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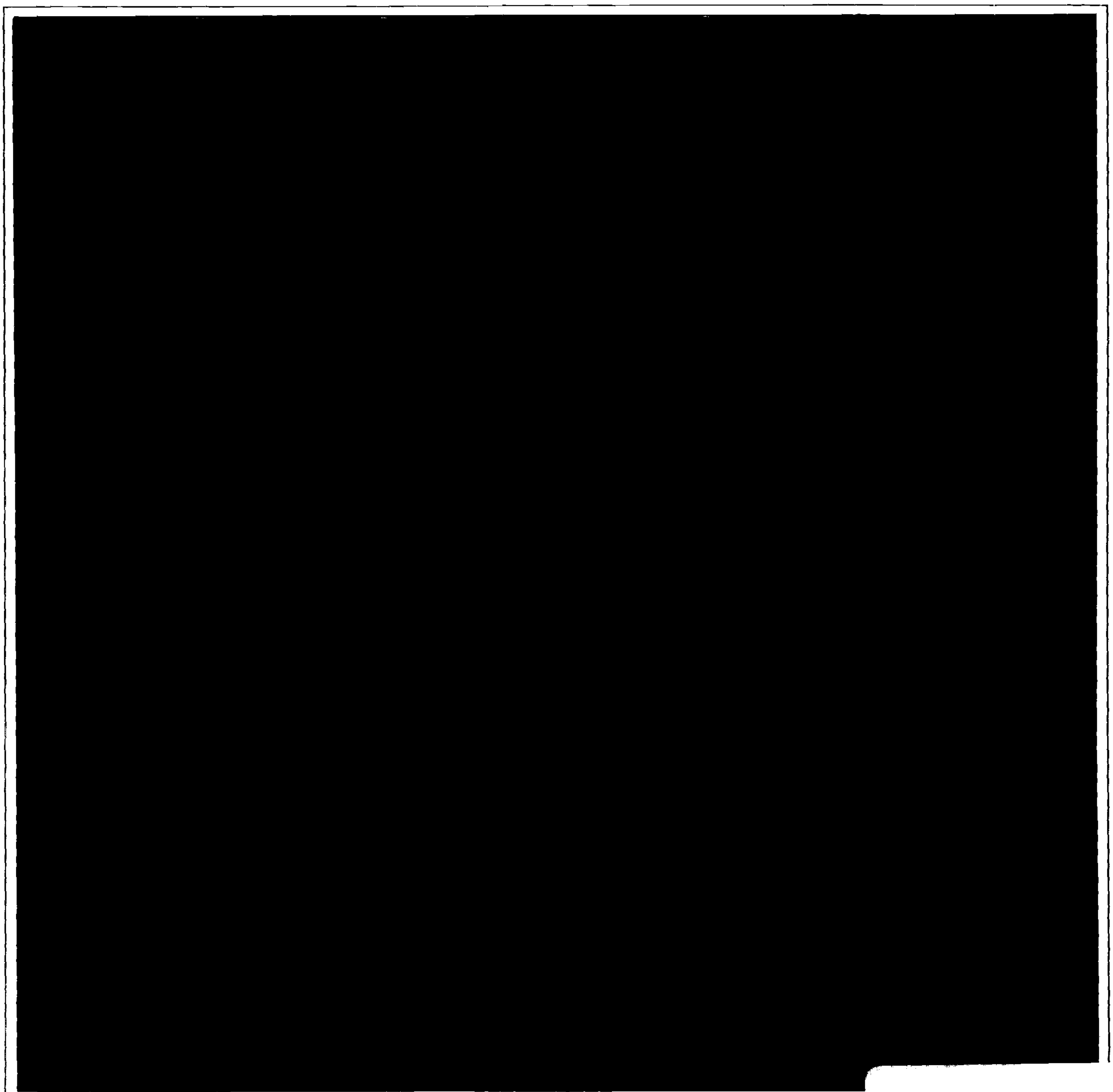
Designed and detailed (BS 8110: 1997)

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



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Contents

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 γ_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 $0{\cdot}87f_y$.

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 × 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 γ_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 γ_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 γ_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				
	dead load		imposed load		wind 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 γ_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 γ_m to be used for the two materials when designing for the ultimate limit state are given below:

Values of γ_m for the ultimate limit state	
Reinforcement	1.05
Concrete	
Flexure or axial load	1·5
Shear strength without shear reinforcement	1·25
Bond strength	1·4
Others (e.g. bearing stress)	$\geqslant$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

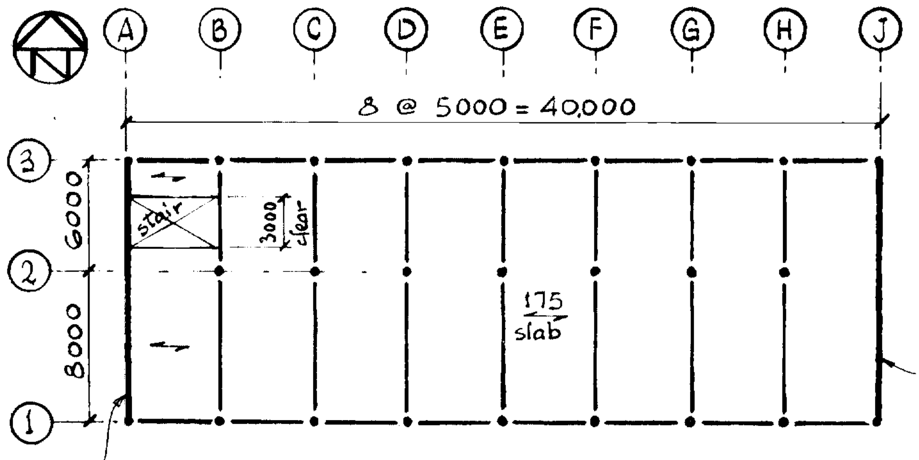
Durability and fire resistance requirements are considered at the onset of the design process because this determines the grade of concrete, the cover, and the size of the members. Usually, for most structures, Part 1 of the Code will be used in which it is assumed that the ultimate limit state will 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

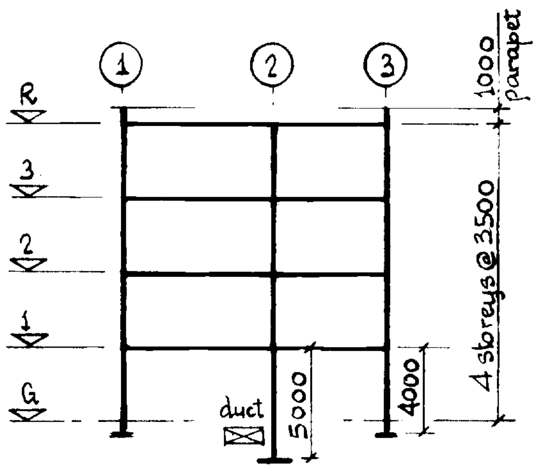
Client / Architect: National Company plc	Engineer responsible: BRRogers
London 9/11/98	Building Regulation authority or other and Date of submission
1. The Building Regulations 1991 2. BS8110 The Structural Use of Concrete, Part 1 1997 3. BS6399 Part 1 1996, Part 2 1995, and Part 3 1988	Relevant Building 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/m² " - finishes = 1.5 kN/m² Floors - imposed (3.0) and partitions (1.0) = 4.0 kN/m² Stairs - imposed = 4.0 kN/m² - finishes to Floors and Stairs = 0.5 kN/m² External Cladding = 5.0 kN/m.	General loading conditions
Speed: 21 m/sec. (basic) Factors: $S_a = 1.05$ $S_b = 1.71$, $S_d = 1.0$, $S_s = 1.0$, $S_p = 1.0$ $C_a = 0.84$, $C_{pe} = +0.8$ (front), $C_{pe} = -0.3$ (rear), $C_f = 0.025$	Wind loading conditions
Severe (external) and Mild (internal) (BS8110 Table 3.2)	Exposure conditions
Stiff clay - no sulphates Allowable bearing pressure = 200 kN/m²	Subsoil conditions
RC footings to columns and walls	Foundation type
Grade 40 with 20mm. max. aggregate (BS8110 Table 3.3) Characteristic strength - main bars, f_y = 460 N/mm² " " - links = 250 N/mm²	Material data
Self weight of concrete = 24.0 kN/m³ All dimensions shown on drawings are in millimetres (mm)	Other relevant information

Structural summary sheet



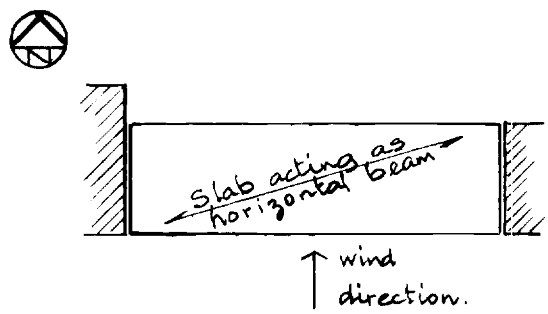
Notes:
1. All cols 300×300
2. Main beams 500×300
3. Edge beams 350×300

TYPICAL FLOOR PLAN.



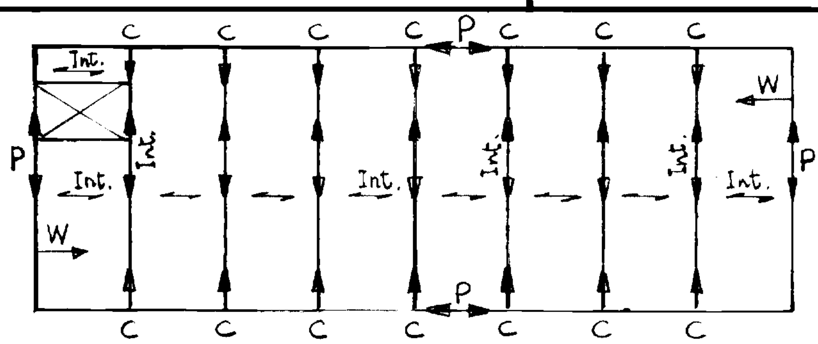
TYPICAL CROSS-SECTION.

Note: Internal col. ftgs. 2750×2750×600



WIND RESISTANCE

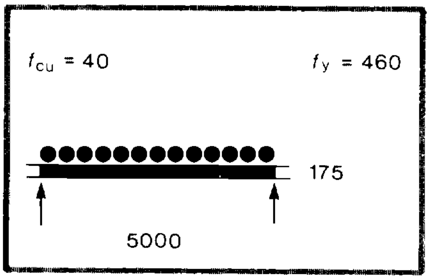
Notes:
1. N→S wind force resisted by end shear walls.
2. Lateral bracing in E→W direction provided by staircase, and infill masonry panels, on grid 1/H-J, and grid 3/H-J.



Key:
P - Peripheral tie
Int. - Internal tie
C - Column (external) tie.
W - Wall " "

3.12.3 TIE PROVISION - horizontal ties

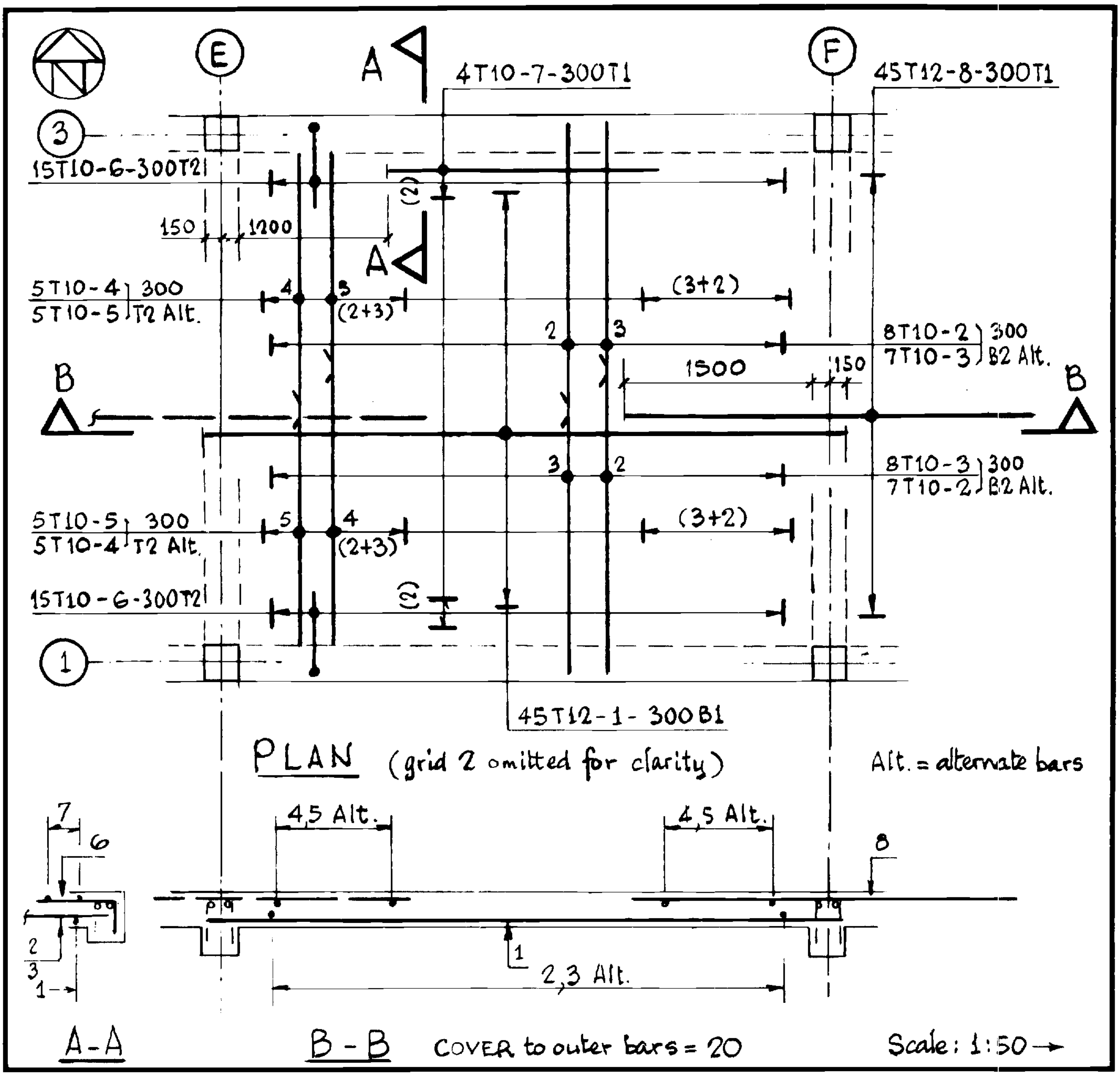
3.12.3.4.2 $F_t = (20 + 4n_o) = 20 + 16 = 36 < 60\,kN.$



