Designed and detailed
(BS 8110: 1997)

J. B. Higgins and B. R. Rogers MA, CEng, MICE
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(BS 8110: 1997)

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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.


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.

<table>
<thead>
<tr>
<th>Ultimate limit state</th>
<th>Serviceability limit states</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Deflection</td>
</tr>
<tr>
<td>Objective</td>
<td>Provision of adequate safety</td>
</tr>
<tr>
<td>Loading regime</td>
<td>Design ultimate loads</td>
</tr>
<tr>
<td>Performance limit</td>
<td>Structure should not fail</td>
</tr>
</tbody>
</table>

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:

\[
\text{Characteristic strength} = \text{Mean or Average strength} - 1.64 \times \text{Standard deviation}
\]

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:

\[ \text{Design load} = \text{Characteristic load} \times \gamma_l \]

where \( \gamma_l \) 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 \( \gamma_l \) 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 \( \gamma_l \) 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.

<table>
<thead>
<tr>
<th>Combination of loads</th>
<th>Partial safety factor to be applied to</th>
<th>Partial safety factor to be applied to</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>dead load</td>
<td>imposed load</td>
</tr>
<tr>
<td></td>
<td>adverse</td>
<td>beneficial</td>
</tr>
<tr>
<td>1 Dead and imposed</td>
<td>1.4</td>
<td>1.0</td>
</tr>
<tr>
<td>2 Dead and wind</td>
<td>1.4</td>
<td>1.0</td>
</tr>
<tr>
<td>3 Dead and wind</td>
<td>1.2</td>
<td>1.2</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Values of ( \gamma_m ) for the ultimate limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement</td>
</tr>
<tr>
<td>Concrete</td>
</tr>
<tr>
<td>Flexure or axial load</td>
</tr>
<tr>
<td>Shear strength without shear reinforcement</td>
</tr>
<tr>
<td>Bond strength</td>
</tr>
<tr>
<td>Others (e.g. bearing stress)</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Client</th>
<th>National Company plc</th>
<th>Engineer responsible</th>
<th>Building Regulation authority or other and Date of submission</th>
<th>Relevant Building Regulations and Design Codes</th>
<th>Intended use of structure</th>
<th>Fire resistance requirements</th>
<th>General loading conditions</th>
<th>Wind loading conditions</th>
<th>Exposure conditions</th>
<th>Subsoil conditions</th>
<th>Foundation type</th>
<th>Material data</th>
<th>Other relevant information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Architect</td>
<td>London</td>
<td>9/11/98</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. The Building Regulations 1991  
2. BS 8110 The Structural Use of Concrete, Part 1 1997  

**Laboratory and Office Block**

1 hour for all elements

- Roof - imposed = 1.5 kN/m²
- Roof - finishes = 1.2 kN/m²
- Floors - imposed (3.0) and partitions (1.0) = 4.0 kN/m²
- Stairs - imposed = 4.0 kN/m²
- Self weight of concrete = 24.0 kN/m³

Speed of wind

21 m/sec (basic)  

**Factors**

\[ S_a = 1.05 \quad S_b = 1.71 \quad S_d = 1.0 \quad S_e = 1.0 \quad S_p = 1.0 \quad C_a = 0.84 \quad C_{pe} = 0.8 \text{ (frnt)} \quad C_{re} = 0.3 \text{ (rear)} \quad C_f = 0.025 \]

Severe (external) and Mild (internal) (BS 8110 Table 3.2)

- Stiff clay - no sulphates  
- Allowable bearing pressure = 200 kN/m²

RC footings to columns and walls

| Characteristic strength - main bars, \( f_y \) | 460 N/mm² | 250 N/mm² |

Other relevant information

All dimensions shown on drawings are in millimetres (mm)
3.12.3 TIE PROVISION - horizontal ties

