300 \times 280} = 0.75$ , $v_c = 0.73 \text{ N/mm}^2$ , $v = \frac{58 \times 10^3}{300 \times 280} = 0.69 \text{ N/mm}^2$ $v < (v_c + 0.4) \therefore$ use min links, $\frac{A_{sv}}{s_v} = \frac{0.4 \times 300}{0.95 \times 250} = 0.51$ Max. $s_v = 0.75 \times 280 = 210 \text{ mm}$ (use 200mm) $\frac{A_{sv}}{s_v} = \frac{157}{200} = 0.79 > 0.51$	 R10 @ 200 throughout span.
3.4.6.1 Table 3.9 3.4.6.5 Table 3.10	<b>DEFLECTION</b> basic $l/d = 22.0$ max, ( $b_w/b = 0.46 > 0.3$ ) $\frac{M}{b d^2} = \frac{43.8 \times 10^6}{650 \times 290^2} = 0.8$ , $f_s = \frac{2 \times 460 \times 364}{3 \times 402} = 278 \text{ N/mm}^2$ $\therefore$ Modification factor = 1.53 $\therefore$ Allowable span/eff. depth ratio = $22 \times 1.53 = 33.7$ $\therefore$ Actual " " " " = $\frac{5000}{290} = 17.2$	$\therefore l/d$ ratio ok.
3.12.11.24 Table 3.10 3.12.11.24	<b>CRACKING</b> Bottom bars $f_s = 278 \text{ N/mm}^2$ (see deflection) Allowable clear spacing = $\frac{47000}{278} = 169 > 168$ , side cover $50 < \frac{169}{2}$ Top bars $f_s = \frac{2 \times 460 \times 436}{3 \times 628} = 213 \text{ N/mm}^2$ , cover $> 50$ , check corner Allowable clear spacing = $\frac{47000}{213} = 220$ , corner distance $100 < \frac{220}{2}$	 ok ok
Table 3.27 3.12.3.5 3.12.8.11	<b>TIE PROVISION</b> Peripheral tie, $F_t = 36$ kN. $A_s = \frac{36 \times 10^3}{460} = 78 \text{ mm}^2$ Use 2T12. Min lap = $35 \times 12 \times \frac{78}{226} = 145 < 300 \text{ mm}$	TOP or BOTTOM 2T12 min. (226 mm²)

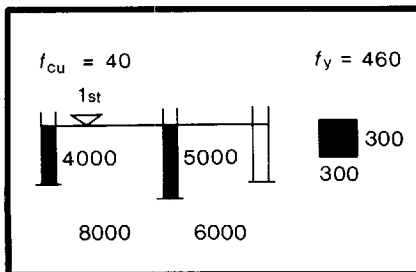


#### Commentary on bar arrangement

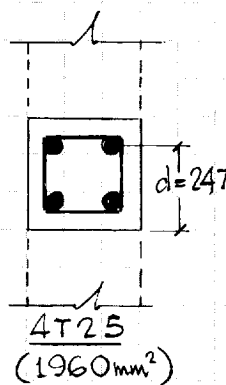
BS 8110 ref	Bar marks	Notes
3.12.8.11	1	Horizontal bars in this member provide the peripheral tie. Minimum lap = 300 mm. The two tension bars are stopped 50 mm from the column face to avoid clashing with the column bars shown in section A-A.
3.12.10.2 Figure 3.24 Table 3.27	2	Separate splice bars are fixed inside vertical column bars. Minimum area = $30\% A_s \text{ mid-span} = 0.3 \times 364 = 109 \text{ mm}^2$ . Use 2T12 = 226 mm <sup>2</sup> . Lap = $35 \times 12 \times 109 / 226 = 203 \text{ mm} > 15 \times 12 = 180 \text{ mm} < 300 \text{ mm}$ . Use 300 mm lap.
3.12.10.2 Figure 3.24	3	Link hanger bars also provide support for slab top reinforcement. Minimum area = $20\% A_s \text{ support} = 0.2 \times 436 = 87 \text{ mm}^2$ . Use 2T12 = 226 mm <sup>2</sup> .
3.12.10.2 Figure 3.24	4	Tension reinforcement over support is fixed inside vertical column bars. Bars are curtailed at $0.25 \text{ span from face of support} = 0.25 \times 5000 = 1250 \text{ mm} > 45 \times 20 = 900 \text{ mm}$
-	5	Closed links are shape code 61.