Floor slab

interior-span solid slab

BS 8110 ref.	CALCULATIONS	OUTPUT
3.3 Table 3.3 " 3.4	DURABILITY and FIRE RESISTANCE Nominal cover for mild conditions of exposure = 20 mm. Max. fire resistance of 175 slab with 20 cover = $1\frac{1}{2}$ hr > 1 hr.	cover = 20 mm. ∴ 1 hr. fire resistance ok
    3.5.2.4 Table 3.12	LOADING Self-weight 0.175×24 = 4.2 Finishes (page 6) = 0.5 Characteristic dead load = 4.7 kN/m² " imposed " (page 6) = 4.0 kN/m² Design load $= (1.4 \times 4.7 + 1.6 \times 4) \times 5.0$ = 64.9 kN/m width	 g_k = 4.7 kN/m² q_k = 4.0 kN/m² F = 64.9 kN/m width
3.5.2.4 Table 3.12	ULTIMATE B.M's Interior mid-span and supports $0.063Fl = 0.063 \times 64.9 \times 5.0 = 20.4$ kNm/m	
3.4.4.4 Table 3.8	REINFORCEMENT Interior mid-span and supports $K = \frac{M}{f_{cu}bd^2} = \frac{20.4 \times 10^6}{40 \times 10^3 \times 149^2} = 0.023$ $z = 149\left(0.5 + \sqrt{\left(0.25 - \frac{0.023}{0.9}\right)}\right) = 145$ (but $\not> 0.95 \times 149 = 141.5$) $A_s = \frac{M}{0.95 f_y z} = \frac{20.4 \times 10^6}{0.95 \times 460 \times 141.5} = 330$ mm²/m Check for Shear: $v = \frac{0.5 \times 64.9 \times 10^3}{10^3 \times 149} = 0.22$ N/mm² $< v_c$	d = 149 mm. Top & Bottom T12 @ 300 (377 mm²/m) ∴ ok.
3.4.6 Table 3.9 Table 3.10	DEFLECTION basic span/eff. depth ratio = 26 max. $\frac{M}{bd^2} = \frac{20.4 \times 10^6}{10^3 \times 149^2} = 0.92$. $f_s = \frac{2 \times 460 \times 330}{3 \times 377} = 268.4$ N/mm² Modification factor for tension reinft. = 1.5 ∴ Allowable span/eff. depth ratio = 26×1.5 = 39 ∴ Actual " " " = $\frac{5000}{149}$ = 33.56	 ∴ l/d ratio ok.
3.12.11.2.7	CRACKING 3d = 3×149 = 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	TIE PROVISION 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 $= \frac{41.8 \times 10^3}{460} = 91$ mm²/m. LAP $= \frac{91 \times 35}{377}\phi = 8.5\phi \not> 300$ mm.	Lap 300 mm @ supports Bottom T12 @ 300 (377 mm²/m)

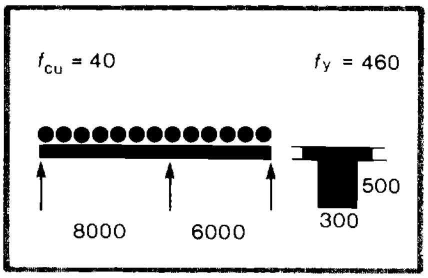


Commentary on bar arrangement

BS 8110 ref	Bar marks	Notes
		All bars are labelled in the form described in the *Standard method of detailing structural concrete*, e.g. 45T12-1-300B1 means that in the bottom outer layer there are 45 Grade 460 Type 2 deformed 12 mm nominal size bars at 300 mm centres and the bar mark is -1-. The bars are numbered in the likely sequence of fixing; the positions of the first and last bars in a string are indicated in plan and section. Intermediate bars have been omitted for clarity.
Table 3.25		Minimum area of tension reinforcement = 0·0013 × 1000 × 175 = 228 mm²/m.
3.12.11.2.7		Maximum clear spacing of tension bars = lesser of 750 mm or $3d$, i.e. $3d = 3 \times 149 = 447$ mm. $h < 200$, therefore no further check on spacing
–	1	Main tension bars T12 @ 300, $A_s = 377$ mm² > minimum 228 mm²/m. – OK. If curtailed, $A_s = 377/2 = 189$ mm² < minimum 228 mm²/m – not OK.
3.12.3.4		Bars lapped 300 mm at bottom support to provide continuous tie.
Table 3.25	2,3	Secondary bars – use T10 @ 300 (262 mm²/m).
3.12.8.11	4,5	Minimum lap = 300 mm > 15 × 10 = 150 mm. Lapping reduces bar lengths for easier handling on site.
–	7	Laps are shown staggered for effective crack control.
3.4.1.5	6	Minimum transverse reinforcement is placed across the full flange width of the edge beam (minimum width = 650 mm, see page 16).
Table 3.25		Minimum area = 0·0015 × 1000 × 175 = 263 mm²/m – use T10 @ 300 (262 mm²/m).
–	8	Main tension bars over support T12 @ 300 as bar mark 1.
3.12.10.3		One curtailment shown at 0·3 effective span from face of support. Further curtailments prevented by minimum area and spacing requirements similar to mark 1.

First-floor main beam

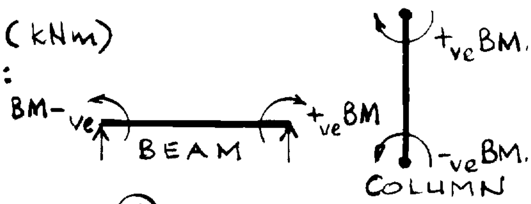
two-span flanged beam



BS 8110 ref.	CALCULATIONS	OUTPUT
3.2.1.2.1	**SUBFRAME ANALYSIS** 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.	
3.3 Tables 3.3, 3.4	**DURABILITY and FIRE RESISTANCE** Nominal cover for mild conditions of exposure = 20 mm Nominal cover for 300 mm wide beam for 1hr. period = 20 mm.	Minimum cover to LINKS 20 mm.
3.2.1.2.2	**LOADING** Dead load from 175 slab (page 8) = 5×4.7 = 23.5 Self-weight (0.5−0.175) 0.3 × 24 = 2.3 ∴ Characteristic dead load on beam 25.8 kN/m. " imposed " " " (p.8) = 5×4 = 20.0 kN/m. Maximum design load = $1.4g_k + 1.6q_k$ = 36.12 + 32 = 68.12 kN/m. Minimum " " = $1.0g_k$ = 25.8 kN/m	$1.0g_k$ = 25.8 kN/m. $1.0q_k$ = 20.0 kN/m. $1.4g_k + 1.6q_k$ = 68.12 kN/m.

BENDING MOMENTS (kNm)

Sign convention used:

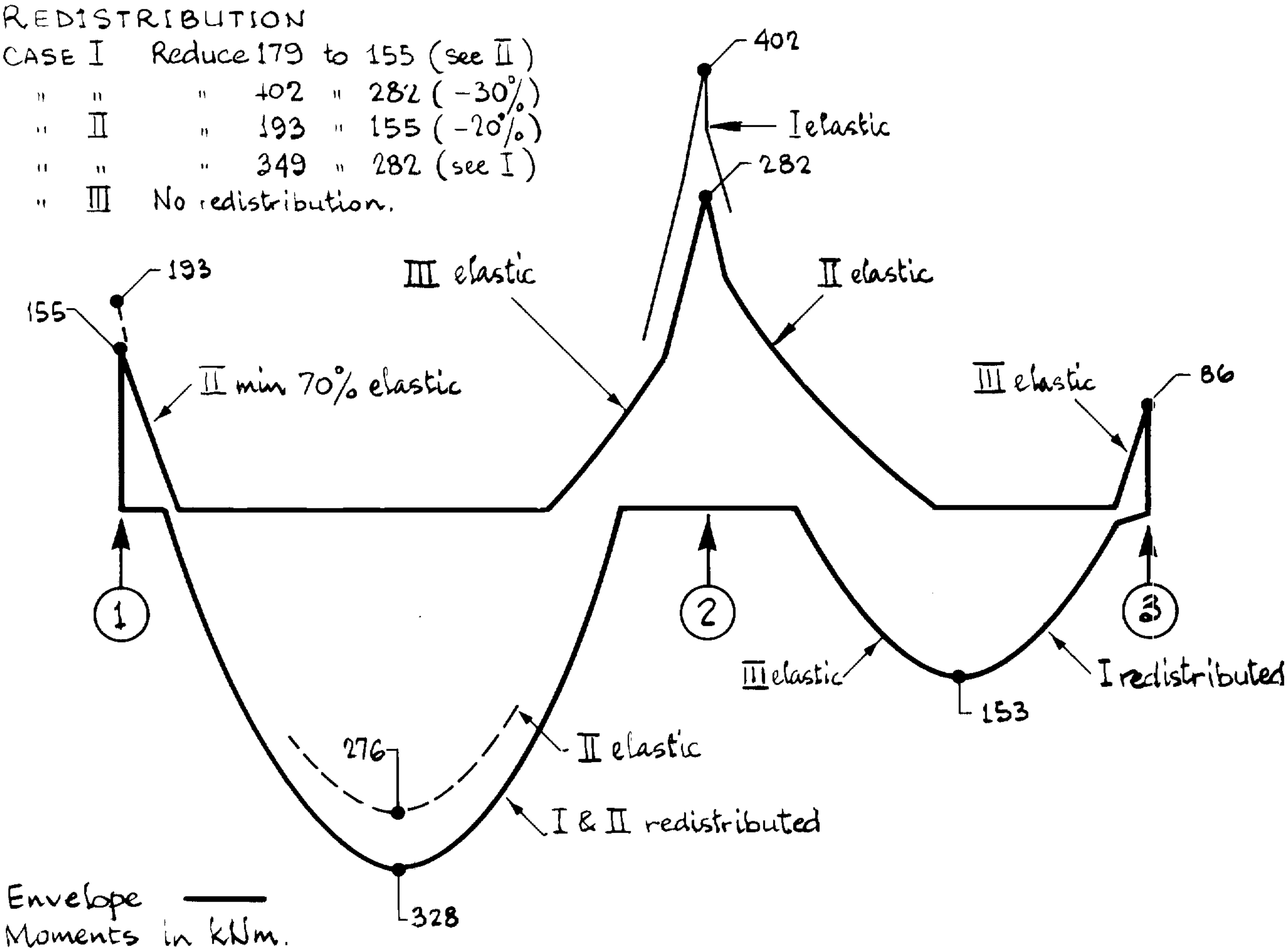


Results from Analysis

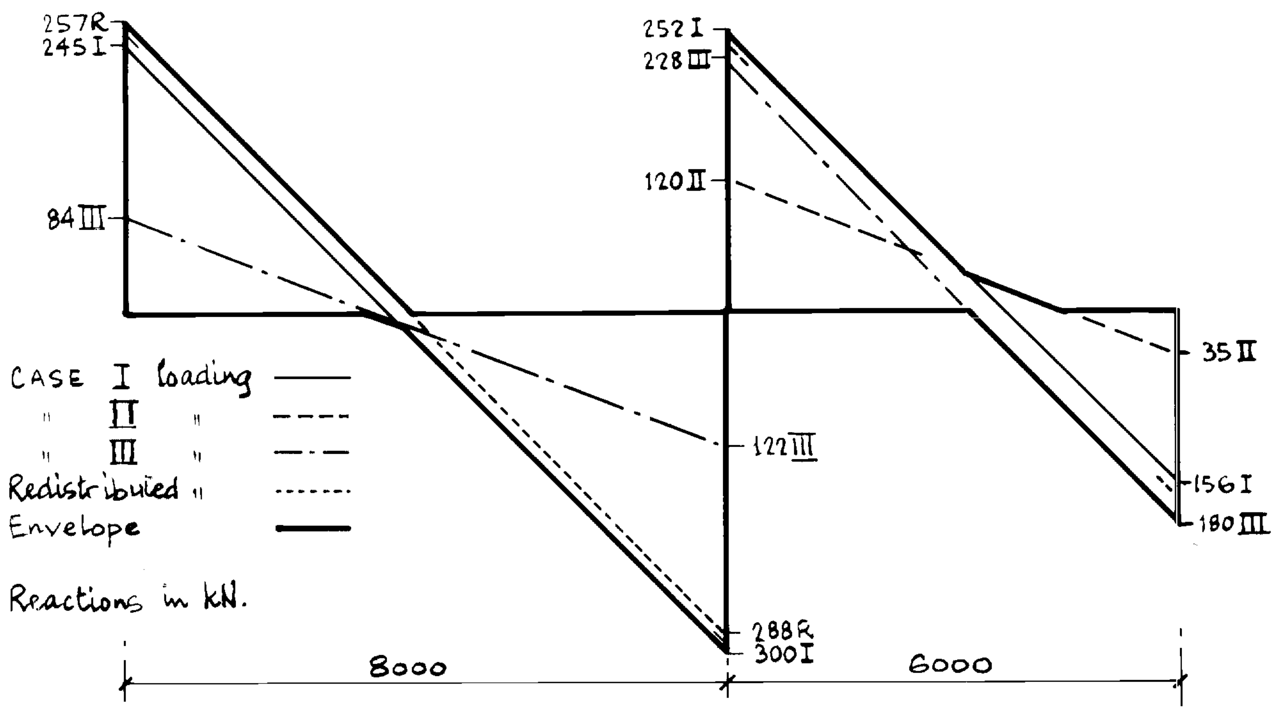
	8m.		②	6m.		
I	68.12			68.12		← Spans CASE I loading
− 179	260	+ 402	− 348	120	+ 60	Beam moments
+ 108			− 35		− 36	Upper Column. mts.
+ 71			− 19		− 24	Lower " "
245		300	252		156	Shear (kN)
II	68.12			25.8		CASE II loading
− 193	276	+ 349	− 250	28	− 4	Beam moments
+ 117			− 65		+ 2.5	Upper Column mts.
+ 76			− 34		+ 1.5	Lower " "
253		292	120		35	Shear (kN)
III	25.8			68.12		CASE III loading
− 54	84	+ 204	− 230	153	+ 86	Beam moments
+ 33			+ 17		− 52	Upper Column mts.
+ 21			+ 9		− 34	Lower " "
84		122	228		180	Shear (kN)

REDISTRIBUTION

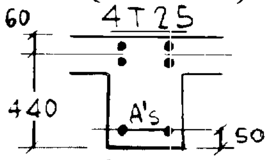
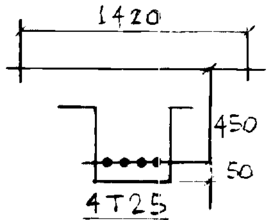
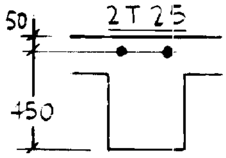
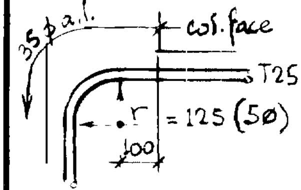
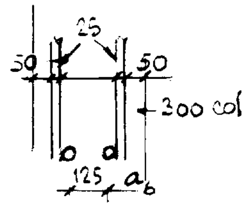
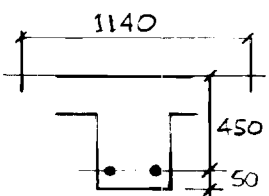
CASE I Reduce 179 to 155 (see II)
" " " 402 " 282 (−30%)
" II " 193 " 155 (−20%)
" " " 349 " 282 (see I)
" III No redistribution.



3.2.2 BENDING MOMENT ENVELOPE



SHEAR FORCE ENVELOPE

BS 8110 ref.	CALCULATIONS	OUTPUT
	MAIN BEAM, 1st. FLOOR (continued)	
	Internal Support:	
	From B.M. envelope, page 11, $M = 282$ kNm.	
3.2.2.1	$\beta_b = 282/402 = 0.7$, $\quad x/d \leqslant 0.3$	
3.4.4.4	$K' = 0.402(0.7-0.4) - 0.18(0.7-0.4)^2 = 0.104$	main tension
	$K = \frac{M}{f_{cu}bd^2} = \frac{282\times10^6}{40\times300\times440^2} = 0.121 > 0.104$	(1960 mm²) 4T25
	Comp. reinforcement reqd, $d' = 50$, $d'/x = 0.38 > 0.37$	
	$x = (d-z)/0.45$, $\therefore z/d = 1 - 0.45\times0.3 = 0.865$	
	$A'_s = \frac{(0.121-0.104)\times40\times300\times440^2}{0.95\times460\times(440-50)} = 232\ \text{mm}^2$	2T25
Table 3.25	Check minimum $A'_s = 0.2(300\times500)/100 = 300\ \text{mm}^2$	(982 mm²)
	$A_s = \frac{0.104\times40\times300\times440^2}{0.95\times460\times0.865\times440} + 232 = \underline{1685}\ \text{mm}^2$	compression
	8m mid-span	
	From B.M. envelope, page 11, $M = 328$ kNm	
3.4.1.5	Eff. flange width $= b + (0.2\times0.7\ell) = 300 + 0.14\times8000 = 1420$ mm.	
3.4.4.4	$\frac{M}{f_{cu}bd^2} = \frac{328\times10^6}{40\times1420\times450^2} = 0.029$	
	$z/d = \left(0.5+\sqrt{0.25-\frac{0.029}{0.9}}\right) = 0.97 > 0.95\ \therefore x = \frac{450(1-0.95)}{0.45} = 50$ mm (N.A. in flange)	4T25
	$A_s = \frac{328\times10^6}{0.95\times460\times0.95\times450} = \underline{1756}\ \text{mm}^2$	(1960 mm²) main tension
	8m. end-support	
	From B.M. envelope, page 11, $M = 155$ kNm	tension
3.2.2.1	$\beta_b = 155/193 = 0.8$, $\quad x/d \leqslant 0.4$	(982 mm²) 2T25
3.4.4.4	$\frac{M}{f_{cu}bd^2} = \frac{155\times10^6}{40\times300\times450^2} = 0.064$	
	$z/d = \left(0.5+\sqrt{0.25-\frac{0.064}{0.9}}\right) = 0.92\quad \therefore \frac{x}{d} = \left(\frac{1-0.92}{0.45}\right) = 0.18$ (ok)	
	$A_s = \frac{155\times10^6}{0.95\times460\times0.92\times450} = \underline{857}\ \text{mm}^2$	
	Check force in bar $= \frac{775\times0.95\times460\times491}{875\times10^3} = 190$ kN.	
3.12.8.25.2	$\therefore$ Int. bend radius, $r = \frac{190\times10^3(1+50/125)}{2\times40\times25} = 133$, say 125 mm.	
	6m. end-support	
	From B.M. envelope, page 11, $M = 86$ kNm.	
	$\frac{M}{f_{cu}bd^2} = 0.35$, $\quad z/d = 0.95$	
	$A_s = \frac{86\times10^6}{0.95\times460\times0.95\times450} = \underline{460}\ \text{mm}^2$	
	6m. mid-span	
	From B.M. envelope, page 11, $M = 153$ kN.	
3.4.1.5	Eff. flg. width $= 300 + 0.14\times6000 = 1140$ mm.	
	$\frac{M}{f_{cu}bd^2} = 0.017$, $\quad z/d = 0.95$	
	$A_s = \frac{153\times10^6}{0.95\times460\times0.95\times450} = \underline{819}\ \text{mm}^2$	
Table 3.25	Check for minimum reinforcement in flanged beam	2T25
	web in tension $0.0018\times300\times500 = 270\ \text{mm}^2$ ok	(982 mm²)
	flange in tension over supports, $0.0026\times300\times500 = 390\ \text{mm}^2$ ok	main tension

BS 8110 ref.	CALCULATIONS	OUTPUT

3.4.5 — **SHEAR REINFORCEMENT** — Output: $d = 450$ mm.

Min. effective tension reinft. = 2T25 ($982\ mm^2$)

3.4.5.4, Table 3.8: $\frac{100A_s}{bd} = \frac{100\times982}{300\times450} = 0.73$, $v_c = 0.57\left(\frac{40}{25}\right)^{\frac{1}{3}} = 0.66\ N/mm^2$

Table 3.7: Min. links $= \frac{A_{sv}}{S_v} = \frac{b\times0.4}{0.95f_{yv}} = \frac{300\times0.4}{0.95\times250} = 0.51$

3.4.5.5: Max. $S_v = 0.75\times450 = 338$, say 300 mm. Try R12@300 as min.

Table 3.7: $\frac{A_{sv}}{S_v} = \frac{226}{300} = 0.75$ $\therefore (v - v_c) = \frac{0.75\times0.4}{0.51} = 0.59\ N/mm^2$

Table 3.7:

Select R12 Links	A_{sv}	S_v	300	175	150	125
	$226\ mm^2$	A_{sv}/S_v	0.75	1.29	1.50	1.81

Table 3.7, 3.4.5.10:

Location	V kN @ eff. d from support face.	$v = V/bd\ N/mm^2$	$v - v_c$	A_{sv}/S_v	Links (Output)
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 kN	1.25	0.59	0.75	R12 @ 300

3.4.6.1 — **DEFLECTION** — basic l/d ratio = 20.8 max. ($\frac{b_w}{b} < 0.3$)

Table 3.9, 3.4.6.5: $\frac{M}{bd^2} = \frac{328\times10^6}{1420\times450^2} = 1.14$, $f_s = \frac{2\times460\times1756\times276}{3\times1960\times328} = 231.2\ N/mm^2$

Table 3.10: Modification factor = 1.55, $\therefore$ Allowable $l/d = 20.8\times1.55 = 32.24$

Actual $l/d = \frac{8000}{450} = 17.8$ — Output: $\therefore$ l/d ratio ok.

3.12.11.2 — **CRACKING** limit crack widths by limiting bar spacing

Ref.	Tension reinf.		M redistr. %	mm actual	mm allowed	mm actual	mm allowed
Table 3.28	8m. external support	T	-20	100	< 125	-	-
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	-	-

Output: $\therefore$ crack widths ok

3.12.3 — **TIE PROVISION** F_t = 36 kN/m width (see page 7)

a. Internal (N→S) tie force for 8m. span

3.12.3.4.2: $= F_t\left(\frac{g_k + q_k}{7.5}\right)\frac{l_r}{5} = 36\left(\frac{4.7+4.0}{7.5}\right)\frac{8}{5} = 66.8\ kN/m$ width

$\therefore$ Min. area continuity reinf. $= 66.8\times5\times\frac{10^3}{460} = \underline{726\ mm^2}$ — Output: 2T25 ($982\ mm^2$)

3.12.8.13b: Cover to lapped bars = 35 < (2×25) mm. Spacing between adjacent laps = 100 < (6×25) mm. $\therefore$ use factor 1.4

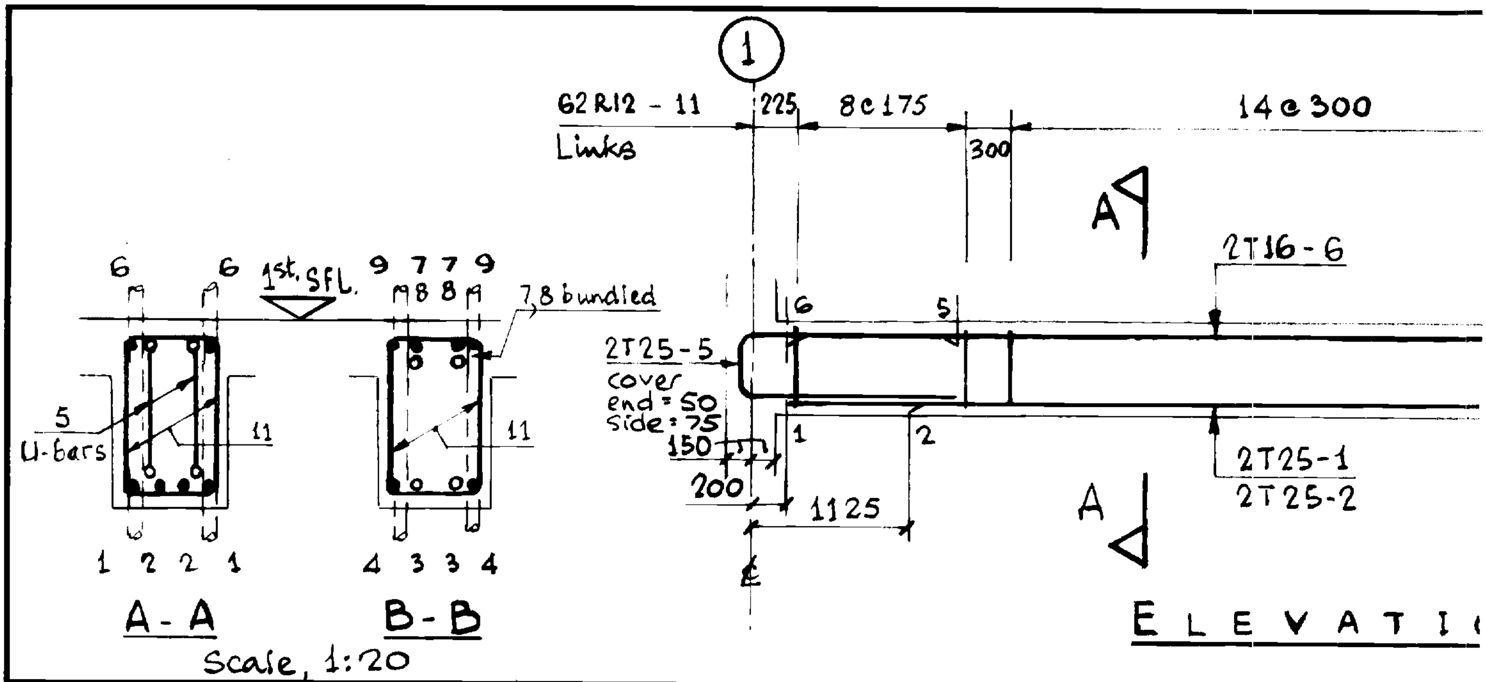
Table 3.27: $\therefore$ Lap $= 49\times25\times\frac{726}{982} = 906$ mm — Output: Use bottom bars already provided but made continuous using 1000 laps