3.12.3.4.2 \( F_t = (20 + 4n_o) = 20 + 16 = 36 < 60 \text{ kN} \)
### Floor slab

**interior-span solid slab**

<table>
<thead>
<tr>
<th>BS 8110 ref.</th>
<th>CALCULATIONS</th>
<th>OUTPUT</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.3.3 Table 3.3</td>
<td><strong>DURABILITY and FIRE RESISTANCE</strong>&lt;br&gt;Nominal cover for mild conditions of exposure = 20 mm.&lt;br&gt;Max. fire resistance of 175 slab with 20 cover = 1½ hr &gt; 1 hr.</td>
<td>Cover = 20 mm. (1½ hr fire resistance ok)</td>
</tr>
<tr>
<td>3.5.2.4 Table 3.12</td>
<td><strong>LOADING</strong>&lt;br&gt;Self-weight: $0.175 \times 24 = 4.2$&lt;br&gt;Finishes (page G): $0.5$&lt;br&gt;Characteristic dead load: $4.7 \text{kN/m}^2$&lt;br&gt;Imposed load (page G): $4.0 \text{kN/m}^2$&lt;br&gt;Design load = $(1.4 \times 4.7 + 1.6 \times 4) \times 5.0 = 64.9 \text{kN/m}$&lt;br&gt;Ultimate B.M's: $0.063 \times 0.9 \times 5.0 = 20.4 \text{kN/m}$&lt;br&gt;and supports&lt;br&gt;Reinforcement: Interior mid-span $K = \frac{M}{f_{cd}bd^2} = \frac{20.4 \times 10^6}{40 \times 10^3 \times 149^2} = 0.023$&lt;br&gt;$y = 149(0.5 + \sqrt{(0.25 - 0.023 \div 0.9)}) = 145$ (but $0.95 \times 149 = 141.5$)&lt;br&gt;$A_s = \frac{M}{0.95fy_y} = \frac{20.4 \times 10^6}{0.95 \times 460 \times 141.5} = 33.0 \text{mm}^2$&lt;br&gt;Check for shear: $V = \frac{0.5 \times 64.9 \times 10^3}{10^3 \times 149} = 0.22 \text{N/mm}^2 &lt; V_c$ (ok)</td>
<td></td>
</tr>
<tr>
<td>3.4.4 Table 3.9</td>
<td><strong>DEFLECTION</strong>&lt;br&gt;Basic span/eff. depth ratio = 26 max.&lt;br&gt;$M = 20.4 \times 10^6 = 0.92, f_s = \frac{2 \times 460 \times 330}{3 \times 377} = 268.4 \text{N/mm}^2$&lt;br&gt;Modification factor for tension reinf. = 1.5&lt;br&gt;Allowable span/eff. depth ratio = 26x1.5 = 39&lt;br&gt;Actual = $\frac{5000}{149} = 33.56$ (½d ratio ok)</td>
<td></td>
</tr>
<tr>
<td>3.12.11.27</td>
<td><strong>CRACKING</strong>&lt;br&gt;Spacing between bars = 300-12 = 288 mm &lt; 3d (Spacing ok)</td>
<td></td>
</tr>
<tr>
<td>3.12.3.4</td>
<td><strong>TIE PROVISION</strong>&lt;br&gt;East &amp; West internal tie $F_t = 36 \text{kN/m width}$&lt;br&gt;Tie force: $F_t(\frac{3k + 9k}{7.5}) = 36(\frac{4.7 + 4}{7.5}) = 41.8 \text{kN/m width}$&lt;br&gt;Min. area = $\frac{41.8 \times 10^3}{460} = 91 \text{mm}^2$&lt;br&gt;Lap = $\frac{91 \times 35}{377} = 8.5 \text{d}$&lt;br&gt;Upper: $300 \text{mm}$&lt;br&gt;Bottom: $T12 @ 300$ (377 mm²/m)</td>
<td></td>
</tr>
</tbody>
</table>
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 -I.-