# Columns

slender and short columns



BS 8110 ref.	CALCULATIONS	OUTPUT																																																																																																																																																																																																																																											
3.2.1.2.1	SUB-FRAME ANALYSIS - refer to main beam, page 10.																																																																																																																																																																																																																																												
3.3 Tables	DURABILITY and FIRE RESISTANCE	Minimum cover to links																																																																																																																																																																																																																																											
3.3	Nominal cover for mild conditions of exposure = 20 mm.	Internal 20 (use 30 mm)																																																																																																																																																																																																																																											
	" " " Severe " " " = 40 mm.	External 40 mm.																																																																																																																																																																																																																																											
3.4	" " " 300x300 column for 1 hr. period = 20 mm.																																																																																																																																																																																																																																												
	INTERNAL COLUMN (Foundation → Roof) AXIAL LOADING and MOMENTS from ANALYSIS	F2 (typical)																																																																																																																																																																																																																																											
	<table><tr><th rowspan="3">LOAD CASE</th><th colspan="2">BEAM LOADS kN</th><th colspan="4">COLUMN DESIGN LOADS kN</th><th colspan="4">COL. MOMENTS kNm</th></tr><tr><th colspan="2">TOTAL</th><th colspan="2">IMPOSED</th><th colspan="2">DEAD</th><th colspan="2">TOP</th><th colspan="2">BOTTOM</th></tr><tr><th>1</th><th>2</th><th>1</th><th>2</th><th>1</th><th>2</th><th>1</th><th>2</th><th>1</th><th>2</th></tr><tr><td>LEVEL</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>Roof</td><td>249</td><td>244</td><td>54</td><td>53</td><td>195</td><td>191</td><td>34</td><td>54</td><td></td><td></td><td></td></tr><tr><td></td><td>210</td><td>133</td><td>46</td><td></td><td>164</td><td>133</td><td></td><td></td><td>32</td><td>58</td><td></td></tr><tr><td>SW.</td><td></td><td></td><td></td><td></td><td>9</td><td>9</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td>100</td><td>53</td><td>368</td><td>333</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>3rd Fl.</td><td>298</td><td>290</td><td>140</td><td>136</td><td>158</td><td>154</td><td>32</td><td>58</td><td></td><td></td><td></td></tr><tr><td></td><td>249</td><td>117</td><td>117</td><td></td><td>132</td><td>117</td><td></td><td></td><td>32</td><td>58</td><td></td></tr><tr><td>SW.</td><td></td><td></td><td></td><td></td><td>9</td><td>9</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td>357</td><td>189</td><td>667</td><td>613</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>2nd Fl.</td><td>298</td><td>290</td><td>140</td><td>136</td><td>158</td><td>154</td><td>32</td><td>58</td><td></td><td></td><td></td></tr><tr><td></td><td>249</td><td>117</td><td>117</td><td></td><td>132</td><td>117</td><td></td><td></td><td>35</td><td>65</td><td></td></tr><tr><td>SW.</td><td></td><td></td><td></td><td></td><td>9</td><td>9</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td>614</td><td>325</td><td>966</td><td>893</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>1st Fl.</td><td>300</td><td>292</td><td>141</td><td>137</td><td>159</td><td>155</td><td>19</td><td>34</td><td></td><td></td><td></td></tr><tr><td></td><td>252</td><td>120</td><td>118</td><td></td><td>134</td><td>120</td><td></td><td></td><td>-</td><td>-</td><td></td></tr><tr><td>SW.</td><td></td><td></td><td></td><td></td><td>14</td><td>14</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>Fdns.</td><td></td><td></td><td>873</td><td>462</td><td>1273</td><td>1182</td><td></td><td></td><td></td><td></td><td></td></tr></table>	LOAD CASE	BEAM LOADS kN		COLUMN DESIGN LOADS kN				COL. MOMENTS kNm				TOTAL		IMPOSED		DEAD		TOP		BOTTOM		1	2	1	2	1	2	1	2	1	2	LEVEL												Roof	249	244	54	53	195	191	34	54					210	133	46		164	133			32	58		SW.					9	9									100	53	368	333						3rd Fl.	298	290	140	136	158	154	32	58					249	117	117		132	117			32	58		SW.					9	9									357	189	667	613						2nd Fl.	298	290	140	136	158	154	32	58					249	117	117		132	117			35	65		SW.					9	9									614	325	966	893						1st Fl.	300	292	141	137	159	155	19	34					252	120	118		134	120			-	-		SW.					14	14						Fdns.			873	462	1273	1182						<div><div><div>max.</div><div>max.</div></div><div><div>8000</div><div>6000</div></div></div> <div>LOAD CASE 1</div> <div><div><div>max.</div><div>min.</div></div><div><div>8000</div><div>6000</div></div></div> <div>LOAD CASE 2</div>
LOAD CASE	BEAM LOADS kN		COLUMN DESIGN LOADS kN				COL. MOMENTS kNm																																																																																																																																																																																																																																						
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3.8.1.6 Eqn. 30	(Foundation → 1st. Floor). Effective height $l_e = \beta l_0$																																																																																																																																																																																																																																												
Table 3.19	N → S $\beta = 0.9$ (End condition: top 1, bottom 3) $l_{ex} = 0.9 \times 4.5 = 4.05m$ $\frac{l_{ex}}{h} = \frac{4.05}{0.3} = 13.5$																																																																																																																																																																																																																																												
	E → W $\beta = 0.95$ (End condition: top 2, bottom 3) $l_{ey} = 0.95 \times 4.825 = 4.58m$ $\frac{l_{ey}}{b} = \frac{4.58}{0.3} = 15.26 > 15$																																																																																																																																																																																																																																												
3.8.1.3		∴ SLENDER COLUMN																																																																																																																																																																																																																																											