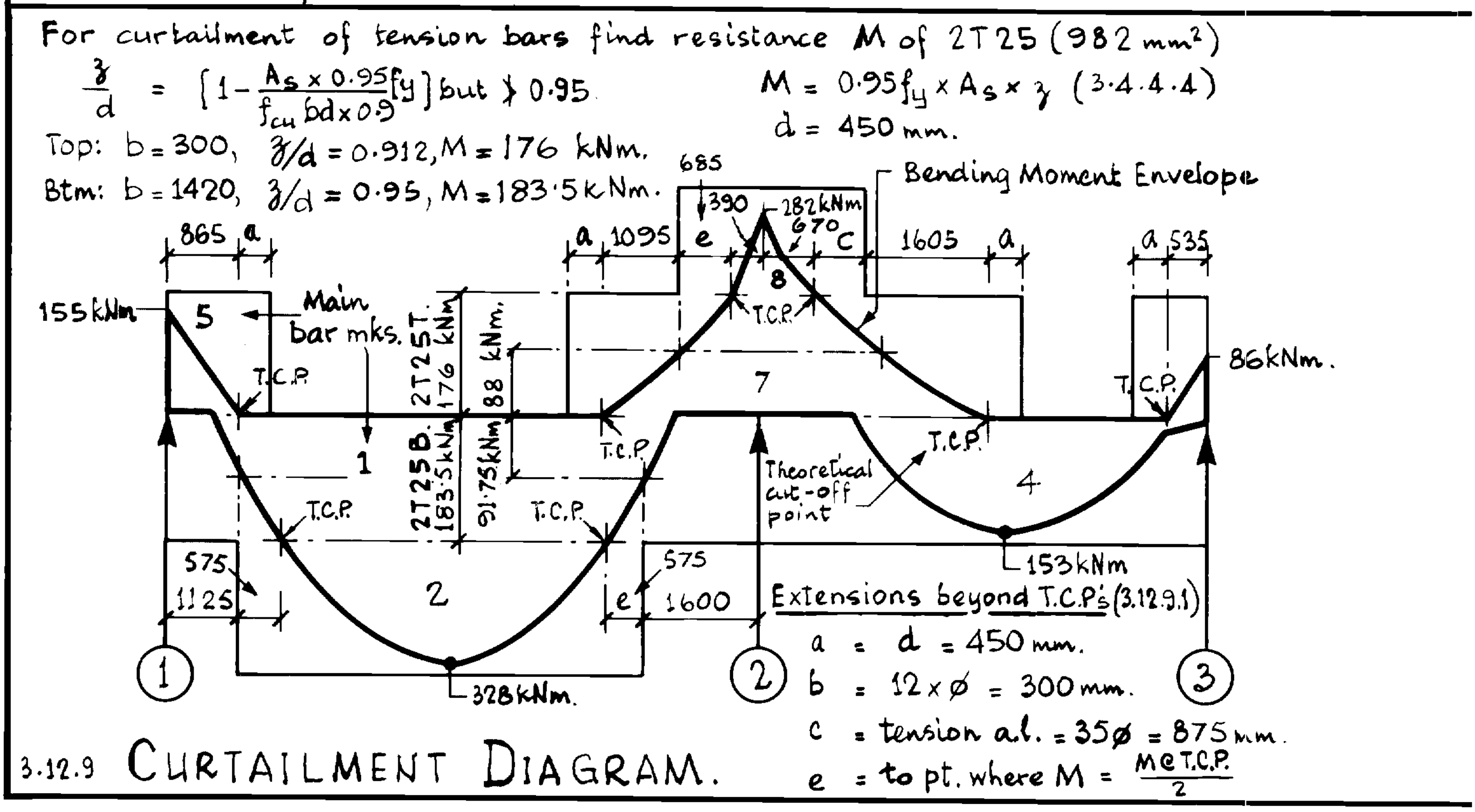
3.12.3.6.1: b. External Column (N→S) tie force.

(i) $\frac{l_s}{2.5}F_t = \frac{3.325}{2.5}\times36 = 47.9 < 2F_t = 72$ kN

(ii) 3% of total design ultimate load carried by column $= 0.03\times1340 = 40.2 < 47.9$ kN

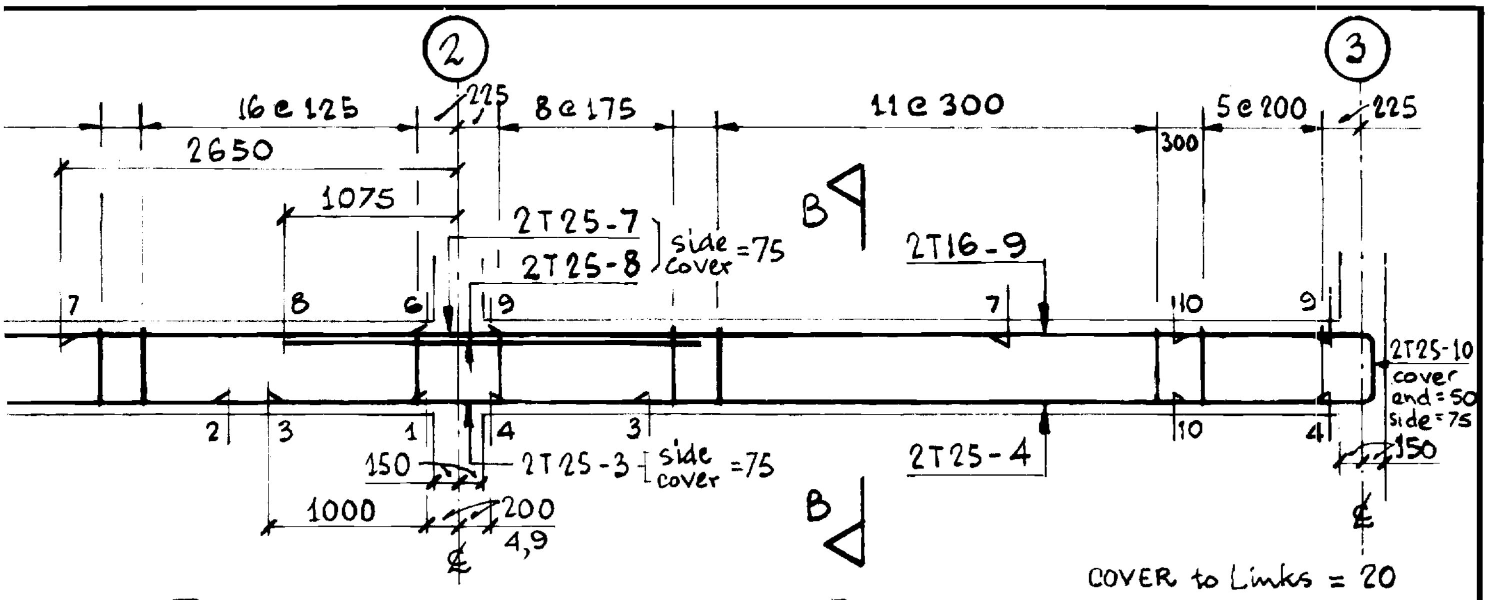
$\therefore$ Min. area of continuity reinf. $= 47.9\times\frac{10^3}{460} = \underline{104\ mm^2}$ — Output: Use U-bar anchorages at external columns.



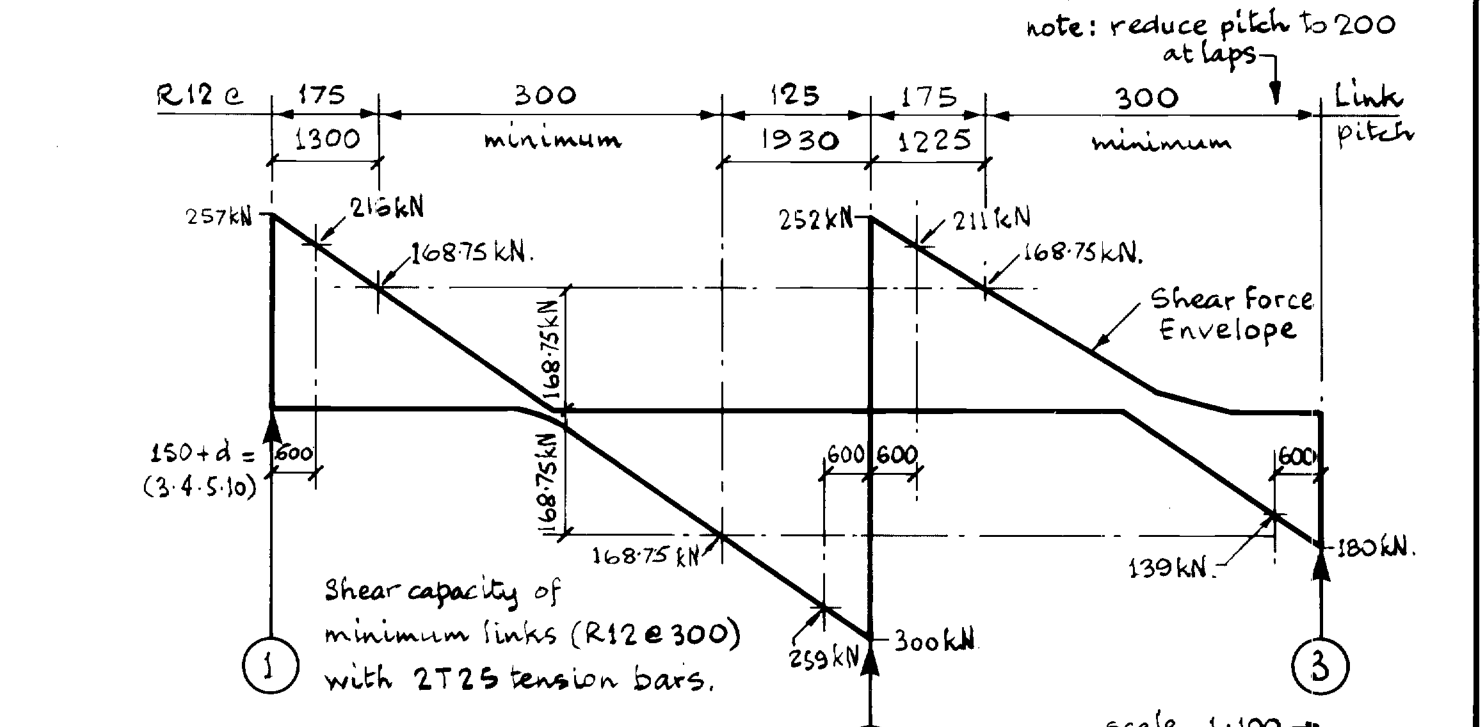


Commentary on bar arrangement

BS 8110 ref	Bar marks	Notes
		This beam shows loose splice bars at each column intersection. This method simplifies detailing and fixing and the span cage can readily be prefabricated.
— 3.3.1.2	1	Tension bars are stopped 50 mm from each column face to avoid clashing with the column bars shown in section A–A. Nominal cover = 20 + 12 = 32 mm > 25 mm, say 35 mm.
3.12.9.1 3.12.8.14	2	Remaining tension bars stopped off as shown in the curtailment diagram above. Check maximum amount of reinforcement at laps < 40% breadth. 4 × 25 = 100 mm < 0·4 × 300 = 120 mm – OK.
— 3.12.3.4	3	Loose bars are fixed inside column bars as shown in section B–B. Although designed as compression bars, these bars also act as internal ties and lap 1000 mm with the adjacent span bars for continuity.
—	4	The two tension bars are stopped 50 mm from the column face to avoid the column bars beyond.
— 3.12.11.1	5,10	Loose U-bars are fixed inside the column bars and provide continuity for column and internal ties. Check minimum distance between tension bars = 25 mm (aggregate size + 5 mm). 300 – 200 = 100 mm > 25 mm – OK.
3.12.9.1		Top legs project from centre-line into span, minimum dimensions shown in the curtailment diagram.