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**

<table>
<thead>
<tr>
<th>Bar mark</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>45T12-1-300B1</td>
<td>Minimum area of tension reinforcement = 0.0013 × 1000 × 175 = 228 mm²/m.</td>
</tr>
<tr>
<td>3.12.11.2.7</td>
<td>Maximum clear spacing of tension bars = lesser of 750 mm or 3d, i.e. 3d = 3 × 149 = 447 mm.</td>
</tr>
<tr>
<td>h &lt; 200, therefore no further check on spacing</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Main tension bars T12 @ 300, A = 377 mm² &gt; minimum 228 mm²/m - OK.</td>
</tr>
<tr>
<td>If curtailed, A = 377/2 = 189 mm² &lt; minimum 228 mm²/m - not OK.</td>
<td></td>
</tr>
<tr>
<td>3.12.3.4</td>
<td>Bars lapped 300 mm at bottom support to provide continuous tie.</td>
</tr>
<tr>
<td>2,3</td>
<td>Secondary bars - use T10 @ 300 (262 mm²/m).</td>
</tr>
<tr>
<td>3.12.8.11</td>
<td>Minimum lap = 300 mm &gt; 15 × 10 = 150 mm. Lapping reduces bar lengths for easier handling on site.</td>
</tr>
<tr>
<td>7</td>
<td>Laps are shown staggered for effective crack control.</td>
</tr>
<tr>
<td>3.4.1.5</td>
<td>Minimum transverse reinforcement is placed across the full flange width of the edge beam (minimum width = 650 mm, see page 16).</td>
</tr>
<tr>
<td>6</td>
<td>Minimum area = 0.0015 × 1000 × 175 = 263 mm²/m - use T10 @ 300 (262 mm²/m).</td>
</tr>
<tr>
<td>8</td>
<td>Main tension bars over support T12 @ 300 as bar mark 1.</td>
</tr>
<tr>
<td>3.12.10.3</td>
<td>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.</td>
</tr>
</tbody>
</table>
# First-floor main beam

**two-span flanged beam**

### 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.

### DURABILITY and FIRE RESISTANCE

Nominal cover for mild conditions of exposure = 20 mm.
Nominal cover for 300 mm wide beam for 1 hr period = 20 mm.

### Loading

Dead load from 175 slab (page 8) = 5 x 4.7 = 23.5
Self-weight (0.5 - 0.175) 0.3 x 24 = 2.3
Characteristic dead load on beam imposed = (P.8) = 5 x 4 = 20.0 kN/m.
Maximum design load = 1.4gK + 1.6gK = 36.12 + 32 = 68.12 kN/m.
Minimum = 1.0gK = 25.8 kN/m + 1.6gK

### Bending Moments (kNm)

Results from Analysis

<table>
<thead>
<tr>
<th>Case</th>
<th>BM</th>
<th>25.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>25.8</td>
<td>68.12</td>
</tr>
<tr>
<td>II</td>
<td>25.8</td>
<td>68.12</td>
</tr>
<tr>
<td>III</td>
<td>25.8</td>
<td>68.12</td>
</tr>
</tbody>
</table>

- **Spans**
  - **CASE I loading**
    - Beam moments
      - Upper Column mts.
      - Lower mts.
    - Shear (kN)
  - **CASE II loading**
    - Beam moments
      - Upper Column mts.
      - Lower mts.
    - Shear (kN)
  - **CASE III loading**
    - Beam moments
      - Upper Column mts.
      - Lower mts.
    - Shear (kN)
**REDISTRIBUTION**

**CASE I** Reduce 179 to 155 (see II)

- 402 = 282 (-30%)
- 193 = 155 (-20%)
- 349 = 282 (see I)

**CASE III** No redistribution.

**BENDING MOMENT ENVELOPE**

- **CASE I** Loading
- **CASE II**
- **CASE III**
- Redistributed
- Envelope

Envelope --- Moments in kNm.

**SHEAR FORCE ENVELOPE**

Reactions in kN.
### BS 8110 ref.

#### MAIN BEAM, 1st. FLOOR (continued)

### Internal Support:

From B.M. envelope, page 11, \( M = 282 \text{ kNm} \)

\[
\beta_b = \frac{282}{402} = 0.7, \quad \frac{x}{d} < 0.3
\]

\[
K' = 0.402 (0.7 - 0.4) - 0.18 (0.7 - 0.4)^2 = 0.104
\]

\[
K = \frac{M}{f_{cd} d^2} = \frac{282 \times 10^6}{40 \times 300 \times 440^2} = 0.121 > 0.104
\]

Comp. reinforcement reqd.\( d_i = 50 \), \( \frac{d_i}{d} = 0.38 > 0.37 \)

\[
x = (0.5 - 0.45) \cdot 0.45 = 1 - 0.49 \times 0.3 = 0.865
\]

\[
A_S = 0.121 - 0.104 \times 40 \times 300 \times 440 = 232 \text{ mm}^2
\]