BS 8110 ref.	CALCULATIONS	OUTPUT
BS 6399 Part 1  3.8.3.2  Eqn. 32 34 35  3.8.2.4 3.8.3.2 fig. 3.20    3.8.1.1 3.8.3.1  Eqn. 33          BS 6399 Part 1  Eqn. 36          3.12.3.7 2.4.3.2	<p><u>INTERNAL COLUMN</u> (Foundation → 1st. Fl) continued</p> <p><u>Load Case 1</u></p> <p>Imposed = <math>100 + 0.8 \times 773 = 718</math></p> <p>Dead = <math>1273</math></p> <p>N = <math>1991 \text{ kN}</math>.</p> <p><math>M_1 = 0, M_2 = 19, 0.4M_2 = 7.6 \text{ kNm}</math></p> <p><math>M_i = 0.4M_1 + 0.6M_2 = 0 + 0.6 \times 19 = 11.4 &gt; 7.6</math></p> <p><math>M_{add} = \frac{Nh}{2000} \left(\frac{e_c}{b'}\right)^2 = \frac{1991 \times 0.3 \times 13.5^2}{2000} = 54.4 \text{ kNm}</math></p> <p>Min. eccentricity = <math>e_{min} = 0.05 \times 300 = 15 \text{ mm}</math></p> <p>Max. design moment will be greatest of:</p> <p>(a) <math>M_2 = 19 \text{ kNm}</math></p> <p>(b) <math>M_i + M_{add} = 11.4 + 54.4 = 65.8 \text{ kNm}</math></p> <p>(c) <math>e_{min} \times N = \frac{15}{1000} \times 1991 = 29.9 \text{ kNm}</math></p> <p><math>\frac{N}{bh} = \frac{1991 \times 10^3}{300^2} = 22.1, \quad \frac{M}{bh^2} = \frac{65.8 \times 10^6}{300^3} = 2.44</math></p> <p>Assume <math>d = 300 - 40 - 13 = 247 \text{ mm}</math></p> <p><math>\frac{d}{h} = \frac{247}{300} = 0.82, \quad \frac{100A_{sc}}{bh} = 2.8, \quad A_{sc} = 2520 \text{ mm}^2</math></p> <p><math>N_{bal} = 0.25 f_{cu} b d = 0.25 \times 40 \times 300 \times 247 \times 10^{-3} = 741 \text{ kN}</math></p> <p>Since <math>N &gt; N_{bal}</math> try 4T25 (1960 mm<sup>2</sup>)</p> <p><math>N_{uz} = (0.45 \times 40(300^2 - 1960) + 0.87 \times 460 \times 1960) \times 10^{-3} = 2369 \text{ kN}</math></p> <p><math>K = \frac{2369 - 1991}{2369 - 741} = 0.23</math></p> <p>Design Moment = <math>11.4 + 0.23 \times 54.4 = 23.9 &lt; 29.9 \text{ kNm}</math></p> <p><math>\frac{100A_{sc}}{bh} = \frac{100 \times 1960}{300^2} = 2.18, \quad \frac{N}{bh} = 22.1 \text{ (as before)}</math></p> <p>From chart, <math>\frac{M}{bh^2} = 1.6, \quad M = 43.2 &gt; 29.9 \text{ kNm}</math> ok.</p> <p>Check, considering: load case 1 above 1st floor, and load case 2 and moment at 1st floor.</p> <p>Imposed = <math>100 + 0.8(514 + 137) = 621</math></p> <p>Dead = <math>(966 + 289) = 1255</math></p> <p>N = <math>1876 \text{ kN}</math></p> <p><math>M_1 = 0, M_2 = 34 \text{ kNm}</math></p> <p><math>M_i = 0 + 0.6 \times 34 = 20.4 \text{ kNm}</math></p> <p><math>M_{add} = \frac{1876 \times 54.4}{1991} = 51.3 \text{ kNm}, \quad k = \frac{2369 - 1876}{2369 - 741} = 0.3</math></p> <p>Design Moment = <math>20.4 + 0.3 \times 51.3 = 35.8 &gt; M_2</math></p> <p><math>\frac{N}{bh} = 20.8, \quad \frac{100A_{sc}}{bh} = 2.18, \quad \frac{M}{bh^2} = 2.0</math></p> <p><math>M = 54 &gt; 35.8 \text{ kNm}</math> ok.</p> <p><u>TIE PROVISION</u> Load = <math>1.05(214 + 161/3) = 281 \text{ kN}</math></p> <p><math>A_s = 281 \times 10^3 / 460 = 610 \text{ mm}^2 &lt; 1960 \text{ mm}^2</math> ok.</p>	<p>Max. design Mt.</p> <p>65.8 kNm.</p> <p>Use <math>d/h = 0.8</math> (chart page 28) BS 8110 Part 3: 1985</p> <p>ok.</p>  <p>ok.</p> <p>ok.</p>

**EXTERNAL COLUMN (Foundation → Roof)**  
**AXIAL LOADING and MOMENT from ANALYSIS**

F1 (typical)

		BEAM LOADS kN.		COLUMN DESIGN LOADS kN.				COL. MOMENTS kNm.			
		TOTAL		IMPOSED		DEAD		TOP		BOTTOM	
LOAD CASE		1	2	1	2	1	2	1	2	1	2
LEVEL											
Roof	main	192	197	42	43	150	154	108	115		
	edge	99	99			99	99			98	105
	sw.					9	9				
				42	43	258	262				
3rd.	main	247	255	116	120	131	135	98	105		
	edge	125	125			125	125			98	105
	sw.					9	9				
				158	163	523	531				
2nd.	main	247	255	116	120	131	135	98	105		
	edge	125	125			125	125			108	117
	sw.					9	9				
				274	283	788	800				
1st.	main	245	253	115	119	130	134	71	76		
	edge	125	125			125	125			-	-
	sw.					11	11				
	Fdns			389	402	1054	1070				

3.8.1.6  
Eqn. 30  
Table 3.19

(Foundation → 1<sup>st</sup> floor)  
 Effective height  $l_e = \beta l_0$   
 $N \rightarrow S \quad \beta = 0.9$  (end condition: top 1, bottom 3)  
 $l_{ex} = 0.9 \times 3.5 = 3.15 \text{ m}$   
 $\frac{l_{ex}}{h} = \frac{3.15}{0.3} = 10.5$

3.8.1.3

$E \rightarrow W \quad \beta = 0.9$  (end condition: top 1, bottom 3)  
 $l_{ey} = 0.9 \times 3.65 = 3.285$   
 $\frac{l_{ey}}{b} = \frac{3.285}{0.3} = 10.95 < 15$

∴ SHORT COLUMN.

BS6399  
Part 1

Design using load case 2  
 Just above 1<sup>st</sup> floor,  $N = 800 + 43 + 0.9 \times 240 = 1059 \text{ kN}$ .  
 $M = 117 \text{ kNm}$ .

$$\frac{N}{bh} = \frac{1059 \times 10^3}{300^2} = 11.8, \quad \frac{M}{bh^2} = \frac{117 \times 10^6}{300^3} = 4.3$$

$$\text{Assume } d = 300 - 50 - 13 = 237 \text{ mm}, \quad \frac{d}{h} = \frac{237}{300} = 0.79$$

$$\frac{100 A_{sc}}{bh} = 2.3, \quad A_{sc} = 2070 \text{ mm}^2 \quad (4T25 = 1960 \text{ mm}^2)$$

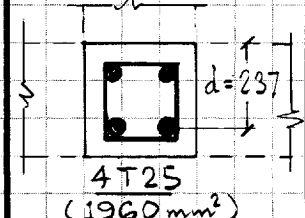
BS6399  
Part 1

Below 1<sup>st</sup> fl. → fdns,  $N = 1059 + 43 + 0.8 \times 359 = 1389 \text{ kN}$ .  
 $M = 76 \text{ kNm}$ .