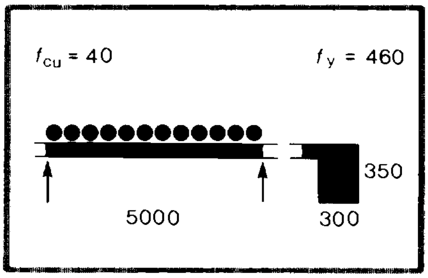


N - TYPICAL MAIN BEAM.
Scale, 1:50



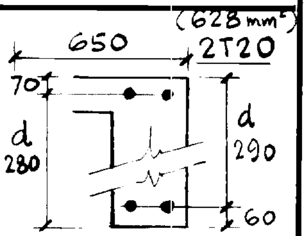
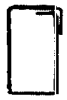
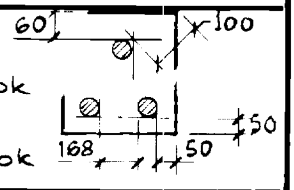
LINK DIAGRAM.

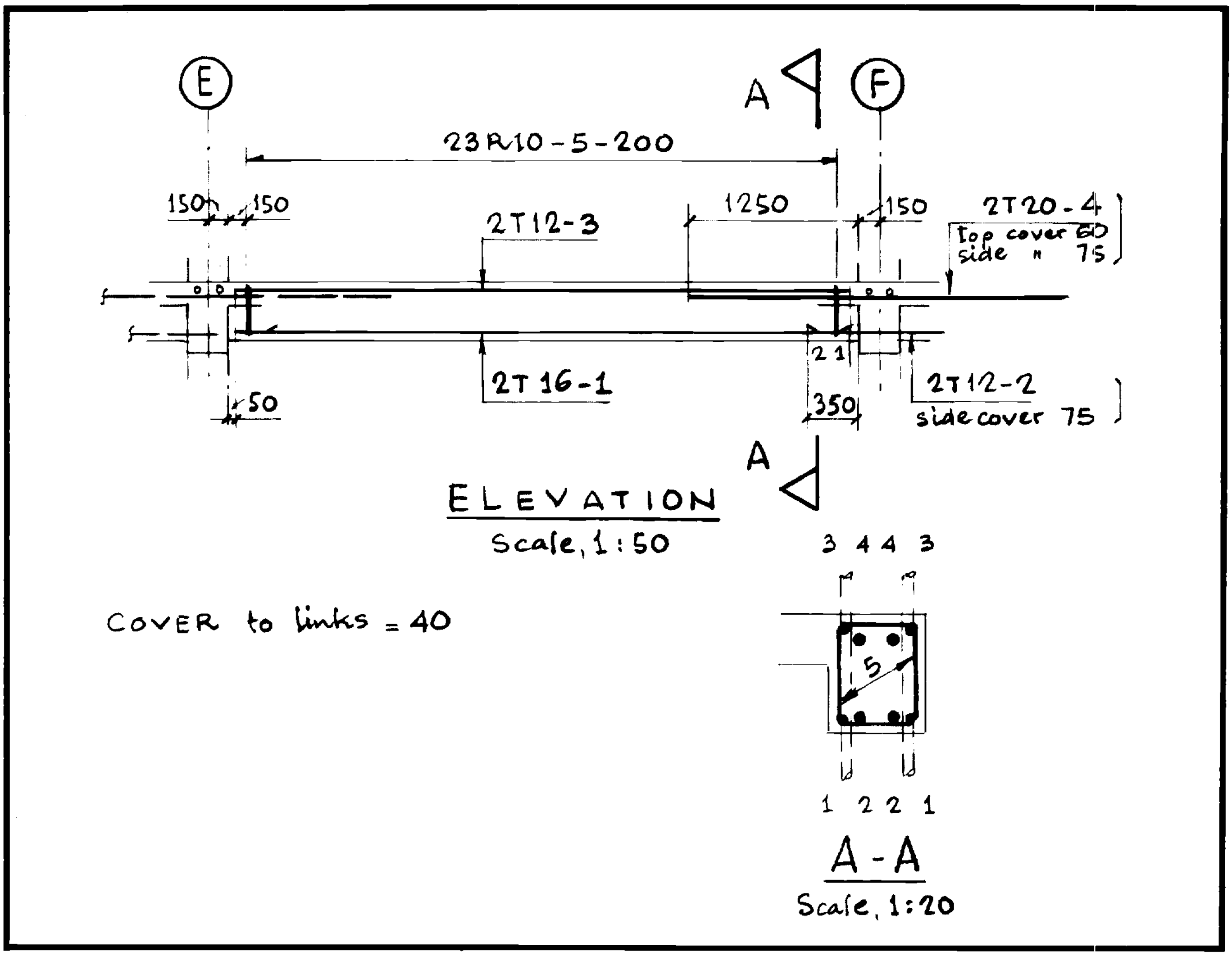
3.12.3.6		Bottom legs lap minimum 1000 mm with span bars to provide continuity for the internal tie.
—	5	Top legs = 865 + 450 = 1315 mm) let both legs Bottom legs = 200 + 1000 = 1200 mm) project 1350 mm, say.
3.12.8.14		Note that the bottom legs are raised to avoid the 40% rule in the lower layer.
3.12.8.3		Check bearing stress inside bends. Use $r = 5\phi$ for each radius to simplify bending.
—	10	Top legs = 535 + 450 = 985 mm) let both legs Bottom legs = 200 + 1000 = 1200 mm) project 1200 mm, say.
3.12.8.3		Use $r = 4\phi$ minimum radius bends.
—	6,9	Link hanger bars are same length as bar marks 1 and 4. Bar is one size larger than links (minimum 12 mm).
3.12.9.1	7,8	The tension bars over the support stop as shown in the curtailment diagram. These bars are fixed inside the column reinforcement as shown in section B–B.
3.12.4.1		These bars are bundled vertically in pairs to reduce congestion and this also allows a gap (minimum 75 mm) for insertion of a vibrator.
—	11	Closed links, shape code 61, are arranged to suit the link diagram above. Open top links, shape code 77, are not suitable for the sizes shown.
3.12.8.12		Note that links at laps are spaced at not greater than 200 mm since cover < 1·5 bar size.



Edge beam

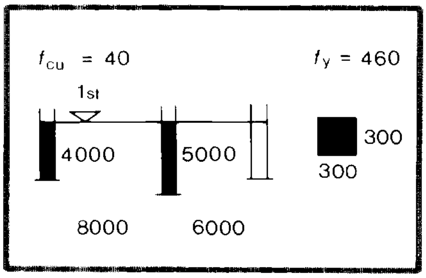
interior-span flanged beam

BS 8110 ref.	CALCULATIONS	OUTPUT
3·3 Tables 3·3, 3·4	DURABILITY and FIRE RESISTANCE Nominal cover for severe conditions of exposure = 40mm. " " " 300 wide beam for 1hr. period = 20 mm.	Minimum cover to links = 40mm
      3.4.3	LOADING Dead load from slab (1250 strip, p.10) = 23·5 × 1·25 = 29·4 Self weight (0·35 − 0·175) × 0·3 × 24 × 5 = 6·3 Cladding @ 5 kN/m (page 6) = 5 × 5 = 25·0 ∴ Characteristic dead load = 60·7 kN. " imposed " from slab (p.10) = 20 × 1·25 = 25·0 kN. Design load = $1{\cdot}4G_k + 1{\cdot}6Q_k$ = 85 + 40 = 125·0 kN.	 G_k = 60·7 kN Q_k = 25·0 kN F = 125·0 kN.
Table 3·5	ULTIMATE B.M's Interior supports: $M = 0{\cdot}08Fl$ = 0·08 × 125 × 5 = 50·0 kNm Mid-int. span: $M = 0{\cdot}07Fl$ = 0·07 × 125 × 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	REINFORCEMENT Interior supports: $\frac{M}{f_{cu}bd^2} = \frac{50\times10^6}{40\times300\times280^2} = 0{\cdot}053$ $\frac{z}{d} = (0{\cdot}5 + \sqrt{(0{\cdot}25 - \frac{0{\cdot}053}{0{\cdot}9})}) = 0{\cdot}937$, $A_s = \frac{50\times10^6}{0{\cdot}95\times460\times0{\cdot}937\times280} = 436\ mm^2$ Mid-int. span: eff. flange width = $300 + \frac{0{\cdot}7}{10}\times5000$ = 650mm. $\frac{M}{f_{cu}bd^2} = \frac{43{\cdot}8\times10^6}{40\times650\times290^2} = 0{\cdot}02$, $\frac{z}{d} = 0{\cdot}95$, $A_s = \frac{43{\cdot}8\times10^6}{0{\cdot}95\times460\times0{\cdot}95\times290} = 364\ mm^2$ Shear force = 0·55F × 125 = 68·75 kN. Eff. tension reinf for shear is 2T20. Design shear force = 68·75 − (0·15 + 0·28) 25 = 58 kN. $\frac{100A_s}{bd} = \frac{100\times628}{300\times280} = 0{\cdot}75$, $v_c = 0{\cdot}73\ N/mm^2$, $v = \frac{58\times10^3}{300\times280} = 0{\cdot}69\ N/mm^2$ $v < (v_c + 0{\cdot}4)$ ∴ use min links, $\frac{A_{sv}}{s_v} = \frac{0{\cdot}4\times300}{0{\cdot}95\times250} = 0{\cdot}51$ Max. S_v = 0·75 × 280 = 210 mm (use 200 mm) $\frac{A_{sv}}{s_v} = \frac{157}{200} = 0{\cdot}79 > 0{\cdot}51$	 2T16 (402 mm²)  R10 @ 200 throughout span.
3·4·6·1 Table 3·9 3·4·6·5 Table 3·10	DEFLECTION basic l/d = 22·0 max, (b_w/b = 0·46 > 0·3) $\frac{M}{bd^2} = \frac{43{\cdot}8\times10^6}{650\times290^2} = 0{\cdot}8$, $f_s = \frac{2\times460\times364}{3\times402} = 278\ N/mm^2$ ∴ Modification factor = 1·53 ∴ Allowable span/eff. depth ratio = 22 × 1·53 = 33·7 ∴ Actual " " " " = $\frac{5000}{290}$ = 17·2	∴ l/d ratio ok.
3.12.11.2.4 Table 3.10 3.12.11.2.4	CRACKING Bottom bars f_s = 278 N/mm² (see deflection) Allowable clear spacing = $\frac{47000}{278}$ = 169 > 168, side cover 50 < $\frac{169}{2}$ Top bars $f_s = \frac{2\times460\times436}{3\times628}$ = 213 N/mm², cover > 50, check corner Allowable clear spacing = $\frac{47000}{213}$ = 220, corner distance 100 < $\frac{220}{2}$	
Table 3·27 3·12·3·5 3·12·8·11	TIE PROVISION Peripheral tie, F_t = 36 kN. $A_s = \frac{36\times10^3}{460}$ = 78 mm² Use 2T12. Min lap = 35 × 12 × 78/226 = 145 < 300 mm	TOP or BOTTOM 2T12 min. (226 mm²)



Commentary on bar arrangement

BS 8110 ref	Bar marks	Notes
3.12.8.11		Horizontal bars in this member provide the peripheral tie. Minimum lap = 300 mm.
–	1	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 × 364 = 109 mm². Use 2T12 = 226 mm². Lap = 35 × 12 × 109/226 = 203 mm > 15 × 12 = 180 mm < 300 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 × 436 = 87 mm². Use 2T12 = 226 mm².
3.12.10.2 Figure 3.24	4	Tension reinforcement over support is fixed inside vertical column bars. Bars are curtailed at 0·25 span from face of support = 0·25 × 5000 = 1250 mm > 45 × 20 = 900 mm
–	5	Closed links are shape code 61.



Columns

slender and short columns

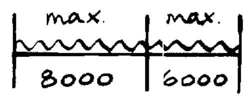
BS 8110 ref.	CALCULATIONS	OUTPUT
3·2·1·2·1	SUB-FRAME ANALYSIS - refer to main beam, page 10.	
3·3 Tables 3·3	DURABILITY and FIRE RESISTANCE Nominal cover for mild conditions of exposure = 20 mm. " " " severe " " " = 40 mm.	Minimum cover to links Internal 20 (use 30 mm) External 40 mm.
3·4	" " " 300x300 column for 1hr. period = 20 mm.	

INTERNAL COLUMN (Foundation → Roof)

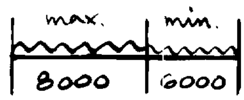
AXIAL LOADING and MOMENTS from ANALYSIS

F2 (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	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				



LOAD CASE 1



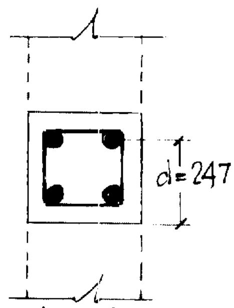
LOAD CASE 2

3·8·1·6 Eqn.30 — (Foundation → 1st. Floor)
Effective height $l_e = \beta l_0$

Table 3·19 — N → S
β = 0·9 (End condition: top 1, bottom 3)
l_{ex} = 0·9 × 4·5 = 4·05m
$\frac{l_{ex}}{h} = \frac{4.05}{0.3}$ = 13·5

E→W
β = 0·95 (End condition: top 2, bottom 3)
l_{ey} = 0·95 × 4·825 = 4·58 m

3·8·1·3 — $\frac{l_{ey}}{b} = \frac{4.58}{0.3}$ = 15·26 > 15 ∴ SLENDER COLUMN

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

BS 8110 ref.	CALCULATIONS	OUTPUT

EXTERNAL COLUMN (Foundation → Roof)

F1 (typical)

AXIAL LOADING and MOMENT from ANALYSIS

		BEAM LOADS kN. TOTAL		COLUMN DESIGN LOADS kN. IMPOSED		DEAD		COL. MOMENTS kNm. 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 (Foundation → 1^{st} floor)

Eqn. 30 Effective height $l_e = \beta l_o$

Table 3·19 N → S $\beta = 0{\cdot}9$ (end condition: top 1, bottom 3)

$l_{ex} = 0{\cdot}9 \times 3{\cdot}5 = 3{\cdot}15\text{m}$

$$\frac{l_{ex}}{h} = \frac{3{\cdot}15}{0{\cdot}3} = 10{\cdot}5$$

E → W $\beta = 0{\cdot}9$ (end condition: top 1, bottom 3)

$l_{ey} = 0{\cdot}9 \times 3{\cdot}65 = 3{\cdot}285$

3·8·1·3 $$\frac{l_{ey}}{b} = \frac{3{\cdot}285}{0{\cdot}3} = 10{\cdot}95 < 15$$

∴ SHORT COLUMN.