Check minimum \( A'_S = 0.2 \times (300 \times 500) / 100 = 300 \text{ mm}^2 \)

\[
A_S = \frac{0.104 \times 40 \times 300 \times 440^2 + 232}{0.95 \times 460 \times 0.865 \times 4.4} = 1685 \text{ mm}^2
\]

### Bm mid-span

From B.M. envelope, page 11, \( M = 328 \text{ kNm} \)

\[
\frac{M}{f_{cd} d^2} = \frac{328 \times 10^6}{40 \times 420 \times 450^2} = 0.029
\]

\[
\frac{x}{d} = 0.5 \sqrt{0.25 - \frac{0.029}{0.9}} \approx 0.97 > 0.95 \quad ; \quad x = 450 (1 - 0.95) = 50 \text{ mm} \quad \text{(N.A. in flange)}
\]

\[
A_S = 0.128 \times 10^6
\]

\[
= 0.95 \times 460 \times 0.95 \times 450 = 1756 \text{ mm}^2
\]

### Bm end-support

From B.M. envelope, page 11, \( M = 155 \text{ kNm} \)

\[
\beta_b = 155 \times 193 = 0.8 \quad \frac{x}{d} < 0.4
\]

\[
\frac{M}{f_{cd} d^2} = \frac{155 \times 10^6}{40 \times 300 \times 450^2} = 0.064
\]

\[
\frac{x}{d} = 0.5 \sqrt{0.25 - \frac{0.064}{0.9}} = 0.92 \quad ; \quad \frac{x}{d} = 1 - 0.92 = 0.10 \text{ (OK)}
\]

\[
A_S = \frac{155 \times 10^6}{0.95 \times 460 \times 0.92 \times 450} = 857 \text{ mm}^2
\]

Check force in bar = \( 775 \times 0.95 \times 460 \times 491 = 190 \text{ kN} \)

### Gm end-support

From B.M. envelope, page 11, \( M = 86 \text{ kNm} \)

\[
\frac{M}{f_{cd} d^2} = 0.35, \quad \frac{x}{d} = 0.95
\]

\[
A_S = \frac{86 \times 10^6}{0.95 \times 460 \times 0.95 \times 450} = 460 \text{ mm}^2
\]

### Gm mid-span

From B.M. envelope, page 11, \( M = 153 \text{ kNm} \)

\[
\frac{M}{f_{cd} d^2} = 0.017, \quad \frac{x}{d} = 0.95
\]

\[
A_S = \frac{153 \times 10^6}{0.95 \times 460 \times 0.95 \times 450} = 819 \text{ mm}^2
\]

### Table 325

Check for minimum reinforcement in flanged beam

web in tension \( \frac{0.0018 \times 300 \times 500 = 270 \text{ mm}^2 \text{ ok}}{\text{flange in tension over supports}: \frac{0.0026 \times 300 \times 500 = 390 \text{ mm}^2 \text{ ok}}{\text{main tension}}

---

### OUTPUT

#### Main tension

\[
\text{main tension (1960 mm}^2\text{)}
\]

### Diagram

- Main tension (1960 mm²)
- Compression (392 mm²)
- Tension (2360 mm²)
- Bending (125 mm)
- Flange in tension over supports (390 mm²)
**BS 8110 Calculations**

### Shear Reinforcement

- **Min. effective tension reinf.**
  \[
  \frac{100A_s}{bd} = \frac{100 \times 982}{300 \times 450} = 0.73, \quad V_c = 0.57 \left( \frac{d}{25} \right)^{0.66} = 0.66 \text{ N/m}^2
  \]

### Tensile Strength

- **Max. shear strength**
  \[
  V_b = 0.75 \times 450 = 338 \text{ N/m} \text{ as min.}
  \]

### Crack Widths

**Limit Crack Widths by Limiting Bar Spacing**

- **Tension reinf.**
  \[
  M = 2T_{25} (982 \text{ mm}^2)
  \]

### Deflection

**Basic b/d ratio**
\[
\frac{M}{bd^2} = \frac{220}{140 \times 450^2} = 0.075, \quad f_0 = 2 \times 460 \times 175 \times 0.75 = 635 \text{ N/m}^2
\]