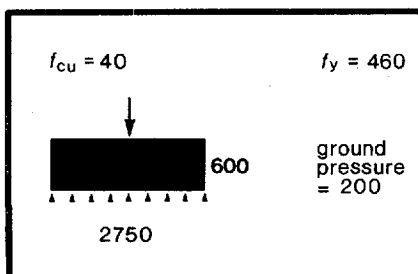
$$\frac{N}{bh} = 15.4, \quad \frac{M}{bh^2} = 2.81, \quad \frac{100 A_{sc}}{bh} = 1.5$$

$$A_{sc} = 1350 \text{ mm}^2$$

use  $\frac{d}{h} = 0.8$   
 (chart 1 page 28)  
 BS8110, Part 3, 985

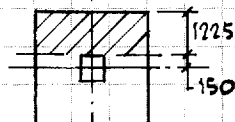
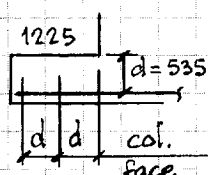
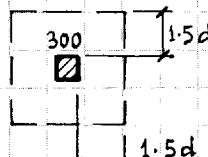


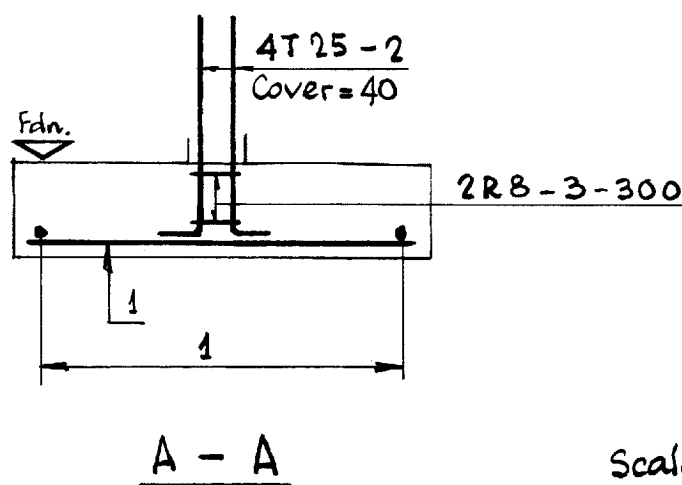
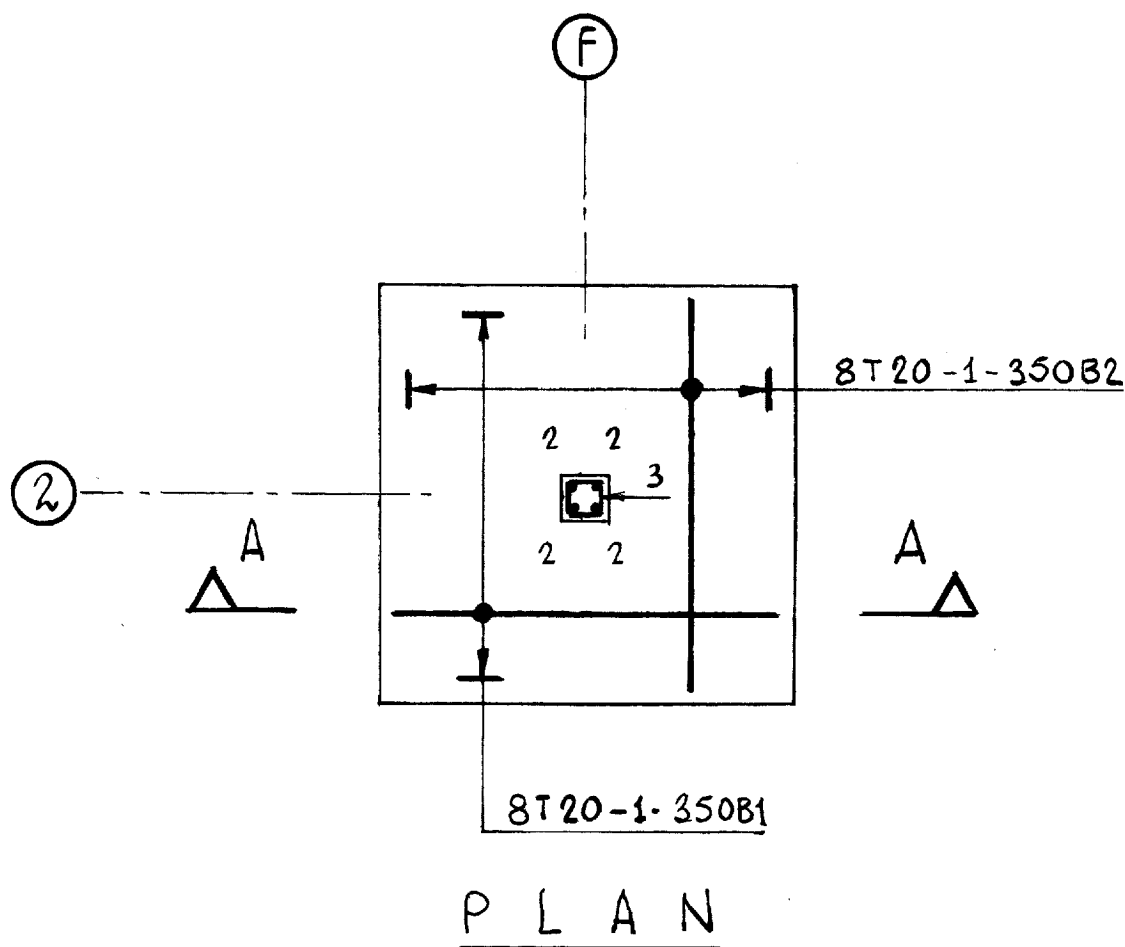