Design using load case 2

BS6399 Part 1 Just above 1^{st} floor, $N = 800 + 43 + 0{\cdot}9 \times 240 = 1059$ kN.

$M = 117$ kNm.

$$\frac{N}{bh} = \frac{1059 \times 10^3}{300^2} = 11{\cdot}8, \quad \frac{M}{bh^2} = \frac{117 \times 10^6}{300^3} = 4{\cdot}3$$

Assume $d = 300 - 50 - 13 = 237\text{mm}$, $\frac{d}{h} = \frac{237}{300} = 0{\cdot}79$

use $\frac{d}{h} = 0{\cdot}8$ (chart page 28) BS8110 Part 3 1985

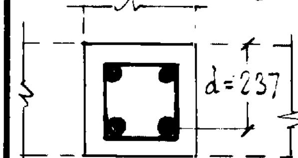
$$\frac{100A_{sc}}{bh} = 2{\cdot}3, \quad A_{sc} = 2070\,\text{mm}^2 \quad (4T25 = 1960\,\text{mm}^2)$$

BS6399 Part 1 Below 1^{st} fl. → fdns, $N = 1059 + 43 + 0{\cdot}8 \times 359 = 1389$ kN.

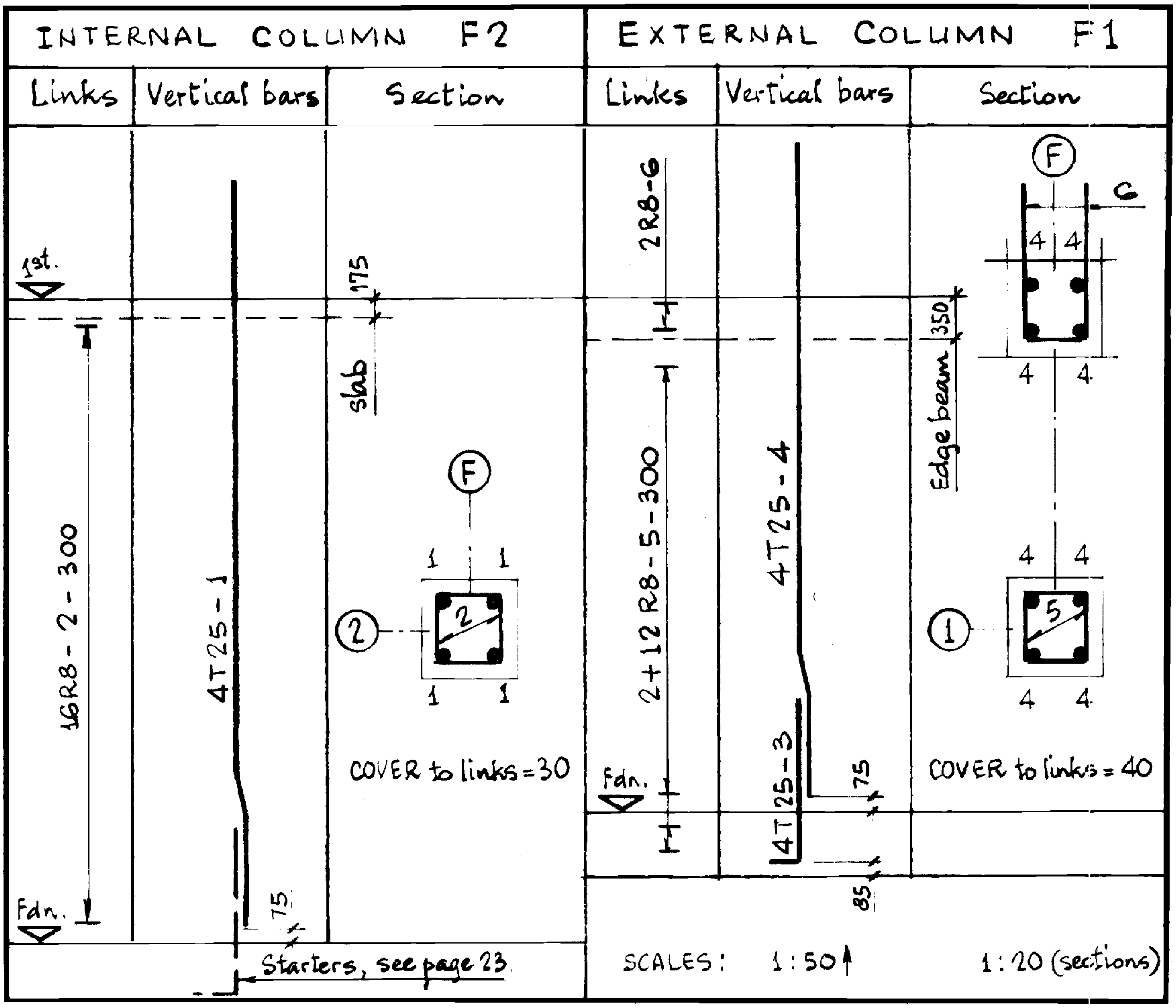
$M = 76$ kNm.

$$\frac{N}{bh} = 15{\cdot}4, \quad \frac{M}{bh^2} = 2{\cdot}81, \quad \frac{100A_{sc}}{bh} = 1{\cdot}5$$

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

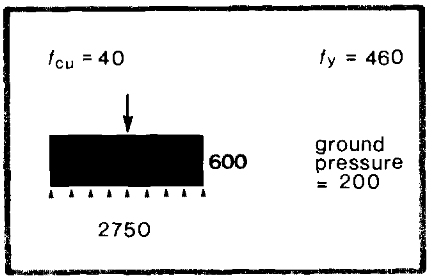


4T25 ($1960\,\text{mm}^2$)



Commentary on bar arrangement

BS 8110 ref	Bar marks	Notes
		The presentation shown above is schematic. This tabular method adapts readily to element repetition. The sections are shown in their relative positions adjacent to the vertical reinforcement.
3.12.5.3	1	Main bars, area > minimum 0·4% *bh*. Slope of crank at lower end = 1:10 maximum. Crank offset = 50 + 10% = 55 mm. Minimum crank length = 350 mm (14⌀). Length of short projection beyond crank = compression lap +, say, 75 mm for tolerance.
3.12.6.2		Reinforcement area at laps < 10% *bh*.
3.12.8.15		Bars project above first-floor slab level to provide a compression lap above the kicker.
Table 3.27		Bar projection = 35 × 0·87/0·95 × 25 mm + 75 mm for kicker = 875 mm, i.e. compression lap = 800 mm.
3.12.7.2	2	A single link is provided, since each vertical bar is restrained by a corner.
3.12.7.1		Minimum size = 25/4, use 8 mm. Maximum spacing = 12 × 25 = 300 mm. (R8 @ 300.)
3.12.8.12		Cover to vertical bar = 40 mm > 1·5 × 25 = 37·5 mm. Links extend to underside of floor slab.
–	3	Normally, starter bars are detailed with the footing, as column F2. It can be economic to detail starters with the column above as shown. In this case it is advisable to schedule the starter bars so that they can be processed together with the footing. Note with this detail that the section at mid-height also applies to the starter bar arrangement. The starter bars would be shown dotted on the footing detail together with a suitable cross-reference. Bars project above the top of the base to provide a compression lap above the kicker
Table 3.27		= 35 × 0·87/0·95 × 25 + 75 = 875 mm, i.e. lap = 800 mm.
–	4	As bar mark 1, but bars provide a tension lap above 1st floor kicker. Cover = 50 mm.
3.12.8.13b		Clear distance between adjacent laps = 100 mm < 6 × 25 mm; i.e. use factor 1.4
Table 3.27		Projection = 1·4 × 35 × 0·87/0·95 × 25 + 75 = 1195 mm, say 1200 mm, i.e. tension lap = 1125 mm.
3.12.8.14		Sum of bar sizes at tension lap = 4 × 25 = 100 mm. 100/300 × 100 = 33% < 40% – OK. This detail provides the maximum lever arm and is the preferred detail for column/beam intersections.
–	5	Similar to mark 2 links, but extending to underside of main beam. Cover to vertical bars = 50 mm.
–	6	These U-bars are provided to restrain the vertical bars in the external face of the column.

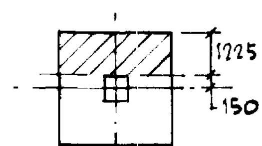
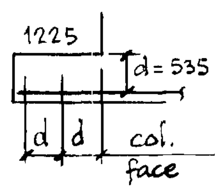
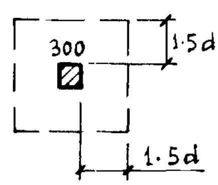


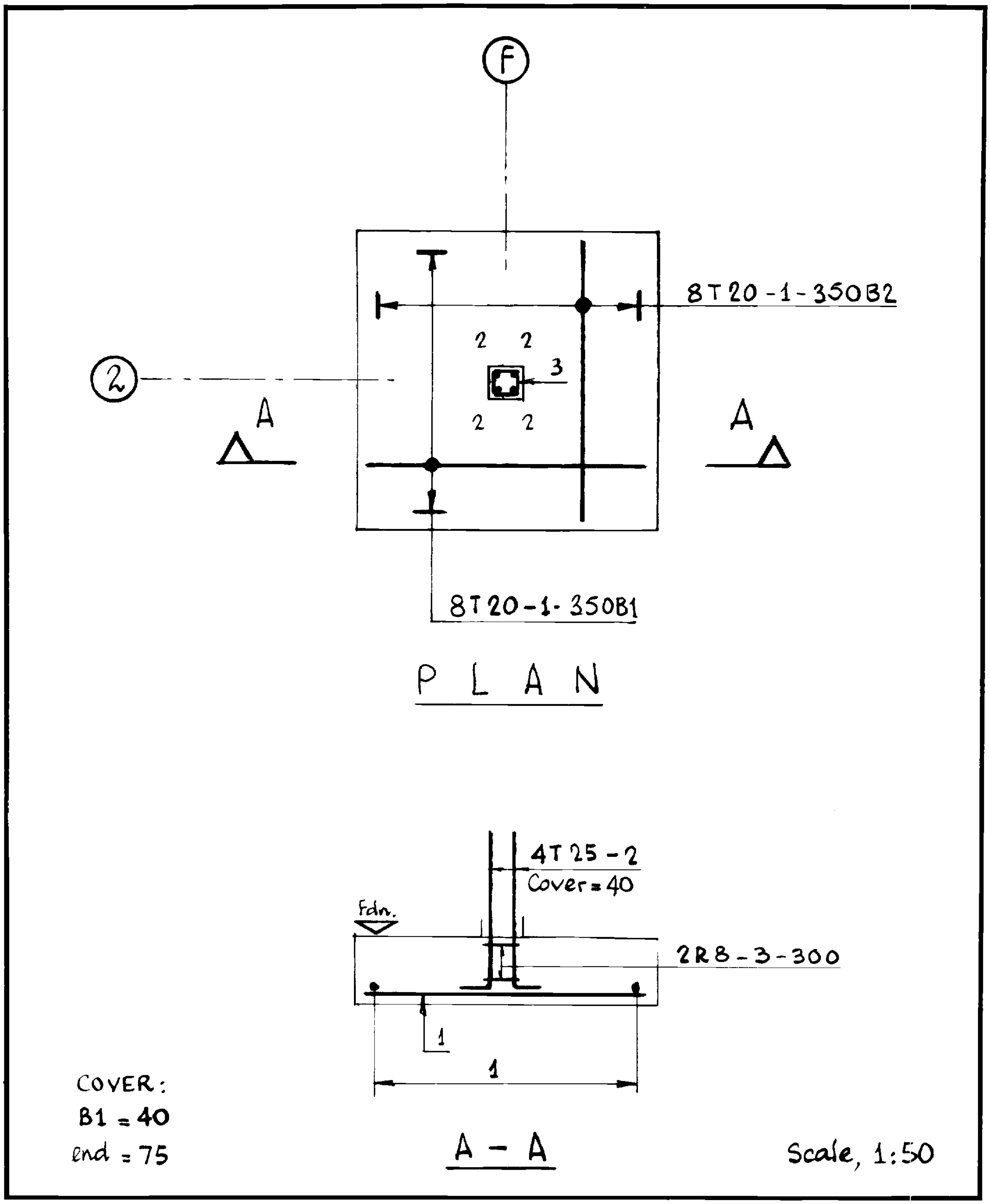
Foundation

reinforced pad footing

BS 8110 ref.	CALCULATIONS	OUTPUT
Table 3.3 3.3.1.4	DURABILITY. Min. nominal cover, moderate exposure = 30 mm. Use nominal 40mm. cover against blinding.	Nominal cover 40mm. btm, 75mm. ends