### Frame Analysis

**Cover to lapped bars**
\[
35 < 2 \times 25 \text{ mm}
\]

**Spacing between adjacent laps**
\[
100 < 6 \times 25 \text{ mm}
\]

**Lap**
\[
49 \times 25 < 382 = 906 \text{ mm}
\]

**Use Bottom Bars**

Use 1/2 bars at external columns.
Commentary on bar arrangement

<table>
<thead>
<tr>
<th>BS 8110 ref</th>
<th>Bar marks</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>3.3.1.2</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>3.12.9.1</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>3.12.8.14</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>3.3.2.3</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>3.4.2.10</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>3.12.9.1</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

- This beam shows loose splice bars at each column intersection. This method simplifies detailing and fixing the span cage can readily be prefabricated.
- 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.
- Remaining tension bars stopped off as shown in the curtailment diagram above.
- Check maximum amount of reinforcement at laps < 40% breadth.
  - 4 x 25 = 100 mm < 0.4 x 300 = 120 mm - OK.
- 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.
- The two tension bars are stopped 50 mm from the column face to avoid the column bars beyond.
- 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.
- Top legs project from centre-line into span, minimum dimensions shown in the curtailment 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. Check bearing stress inside bends. Use 75° 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 75° 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 < 15 bar size.
# Edge beam

**interior-span flanged beam**

## BS 8110 ref.

### DURABILITY and FIRE RESISTANCE

- Nominal cover for severe conditions of exposure = 40 mm.
- **300 wide beam for 1 hr. period** = 20 mm.

### LOADING

- Dead load from slab (1250 strip, p.10) = 23.5 x 1.25 = 29.4 kN
- Self weight (0.35 - 0.175) x 0.3 x 24 x 5 = 6.3 kN
- Cladding = 5 kN/m (page 6) = 5 x 5 = 25.0 kN
- Characteristic dead load = 60.7 kN
- Imposed load from slab (p.10) = 20 x 1.25 = 25.0 kN
- Design load = 1.4 Gk + 1.6 Qk = 85 + 40 = 125.0 kN.

### ULTIMATE B.M.'s

- **Interior supports:** M = 0.08 FL = 0.08 x 12.5 x 5 = 50.0 kNm
- **Mid-int span:** M = 0.07 FL = 0.07 x 12.5 x 5 = 43.8 kNm

### REINFORCEMENT

- **Interior supports:** \( f_{cd} \) = \( f_{cd} \) = 0.053
- \( \frac{f_{cd}}{d} \) = (0.5 + (0.25 - \frac{0.35}{0.9}) = 0.937, A_s = \frac{50 \times 10^6}{40 \times 300 \times 280} = 0.053 \text{ mm}^2

### MID-INT SPAN

- **Eff. flange width** = 1000 + \( \frac{7}{10} \times 5000 = 650 \text{ mm}
- **Shear force** = 0.55 F x 125 = 68.75 kN
- **Design shear force** = 68.75 - (0.15 + 0.28) = 58 kN
- **Table 3-5**
- **Table 3-6**

### SHEAR FORCE

- **Table 3-6**
- **Table 3-7**

### DEFLECTION

- Basic \( f_{cd} \) = 22.0 mm, \( \frac{f}{d} \) = 0.46 > 0.3
- **Modification factor** = 1.53
- **Allowable span/eff. depth ratio** = 22 x 1.53 = 33.7
- **Actual** = 22 x 1.53 = 33.7

### CRACKING

- **Bottom bars** \( f_s = 278 \text{ kN/m}^2 \)
- **Allowable clear spacing** = \( \frac{4700}{278} = 16.9 \text{ mm} \), side cover \( 25 < \frac{16.9}{2} \), check corner
- **Top bars** \( f_s = 2 \times 460 \times 436 = 71.5 \text{ kN/m} \), cover \( 25 < \frac{16.9}{2} \), check corner
- **Allowable clear spacing** = \( \frac{4700}{230} = 20.4 \text{ mm} \)

### TIE PROVISION

- **Peripheral tie, F_t = 36 kN, At = \frac{36 \times 10^3}{460} = 78 \text{ mm}^2
- Use 2T12, Min. lap = 35 x 12 x 78 = 145 < 300 \text{ mm}

### OUTPUT

- Minimum cover to links = 40 mm
- \( \phi_K = 60.7 \text{ kN} \)
- \( \phi_K = 25.0 \text{ kN} \)
- \( F = 125.0 \text{ kN} \)
- \( 2T12 \text{ min.} \)
- **Table 3-7**
- **Table 3-8**
- **Table 3-9**
Commentary on bar arrangement