# Foundation

reinforced pad footing

BS 8110 ref.	CALCULATIONS				OUTPUT
Table 3.3 3.3.1.4	<b>DURABILITY.</b> Min. nominal cover, moderate exposure = 30 mm. Use nominal 40mm. cover against blinding.				Nominal cover 40mm. btm, 75mm. ends
	<b>LOADING - F2 (see page 19)</b>	Dead	Imposed	Tot. kN.	
	Internal Column - 'Design'	1273	718	1991	
	" " - 'Service'	1273/1.4 = 909	718/1.6 = 449	1358	
	Allow 10kN/m <sup>2</sup> extra over soil displaced by concrete and for gnd. fl. loading. ∴ Pad area required = $\frac{1358}{(200-10)} = 7.15 \text{ m}^2$ ∴ Adopt 2.75 x 2.75 x 0.6 thick ftg, area provided = $7.5 > 7.15 \text{ m}^2$ then U.L.S. 'Design' pressure = $\frac{1991}{2.75^2} = 263 \text{ kNm}^2$				
	<b>REINFORCEMENT</b>				
3.4.4.4	Moment at face of column	= $263 \times 2.75 \times \frac{1.225^2}{2}$	= 543 kNm		d = 535 mm.
	Average d	= 600 - 40 - 25	= 535 mm.		
	$\frac{M}{f_{cu} b d^2}$	= $\frac{543 \times 10^6}{40 \times 2750 \times 535^2}$	= 0.017		8T20@350 B. EW. (2512 mm <sup>2</sup> )
	$\frac{x}{d} = 0.95,$	$A_s = \frac{543 \times 10^6}{0.95 \times 460 \times 0.95 \times 535}$	= 2445 mm <sup>2</sup>		
3.4.5.4 3.11.3.4(1) 3.4.5.8	<b>ULTIMATE SHEAR</b> Minimum $v_c = 0.34 (40/25)^{1/3} = 0.4 \text{ N/mm}^2$ Condition 1: Shear force V @ 'd' from col. face = $\frac{1991}{2} \times \left( \frac{1375 - 150 - 535}{1375} \right) = 500 \text{ kN}$ " stress v " " " " = $\frac{500 \times 10^3}{2750 \times 535} = 0.34 \text{ N/mm}^2 < 2v_c$ Shear force V @ '2d' from col. face = $\frac{1991}{2} \times \left( \frac{1375 - 150 - 2 \times 535}{1375} \right) = 112 \text{ kN}$ " stress v " " " " = $\frac{112 \times 10^3}{2750 \times 535} = 0.08 \text{ N/mm}^2 < v_c$				
3.11.3.4(2) 3.7.7.2	Condition 2: (punching shear, usually the more critical) $V_{max} = \frac{V}{u_{od}} = \frac{1991 \times 10^3}{4 \times 300 \times 535} = 3.1 < 5.0 \text{ N/mm}^2$ Critical perimeter = $4(300 + 3 \times 535) = 7620 \text{ mm}$ Area within " " = $(0.3 + 3 \times 0.535)^2 = 3.63 \text{ m}^2$ Shear on " " = $263(2.75^2 - 3.63) = 1034 \text{ kN}$ $v = \frac{1034 \times 10^3}{7620 \times 535} = 0.25 \text{ N/mm}^2 < v_c$				
3.12.11.2.7	<b>CRACKING</b> clear distance = 330 < 750 mm ok $\frac{100 A_s}{b d} = \frac{100 \times 2445}{2750 \times 535} = 0.17 < 0.3\%$				∴ crack widths ok.
3.11.3.2	$l_c = 0.5 \times 2750 = 1375 < 0.25(3c + 9d) = 0.25(3 \times 300 + 9 \times 535) = 1429$ ∴ Space bars uniformly across section, say @ 350 ctrs. each way.				



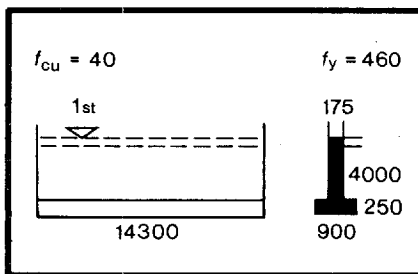
COVER:  
B1 = 40  
end = 75

Scale, 1:50

#### Commentary on bar arrangement

BS 8110 ref	Bar marks	Notes
3.12.8.1 Table 3.27	1	Straight bars extend full width of base, less end covers. Bars should project a minimum tension bond length beyond the column face = $35 \times 20 = 700$ mm < 1150 mm – OK.
3.3.1.4		The underside of base is concrete blinded, cover = 40 mm.
– Table 3.27	2	Column starter bars are wired to bottom mat. Minimum projection above the top of base is a compression lap + kicker = $35 \times 0.87/0.95 \times 25 + 75 = 875$ mm, i.e. lap = 800 mm (see p. 21).
–	3	Links are provided to stabilize and locate the starter bars during construction. These are the same size as the column links above.