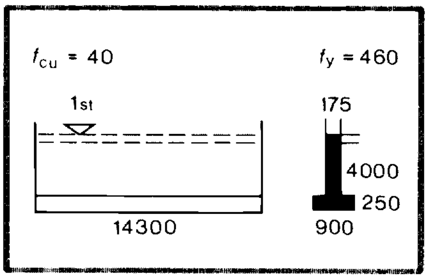
LOADING – F2 (see page 19)	Dead	Imposed	Tot. kN.
Internal Column – 'Design'	1273	718	1991
" " – 'Service'	1273/1.4 = 909	718/1.6 = 449	1358

BS 8110 ref.	CALCULATIONS	OUTPUT
	Allow 10kN/m² extra over soil displaced by concrete and for gnd. fl. loading. ∴ Pad area required = $1358/(200-10)$ = 7.15 m² ∴ Adopt 2.75 × 2.75 × 0.6 thick ftg, area provided = 7.5 > 7.15 m² then U.L.S. 'Design' pressure = $1991/2.75^2$ = 263 kN/m²	
  3.4.4.4	REINFORCEMENT Moment at face of column = $263 \times 2.75 \times \frac{1.225^2}{2}$ = 543 kNm Average d = 600 − 40 − 25 = 535 mm. $\frac{M}{f_{cu}bd^2} = \frac{543\times10^6}{40\times2750\times535^2}$ = 0.017 $\frac{z}{d} = 0.95$, $A_s = \frac{543\times10^6}{0.95\times460\times0.95\times535}$ = 2445 mm²	d = 535 mm.  8T20@350 B. EW. (2512 mm²)
3.4.5.4 3.11.3.4(1) 3.4.5.8 3.11.3.4(2) 3.7.7.2	ULTIMATE SHEAR Minimum $v_c = 0.34(40/25)^{1/3} = 0.4$ N/mm² Condition 1: Shear force V @'d' from col. face = $\frac{1991}{2}\times\left(\frac{1375-150-535}{1375}\right)$ = 500 kN " stress v " " " " = $\frac{500\times10^3}{2750\times535}$ = 0.34 N/mm² < $2v_c$ Shear force V @'2d' from col. face = $\frac{1991}{2}\times\left(\frac{1375-150-2\times535}{1375}\right)$ = 112 kN " stress v " " " " = $\frac{112\times10^3}{2750\times535}$ = 0.08 N/mm² < v_c Condition 2: (punching shear, usually the more critical) $v_{max} = \frac{V}{u_o d} = \frac{1991\times10^3}{4\times300\times535}$ = 3.1 < 5.0 N/mm² Critical perimeter = 4(300 + 3 × 535) = 7620 mm. Area within " " = (0.3 + 3 × 0.535)² = 3.63 m² Shear on " " = 263(2.75² − 3.63) = 1034 kN. v = $\frac{1034\times10^3}{7620\times535}$ = 0.25 N/mm² < v_c	 
3.12.11.2.7 3.11.3.2	CRACKING clear distance = 330 < 750 mm ok $\frac{100A_s}{bd} = \frac{100\times2445}{2750\times535}$ = 0.17 < 0.3% $l_c = 0.5\times2750 = 1375 < 0.25(3c+9d) = 0.25(3\times300+9\times535) = 1429$ ∴ Space bars uniformly across section, say @ 350 crs. each way.	∴ crack widths ok.



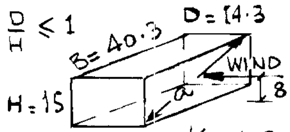
Commentary on bar arrangement

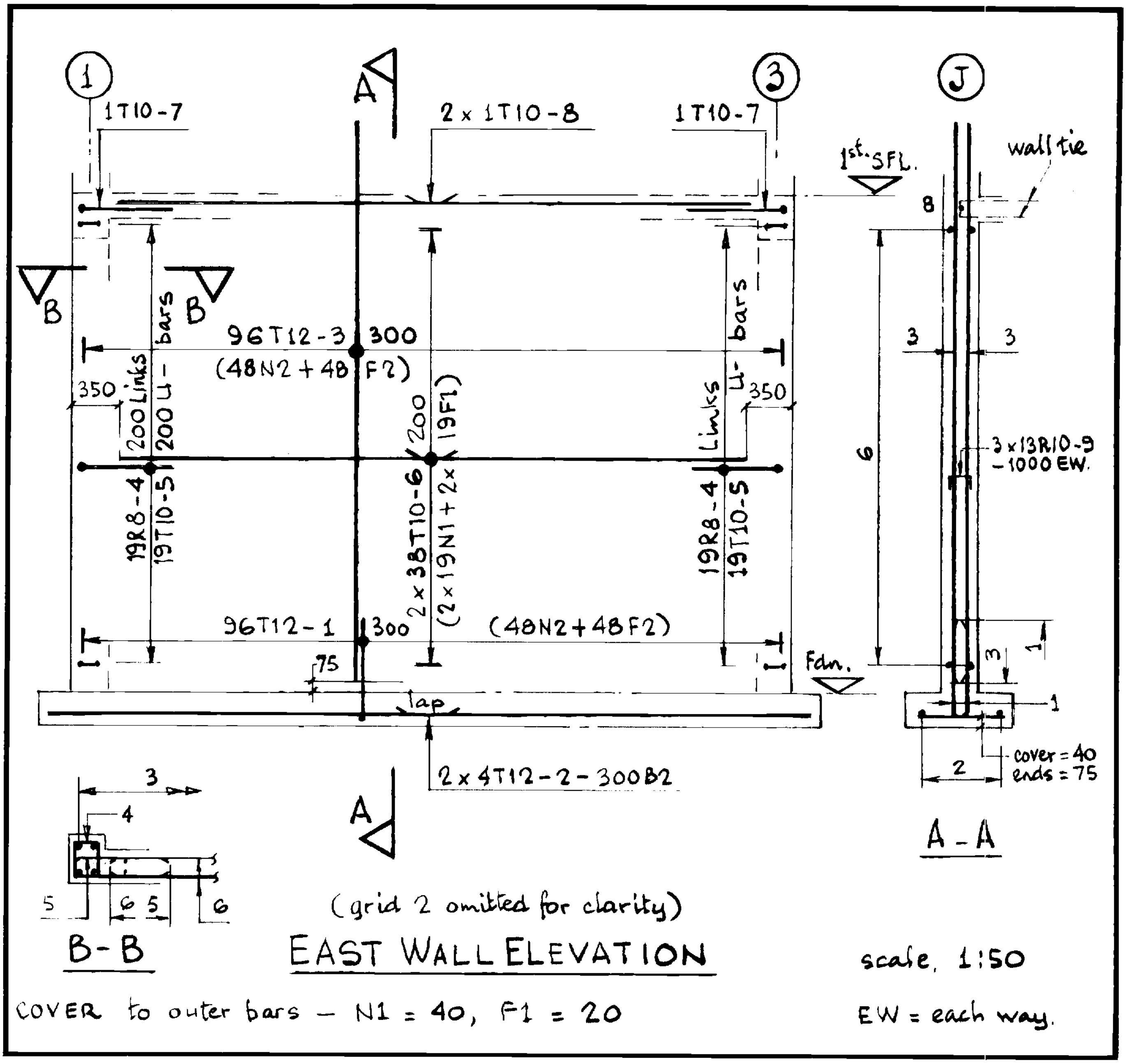
BS 8110 ref	Bar marks	Notes
3.12.8.1	1	Straight bars extend full width of base, less end covers.
Table 3.27		Bars should project a minimum tension bond length beyond the column face = 35 × 20 = 700 mm < 1150 mm – OK.
3.3.1.4		The underside of base is concrete blinded, cover = 40 mm.
– **Table 3.27**	2	Column starter bars are wired to bottom mat. Minimum projection above the top of base is a compression lap + kicker = 35 × 0·87/0·95 × 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

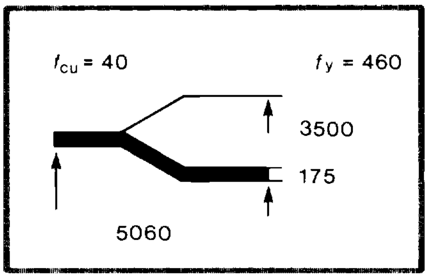
external plain concrete wall

BS 8110 ref.	CALCULATIONS	OUTPUT
3.9.4.3 1.2.4	Design as braced plain concrete wall Eff. ht., fdn. → 1st fl. $l_e = 0.875 \times 3825 = 3350$ mm. $\frac{l_e}{h} = \frac{3350}{175} = 19.2 > 12$	175 mm thick ∴ SLENDER WALL
Table 3.3 fig 3.2	DURABILITY and FIRE RESISTANCE Nominal cover: (a) severe exposure = 40 mm. (b) mild = 20 mm. Fire resistance of 175 plain wall = 1½ > 1 hour	external cover = 40 mm internal " = 20 mm ∴ 1hr. fire resistance ok.
	LOADING @ top of foundations Dead load from 1st, 2nd, 3rd fl. + roof = $0.5(3\times23.5+28.5)$ = 49.5 kN/m. self weight = $0.175\times24\times15.5$ = 65.1 Characteristic dead load 114.6 kN/m " imposed " (slabs) = $2.5(1.5+3\times4\times0.8)$ = 27.8 kN/m.	∴ $g_k = 114.6$ kN/m. ∴ $q_k = 27.8$ kN/m
	WIND LOADING $V_s = V_b S_a = 22$ m/sec, $V_e = V_s S_b = 37.7$ m/sec Dynamic pressure $q_s = 0.613 V_e^2 = 0.613\times37.7^2\times10^{-3} = 0.87$ kN/m² Total wind load = $0.85\,C_a\,(C_{pe.front} - C_{pe.rear})\,q_s\,(1+C_r)\,A$ $= 0.85\times0.84\times(0.8+0.3)\times0.87\times1.025\times40.3\times15 = 423.4$ kN M end wall = $0.5\times423.4\times8.0$ = 1694 kNm, $f = \pm\frac{1694\times6}{14.3^2} = \pm49.7$ kN/m	BS 6399 Part 2 $\frac{D}{H} \le 1$ D = 14.3  a = 43 m $K_b = 1.0$ $C_a = 0.84$ (fig 4)
2.4.3.1 Table 2.1 3.9.4.16 eqn 43 eqn 44	VERTICAL LOADING INTENSITY (U.L.S. Design Loads) Load combination 1. $f_1 = 1.4\times114.6 + 1.6\times27.8$ = 204.9 kN/m. " " 2. $f_2 = 1.4\times114.6 + 1.4\times49.7$ = 230 kN/m $f_2 = 1.0\times114.6 - 1.4\times49.7$ = 45 kN/m " " 3. $f_3 = 1.2\times(114.6+27.8+49.7)$ = 230.5 kN/m $e_x = h/20$, $e_a = l_e^2/2500h = (19.2^2/2500)h$ = 0.15h $n_w \le 0.3(h - \frac{2h}{20})f_{cu} \le 0.27h.f_{cu}$ ≤ 1890 kN/m $n_w \le 0.3(h - \frac{1.2h}{20} - 2\times0.15h)f_{cu} \le 0.192h f_{cu}$ ≤ 1344 kN/m Actual U.L.S Design Loads < n_w in both cases	 (min) (max) ∴ ok
3.9.4.18	SHEAR Horizontal design shear force $V = 1.4\times\frac{423.4}{2} = 296.4$ kN Min. vertical " load $N = 1.0\times114.6\times14.3 = 1639$ kN. $\frac{V}{N} = \frac{296.4}{1639} = 0.18 < 0.25$, $\frac{V}{hl} = \frac{296.4\times10^3}{175\times14300} = 0.12 < 0.45$ N/mm²	∴ ok.
 3.12.3.5 3.12.3.4.2 3.12.3.4.1 3.12.3.6.1	TIE PROVISION @ 1st. Floor F_t = 36 kN Peripheral tie: $A_s = \frac{36\times10^3}{460}$ = 78 mm² Internal tie force: $\frac{2.5\times36(4.7+4.0)}{7.5}\times\frac{14.3}{5}$ = 299 kN, $A_s = \frac{299\times10^3}{460}$ = 650 mm² Horizontal reinforcement in wall 0.5m above and below slab adequate Wall tie: $\frac{l_s}{2.5} = \frac{3.325\times36}{2.5} < 2F_t$ but > 0.03×204.9 kN, $A_s = \frac{47.9\times10^3}{460}$ = 104 mm²	(78.5 mm²) 1T10 T10 @ 200 EF. HORIZ. (393 mm²/m) use adjacent slab reinf.
    Table 3.25	FOOTING 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/m² < 1.25×190 $M = 230.5\times\frac{0.725^2}{8}$ = 15.1 kNm, $A_s = \frac{15.1\times10^6}{0.95\times460\times0.95\times200}$ = 182 mm² If 250 thick, d = 200 Min. area = $0.13\times10^3\times250$ = 325 mm²	BS 8004 2.3.2.4 Bottom T12 @ 300 EW (377 mm²/m)