<table>
<thead>
<tr>
<th>BS 8110 ref</th>
<th>Bar marks</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.12.8.11</td>
<td>1</td>
<td>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.</td>
</tr>
<tr>
<td>3.12.10.2</td>
<td>2</td>
<td>Separate splice bars are fixed inside vertical column bars.</td>
</tr>
<tr>
<td>Figure 3.24</td>
<td></td>
<td>Minimum area = 30% $A_{\text{min-span}} = 0.3 \times 364 = 109$ mm$^2$. Use 2T12 = 226 mm$^2$.</td>
</tr>
<tr>
<td>Table 3.27</td>
<td></td>
<td>Lap = $35 \times 12 \times 109/226 = 203$ mm &gt; $15 \times 12 = 180$ mm &gt; $300$ mm. Use 300 mm lap.</td>
</tr>
<tr>
<td>3.12.10.2</td>
<td>3</td>
<td>Link hanger bars also provide support for slab top reinforcement.</td>
</tr>
<tr>
<td>Figure 3.24</td>
<td></td>
<td>Minimum area = 20% $A_{\text{support}} = 0.2 \times 436 = 87$ mm$^2$. Use 2T12 = 226 mm$^2$.</td>
</tr>
<tr>
<td>3.12.10.2</td>
<td>4</td>
<td>Tension reinforcement over support is fixed inside vertical column bars.</td>
</tr>
<tr>
<td>Figure 3.24</td>
<td></td>
<td>Bars are curtailed at 0.25 span from face of support = $0.25 \times 5000 = 1250$ mm &gt; $45 \times 20 = 900$ mm</td>
</tr>
<tr>
<td>-</td>
<td>5</td>
<td>Closed links are shape code 61.</td>
</tr>
</tbody>
</table>
Columns

slender and short columns

BS 8110 ref

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.

3.4 " " 300x300 column for 1hr period = 20 mm.

External 40 mm.

INTERNAL COLUMN (Foundation → Roof)

AXIAL LOADING and MOMENTS FROM ANALYSIS

<table>
<thead>
<tr>
<th>LOAD CASE</th>
<th>TOTAL</th>
<th>IMPOSED</th>
<th>DEAD</th>
<th>TOP</th>
<th>BOTTOM</th>
</tr>
</thead>
<tbody>
<tr>
<td>LEVEL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof</td>
<td>248</td>
<td>244</td>
<td>54</td>
<td>9</td>
<td>34</td>
</tr>
<tr>
<td>Sw</td>
<td>210</td>
<td>133</td>
<td>46</td>
<td>9</td>
<td>32</td>
</tr>
<tr>
<td>3rd Fl.</td>
<td>298</td>
<td>290</td>
<td>140</td>
<td>9</td>
<td>100</td>
</tr>
<tr>
<td>Sw</td>
<td>249</td>
<td>117</td>
<td>117</td>
<td>9</td>
<td>357</td>
</tr>
<tr>
<td>2nd Fl.</td>
<td>298</td>
<td>290</td>
<td>140</td>
<td>9</td>
<td>357</td>
</tr>
<tr>
<td>Sw</td>
<td>249</td>
<td>117</td>
<td>117</td>
<td>9</td>
<td>357</td>
</tr>
<tr>
<td>1st Fl.</td>
<td>300</td>
<td>292</td>
<td>141</td>
<td>9</td>
<td>614</td>
</tr>
<tr>
<td>Sw</td>
<td>252</td>
<td>120</td>
<td>118</td>
<td>9</td>
<td>614</td>
</tr>
<tr>
<td>Fans.</td>
<td>873</td>
<td>462</td>
<td>1273</td>
<td>14</td>
<td>1182</td>
</tr>
</tbody>
</table>

F2 (Typical)

3.8.1.6 (Foundation → 1st Floor).