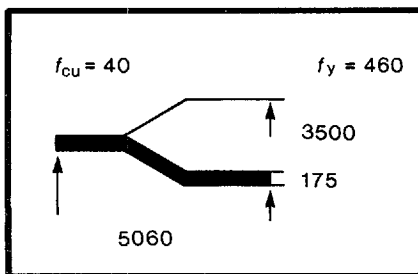


# Shear wall

external plain concrete wall

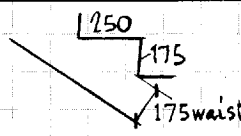
BS 8110 ref.	CALCULATIONS	OUTPUT
3.9.4.3 1.2.4	Design as braced plain concrete wall Eff. ht., $f_{dn} \rightarrow 1^{st}$ fl. $l_e = 0.875 \times 3825 = 3350$ mm, $\frac{l_e}{h} = \frac{3350}{175} = 19.2 > 12$	175 mm thick $\therefore$ SLENDER WALL
Table 3.3 fig 3.2	<u>DURABILITY and FIRE RESISTANCE</u> Nominal cover: (a) severe exposure = 40 mm. (b) mild = 20 mm. Fire resistance of 175 plain wall = $1\frac{1}{2} > 1$ hour	external cover = 40 mm internal " = 20 mm $\therefore$ 1 hr. fire resistance ok.
	<u>LOADING</u> e top of foundations Dead load from 1 <sup>st</sup> , 2 <sup>nd</sup> , 3 <sup>rd</sup> fl. + roof = $0.5(3 \times 23.5 + 28.5) = 49.5$ kN/m. self weight = $0.175 \times 24 \times 15.5 = 65.1$ Characteristic dead load = 114.6 kN/m " imposed " (slabs) = $2.5(1.5 + 3 \times 4 \times 0.8) = 27.8$ kN/m.	$\therefore q_k = 114.6$ kN/m. $\therefore q_{fk} = 27.8$ kN/m.
	<u>WIND LOADING</u> $V_s = V_b S_a = 22$ m/sec, $V_e = V_b S_b = 37.7$ m/sec Dynamic pressure $q_s = 0.613 V_e^2 = 0.613 \times 37.7^2 \times 10^{-3} = 0.87$ kN/m <sup>2</sup> Total wind load = $0.85 C_a (C_{pe, front} - C_{pe, rear}) q_s (1 + C_r) A$ = $0.85 \times 0.84 \times (0.8 + 0.3) \times 0.87 \times 1.025 \times 40.3 \times 15 = 423.4$ kN M end wall = $0.5 \times 423.4 \times 8.0 = 1694$ kNm, $f = \frac{1694 \times 6}{14.3^2} = \pm 49.7$ kN/m	BS 6399 Part 2 $\frac{D}{H} \leq 1$ $D = 14.3$ $H = 15$ $\alpha = 43^\circ$ $C_a = 0.84$ (fig 4)
2.4.3.1 Table 2.1	<u>VERTICAL LOADING INTENSITY</u> (U.L.S. Design Loads) Load combination 1. $f_1 = 1.4 \times 114.6 + 1.6 \times 27.8 = 204.9$ kN/m. " " 2. $f_2 = 1.4 \times 114.6 + 1.4 \times 49.7 = 230$ kN/m. " " 3. $f_3 = 1.0 \times 114.6 - 1.4 \times 49.7 = 45$ kN/m (min) " " 4. $f_4 = 1.2 \times (114.6 + 27.8 + 49.7) = 230.5$ kN/m (max)	
3.9.4.16 eqn 43 eqn 44	$e_x = \frac{h}{20}$ , $e_a = \frac{l_e^2}{2500h} = \frac{(19.2^2)}{2500} h = 0.15h$ $n_w \leq 0.3(h - \frac{2h}{20}) f_{cu} \leq 0.27h f_{cu} \leq 1890$ kN/m $n_w \leq 0.3(h - \frac{1.2h}{20} - 2 \times 0.15h) f_{cu} \leq 0.192h f_{cu} \leq 1344$ kN/m Actual U.L.S. Design Loads $< n_w$ in both cases	$\therefore$ ok
3.9.4.18	<u>SHEAR</u> Horizontal design shear force $V = 1.4 \times \frac{423.4}{2} = 296.4$ kN Min. vertical " load $N = 1.0 \times 114.6 \times 14.3 = 1639$ kN. $\frac{V}{N} = \frac{296.4}{1639} = 0.18 < 0.25$ , $\frac{V}{hl} = \frac{296.4 \times 10^3}{175 \times 14300} = 0.12 < 0.45$ N/mm <sup>2</sup>	$\therefore$ ok.
3.12.3.5 3.12.3.4.2 3.12.3.4.1 3.12.3.6.1	<u>TIE PROVISION @ 1st. Floor</u> Peripheral tie: $A_s = \frac{36 \times 10^3}{460} = 78$ mm <sup>2</sup> Internal tie force: $\frac{2.5 \times 36(4.7 + 4.0)}{7.5} \times \frac{14.3}{5} = 299$ kN, $A_s = \frac{299 \times 10^3}{460} = 650$ mm <sup>2</sup> Horizontal reinforcement in wall 0.5m above and below slab adequate Wall tie: $\frac{l_s}{2.5} = \frac{3.325 \times 36}{2.5} < 2ft$ but $> 0.03 \times 204.9$ kN, $A_s = \frac{47.9 \times 10^3}{460} = 104$ mm <sup>2</sup>	(78.5 mm <sup>2</sup> ) 1T10 T10 @ 200 EF. HORIZ. (393 mm <sup>2</sup> /m) use adjacent slab reinf.
Table 3.25	<u>FOOTING</u> Max. pressure due dead + imp + wind = $114.6 + 27.8 + 49.7 = 192$ kN/m If 900 wide, pressure = $\frac{192}{0.9} = 213$ kN/m <sup>2</sup> $< 1.25 \times 190$ $M = 230.5 \times \frac{0.725^2}{8} = 15.1$ kNm, $A_s = \frac{15.1 \times 10^6}{0.95 \times 460 \times 0.95 \times 200} = 182$ mm <sup>2</sup> If 250 thick, $d = 200$ Min. area = $0.13 \times 10^3 \times 250 = 325$ mm <sup>2</sup>	BS 8004 2.3.2.4 Bottom T12 @ 300 EW (377 mm <sup>2</sup> /m)