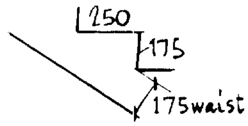
Commentary on bar arrangement

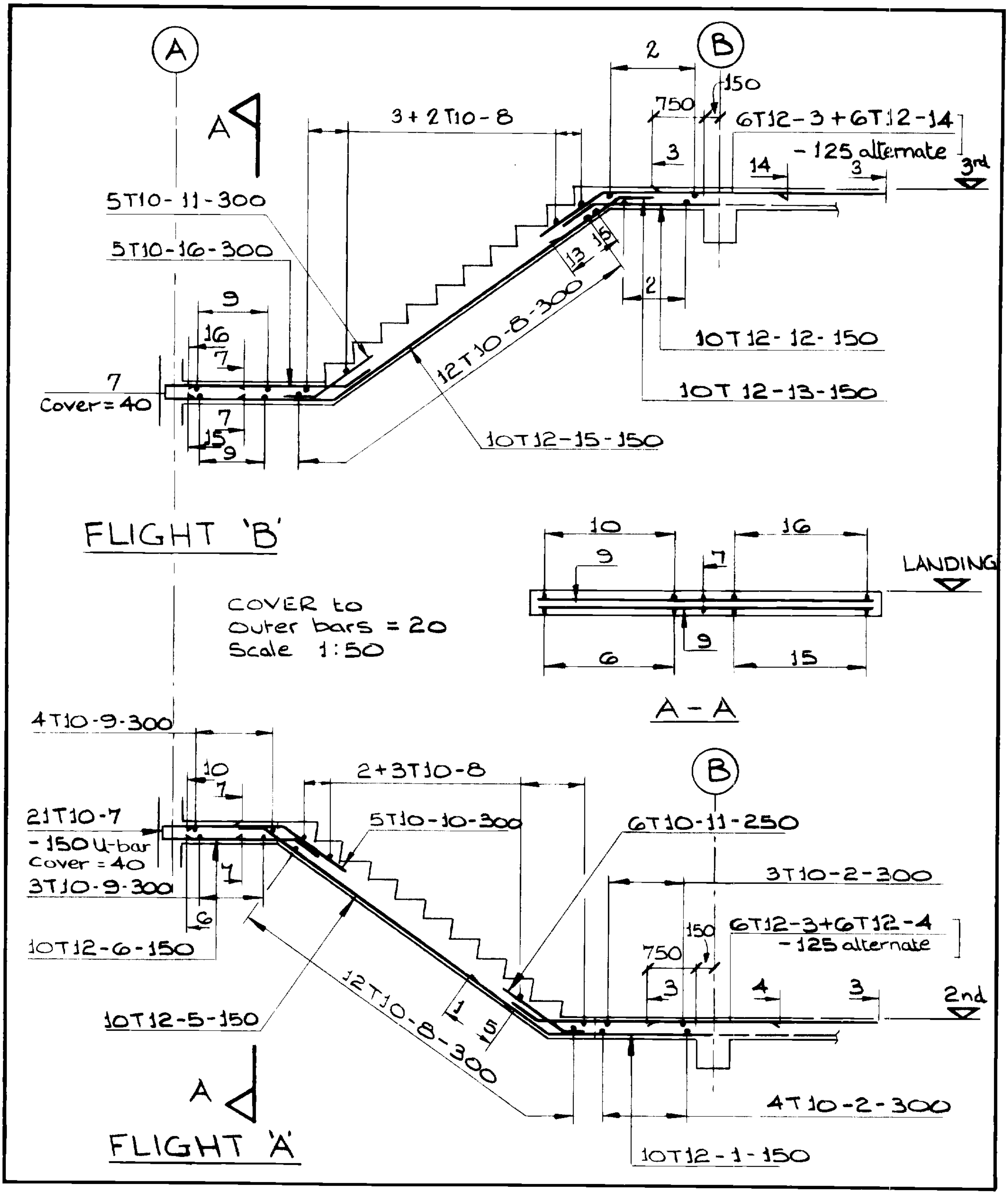
BS 8110 ref	Bar marks	Notes
– Table 3.27 3.3.1.4	1	Wall starters match vertical reinforcement. Minimum projection of horizontal legs beyond the wall face is a design tension bond length = 35 × 182/377 × 12 = 203 mm < 287 mm. This provides the footing reinforcement. Minimum projection above top of base is a compression lap + kicker = 35 × 12 + 75 = 495 mm, say 525 mm, i.e. lap = 450 mm. Underside of footing is concrete blinded, cover = 40 mm.
Table 3.25	2	Minimum longitudal reinforcement provided.
3.9.4.19	3	Minimum vertical reinforcement. Area = 0·25% × 1000 × 175 = 438 mm²/m. (T12 @ 300 EF = 754 mm²/m.) T12 bars provide reasonable rigidity for handling and help stabilize the cage during erection. Minimum projection above top of first-floor level is a compression lap + kicker = say 525 mm. Lap = 450 mm.
– Table 3.27	4,5,6	Minimum horizontal reinforcement. Area = 438 mm²/m. (T10 @ 200 EF = 786 mm²/m.) Provide at least a tension lap = 35 × 10 = 350 mm, say 450 mm to satisfy shrinkage and thermal requirements. Bars are placed outside vertical reinforcement to provide maximum control against shrinkage and thermal cracking. Those bars in the wall 0·5 m below first-floor slab act also as internal ties. Tension lap for tie = 35 × 10 = 350 mm, say 450 mm.
3.12.3.4	7,8	Peripheral tie at first floor. L-bars at either end provide continuity with edge beams. Laps, say 450 mm.
–	9	Wall spacers maintain location of each face of reinforcement.



Staircase

end-span continuous slab

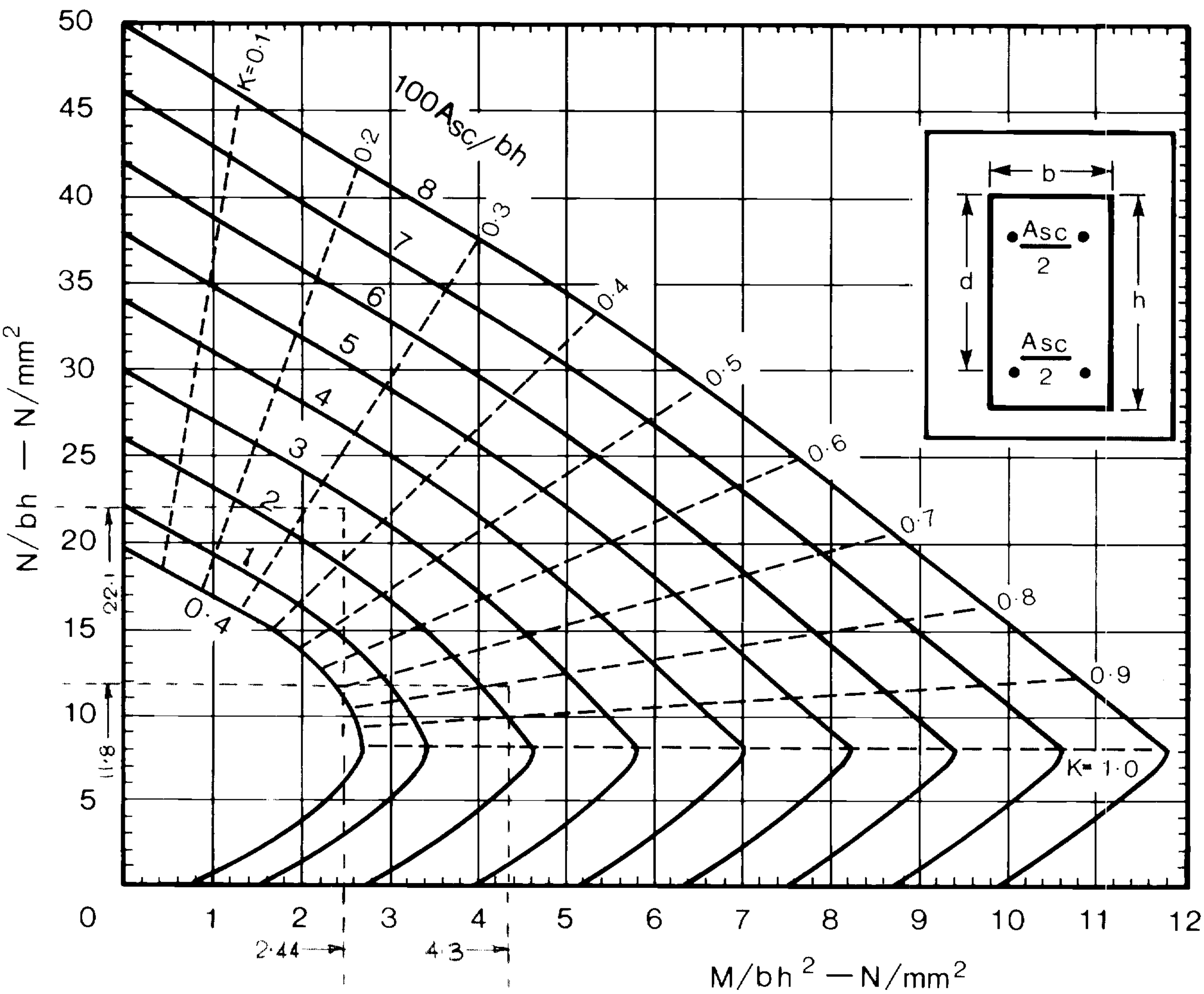
BS 8110 ref.	CALCULATIONS	OUTPUT
3·3 Tables 3·3, 3·4	DURABILITY and FIRE RESISTANCE (as floor slab, page 8)	cover = 20mm 1hr. fire resistance
3.10.1.1 3.10.1.2	LOADING Average slab thickness on plan = 250 mm Self-weight 0·25 × 24 = 6·0 Finishes = 0·5 ∴ Characteristic dead load = 6·5 kN/m² " imposed " (page 6) = 4·0 kN/m² Design load = (1·4×6·5 + 1·6×4·0)5·0 = 77·5 kN/m width	g_k = 6.5 kN/m² q_k = 4.0 kN/m² F = 77.5 kN/m width
3.4.3 Table 3·5	ULTIMATE B.M's 1st. interior support = 0·11 Fl = 0·11 × 77·5 × 5·06 = 43·1 kN/m Near mid-end span = 0·09Fl = 35·3 kN/m	
3.4.4.4 Table 3·8	REINFORCEMENT 1st. interior support, $\frac{M}{f_{cu}bd^2} = 0{\cdot}049$, $\frac{z}{d} = 0{\cdot}94$ $A_s = \frac{M}{0{\cdot}95 f_y z} = \frac{43{\cdot}1 \times 10^6}{0{\cdot}95 \times 460 \times 0{\cdot}94 \times 149} = 704\ mm^2/m$ Near mid-end span, $\frac{M}{f_{cu}bd^2} = 0{\cdot}040$, $\frac{z}{d} = 0{\cdot}95$ $A_s = \frac{M}{0{\cdot}95 f_y z} = \frac{35{\cdot}3 \times 10^6}{0{\cdot}95 \times 460 \times 0{\cdot}95 \times 149} = 571\ mm^2/m$ Check for Shear: $v = \frac{0{\cdot}6 \times 77{\cdot}5 \times 10^3}{10^3 \times 149} = 0{\cdot}31\ N/mm^2 < v_c$	d = 149 mm TOP T12 @ 125 (905 mm²/m) BOTTOM T12 @ 150 (754 mm²/m) ∴ ok.
3·4·6 Table 3·10	DEFLECTION basic span/eff. depth ratio = 26 max. $\frac{M}{bd^2} = \frac{35{\cdot}3 \times 10^6}{10^3 \times 149^2} = 1{\cdot}59$, $f_s = \frac{2 \times 460 \times 571}{3 \times 754} = 232{\cdot}2\ N/mm^2$ Modification factor for service stress 232·2 N/mm² = 1·37 ∴ Allowable span/eff. depth ratio = 26 × 1·37 = 35·62 Actual " " " = $\frac{5060}{149}$ = 34·0	∴ l/d ratio ok.
3·12·11·2·7	CRACKING Spacing between bars < 3×149 = 447 mm. h = 175 < 200mm. ∴ no further checks required	Spacing < 3d. ok. ok.
3·12·3·4	TIE PROVISION East → West Internal Tie Min. area continuity reinf. = 91 mm²/m (as floor slab, p.8) ∴ Total area req'd for width of staircase = 91 × 3 = 273 mm² Provide 2T12 tie bars each side in adjacent slab. LAP, as floor slab, p8.	BOTTOM 4T12 (456 mm²) min. 300mm. 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 × 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 × 1000 × 175 = 228 mm²/m. Use T10 @ 300 = 262 mm²/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 × 571 = 286 mm²/m. Use T10 @ 150 = 524 mm²/m to match spacing of span bars. Laps, say 450 mm.
Table 3.25	10,11	Optional reinforcement. Minimum area = 228 mm². Similar for bar mark 16.

Column design chart



Rectangular columns

f_{cu}	40
f_y	460
d/h	0·80

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.

Designed and detailed (BS 8110 : 1997)

J. B. Higgins and B. R. Rogers

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BETTER BUILT IN concrete