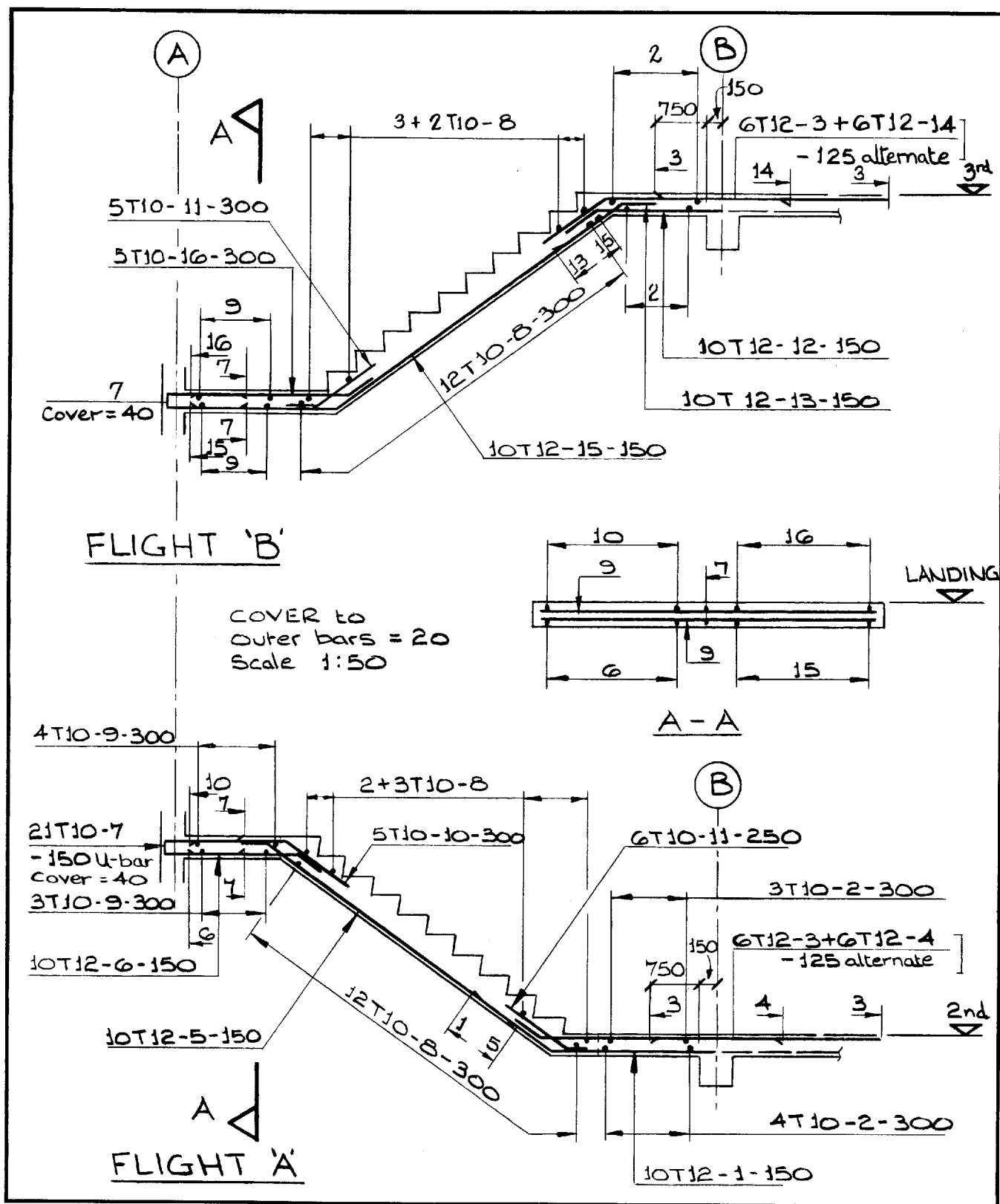




# Staircase

end-span continuous slab

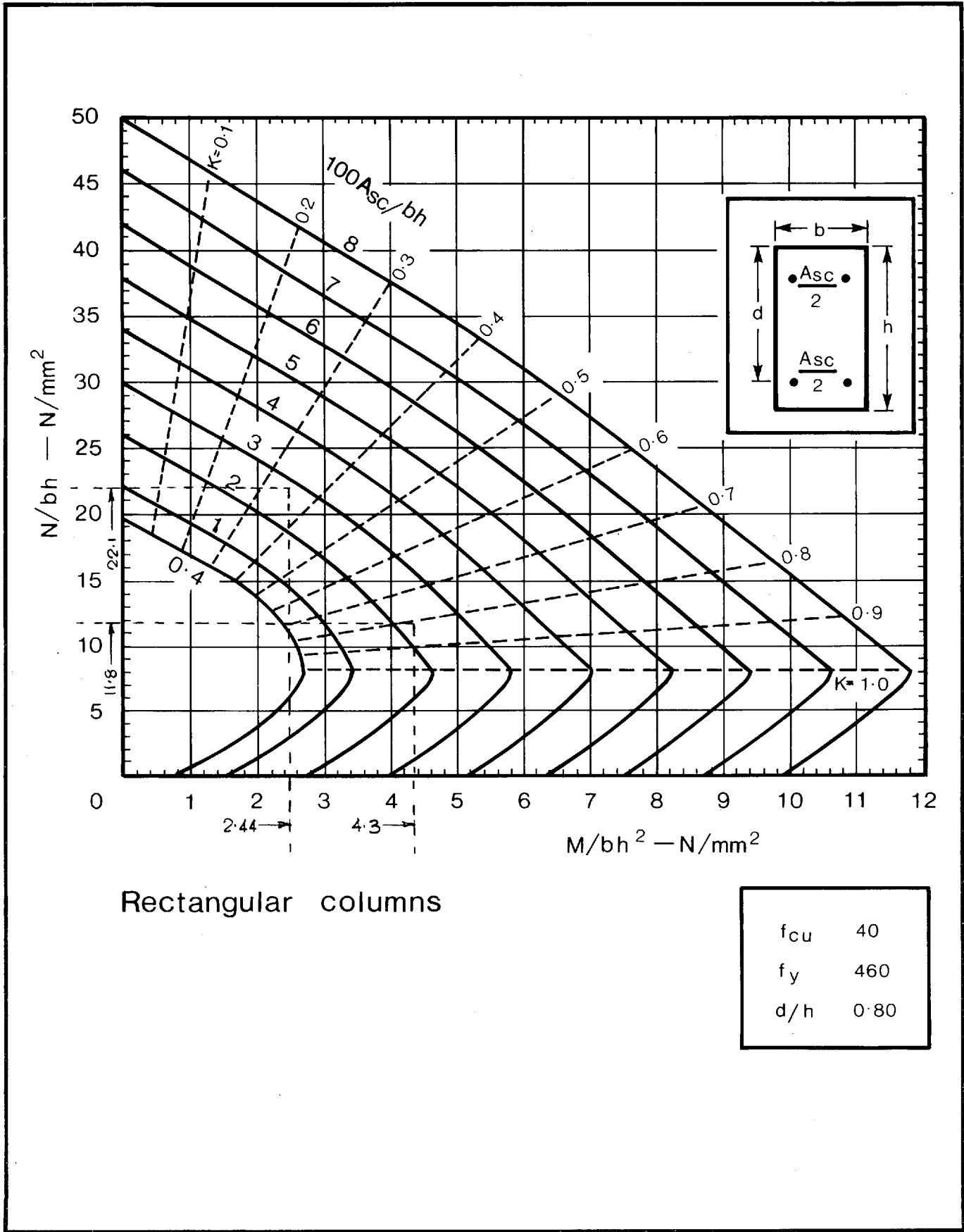
BS 8110 ref.	CALCULATIONS	OUTPUT
3.3 Tables 3.3, 3.4	<u>DURABILITY and FIRE RESISTANCE</u> (as floor slab, page 8)	cover = 20mm 1 hr. fire resistance
3.10.1.1 3.10.1.2	<u>LOADING</u> Average slab thickness on plan = 250 mm Self-weight $0.25 \times 24$ = 6.0 Finishes = 0.5 $\therefore$ Characteristic dead load = 6.5 kN/m <sup>2</sup> " imposed " (page 6) = 4.0 kN/m <sup>2</sup> Design load = $(1.4 \times 6.5 + 1.6 \times 4.0) 5.0$ = 77.5 kN/width	$g_k = 6.5 \text{ kN/m}^2$ $q_k = 4.0 \text{ kN/m}^2$ $F_k = 77.5 \text{ kN/width}$
3.4.3 Table 3.5	<u>ULTIMATE B.M'S</u> 1st. interior support = $0.11 Fl = 0.11 \times 77.5 \times 5.06$ = 43.1 kN/m Near mid-end span = $0.09 Fl$ = 35.3 kN/m	
3.4.4.4	<u>REINFORCEMENT</u> 1st. interior support, $\frac{M}{f_{cu} b d^2} = 0.049$ , $\frac{x}{d} = 0.94$ $A_s = \frac{M}{0.95 f_y z} = \frac{43.1 \times 10^6}{0.95 \times 460 \times 0.94 \times 149} = 704 \text{ mm}^2/\text{m}$ Near mid-end span, $\frac{M}{f_{cu} b d^2} = 0.040$ , $\frac{x}{d} = 0.95$ $A_s = \frac{M}{0.95 f_y z} = \frac{35.3 \times 10^6}{0.95 \times 460 \times 0.95 \times 149} = 571 \text{ mm}^2/\text{m}$	$d = 149 \text{ mm}$ TOP T12 @ 125 (905 mm <sup>2</sup> /m) BOTTOM T12 @ 150 (754 mm <sup>2</sup> /m)
Table 3.8	Check for Shear: $v = \frac{0.6 \times 77.5 \times 10^3}{10^3 \times 149} = 0.31 \text{ N/mm}^2 < V_c$ $\therefore$ ok.	
3.4.6 Table 3.10	<u>DEFLECTION</u> basic span/eff. depth ratio = 26 max. $\frac{M}{b d^2} = \frac{35.3 \times 10^6}{10^3 \times 149^2} = 1.59$ , $f_s = \frac{2 \times 460 \times 571}{3 \times 754} = 232.2 \text{ N/mm}^2$ Modification factor for service stress $232.2 \text{ N/mm}^2 = 1.37$ $\therefore$ Allowable span/eff. depth ratio = $26 \times 1.37 = 35.62$ Actual " " " = $\frac{5060}{149} = 34.0$	$\therefore l/d$ ratio ok.
3.12.11.2.7	<u>CRACKING</u> Spacing between bars $< 3 \times 149 = 447 \text{ mm}$ . $h = 175 < 200 \text{ mm}$ . $\therefore$ no further checks required	Spacing $< 3d$ . ok. ok.
3.12.3.4	<u>TIE PROVISION</u> East $\rightarrow$ West Internal Tie Min. area continuity reinf. = 91 mm <sup>2</sup> /m (as floor slab, p.8) $\therefore$ Total area req'd for width of staircase = $91 \times 3 = 273 \text{ mm}^2$ Provide 2 T12 tie bars each side in adjacent slab. LAP, as floor slab, p.8.	BOTTOM 4 T12 (456 mm <sup>2</sup> ) min. 300 mm. LAPS



#### Commentary on bar arrangement

BS 8110 ref	Bar marks	Notes
Table 3.27	1,5,6	Main tension reinforcement. Lap lengths and anchorage bond lengths = $35 \times 12 = 420$ mm, say 450 mm. Laps are located to facilitate likely construction sequences. Similar for bar marks 12, 13 and 15.
Table 3.25	2,8,9	Secondary reinforcement. Minimum area = $0.0013 \times 1000 \times 175 = 228 \text{ mm}^2/\text{m}$ . Use T10 @ 300 = $262 \text{ mm}^2/\text{m}$ .
Fig 3.25	3,4	Main tension reinforcement over support. 50% curtailed at 0.3 span, remainder at 0.15 span, both measured from face of support. Similar for bar mark 14.
3.12.10.3.2	7	U-bars provide 50% mid-span reinforcement in both top and bottom at end support = $0.5 \times 571 = 286 \text{ mm}^2/\text{m}$ . Use T10 @ 150 = $524 \text{ mm}^2/\text{m}$ to match spacing of span bars. Laps, say 450 mm.
Table 3.25	10,11	Optional reinforcement. Minimum area = $228 \text{ mm}^2$ . Similar for bar mark 16.

# Column design chart



## Information from the Reinforced Concrete Council

### Spreadsheets

Many of the design principles used in this publication will be covered by **spreadsheets for reinforced concrete design** now being developed by the Reinforced Concrete Council. Versions for both BS 8110 and EC2 are in preparation. For details write to the RCC at Century House, Telford Avenue, Crowthorne, Berks RG45 6YS.

### Buildability and whole building economics

It should be stressed that the structural solution presented in this publication has been chosen for the purposes of illustrating analysis, design and reinforcement detailing principles. A typical building frame accounts for only 10% of the whole construction cost, but affects foundations, cladding and service provision. The choice and details of a building's structure should reflect both buildability and overall building economics. Analysis of these factors using a structural optimisation program\* or charts from a publication\*\* suggests that a flat slab alternative may save around 2% of overall building costs and ten days' construction time.

Similarly, rationalisation and simplification of reinforcement will normally speed construction and hence reduce overall construction costs and programme time. Excessive curtailment and tailoring of reinforcement to save material at the expense of rationalisation will prove counter-productive. These aspects are currently being investigated at the European Concrete Building Project at Cardington, and will result in the publication of best practice guidance.

With increasing emphasis on the cost in use of buildings, there is a trend towards the use of exposed soffits for passive cooling. This move to whole life costs will modify the optimum solution, and deep ribbed or coffered slabs are a favoured option to meet daylighting, thermal mass, ventilation and acoustic requirements.

\**Concept* - a computer program that allows the rapid semi-automated choice of concrete frame while considering whole building costs. Produced by the Reinforced Concrete Council. Available from the RCC on 01344 725733.

\*\* *Economic concrete frame elements* - a pre-scheme design handbook, based on BS 8110, that helps designers choose the most viable concrete options. Produced by the Reinforced Concrete Council. Available from the BCA on 01344 725704.

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