Eqn 30 Effective height

\[ l_e = \beta \ell_0 \]

Table 3.19

N → S \( \beta = 0.9 \) (End condition: top 1, bottom 3)

\[ \ell_{ex} = 0.9 \times 4.5 = 4.05 \text{m} \]

\[ \ell_e = \frac{4.05}{0.3} = 13.5 \]

E → W \( \beta = 0.95 \) (End condition: top 2, bottom 3)

\[ \ell_{ex} = 0.95 \times 4.825 = 4.58 \text{m} \]

\[ \ell_e = \frac{4.58}{0.3} = 15.26 > 15 \]

SLENDER COLUMN
INTERNAL COLUMN (Foundation = 1st Fl) continued

Calculation:

Load Case 1

| BS 8110 ref. | Imposed | 100 + 0.8 × 773 | = 718 |
| BS 6399 | Dead | = 1273 |
| Part 1 | N | = 1991 kN. |
| 3.8.3.2 | M₁ = 0, M₂ = 19, 0.4 M₂ = 7.6 kNm. |
| Mᵢ = 0.4 M₁ + 0.6 M₂ = 0 + 0.6 × 19 = 11.4 > 7.6 |
| Eqn. 32 | Mₐdd = \( \frac{N}{1000} \left( \frac{1891 \times 0.3 \times 13.5^2}{2000} \right) = 54.4 \text{ kNm.} \) |
| 3.8.3.2 | Mₘₐₓ, eccentricity = \( e_{\text{mm}} = 0.05 \times 300 = 15 \text{ mm} \) |
| Max. design moment will be greatest of:
| (a) | \( M₂ = 19 \text{ kNm.} \) |
| (b) | \( M₁ + Mₚₐdd = 11.4 + 54.4 = 65.8 \text{ kNm.} \) |
| (c) | \( e_{\text{min}} \times N = \frac{15}{1000} \times 1991 = 29.9 \text{ kNm.} \) |
| N | = \( \frac{1991 \times 13.5}{300^2} = 22.1, \frac{M}{bh^2} = \frac{65.8 \times 10^6}{300^3} = 2.44 \) |
| Assume \( d = 300 - 40 - 13 = 247 \text{ mm.} \) |
| \( \frac{a}{h} = \frac{247}{300} = 0.82, \frac{100 \text{ A}_{\text{sc}}}{bh} = 2.8, \text{ A}_{\text{sc}} = 2520 \text{ mm}^2 \) |

3.8.1.1

| Eqn. 33 | \( N_{\text{ba}} = 0.25 \times \text{bd} = \frac{0.25 \times 40 \times 300 \times 247 \times 10^3}{741 \text{ kN.}} \) |
| Since | \( N > N_{\text{ba}}, \text{try } 4725 \text{ (1960 mm²).} \) |

3.8.3.1

| N_{\text{ba}} = (0.45 \times 40 (300^2 - 1960) + 0.87 \times 460 \times 1960) \times 10^3 | = 2369 \text{ kN.} |
| Eqn. 33 | \( K = \frac{2369 - 1991}{2369 - 74} = 0.23 \) |
| Design Moment = 11.4 + 0.23 × 54.4 = 23.9 > 29.9 \text{ kNm.} |
| \( 100 \text{ A}_{\text{sc}} = \frac{100 \times 1960}{300^2} = 2.18, \frac{N}{bh} = 22.1 \) (as before) |
| From chart, \( \frac{M}{bh^2} = 1.6, \frac{M}{bh^2} = 43.2 > 29.9 \text{ kNm.} \) ok. |

Check, considering:

Load case 1 above 1st Floor, and load case 2 and moment at 1st Floor.

| BS 8110 ref. | Imposed | 100 + 0.8 (514 + 137) | = 621 |
| Part 1 | Dead | = (966 + 289) \( \frac{1255}{1876} \text{ kN.} \) |
| Eqn. 32 | \( M₁ = 0, M₂ = 34 \text{ kNm.} \) |
| \( Mᵢ = 0 + 0.6 \times 34 = 20.4 \text{ kNm.} \) |
| \( Mₐdd = \frac{1876 \times 54.4}{1991} = 51.3 \text{ kNm}, K = \frac{2369 - 1876}{2369 - 74} = 0.3 \) |
| Design Moment = 20.4 + 0.3 × 51.3 = 35.8 > M₂ |
| \( N = 20.8, \frac{100 \text{ A}_{\text{sc}}}{bh} = 2.18, \frac{M}{bh^2} = 2.0 \) |
| \( M = 54 > 35.8 \text{ kNm.} \) ok. |

TIE PROVISION

Load = 1.05 (214 + 16/3) = 281 kN

\( A_{\text{s}} = 281 \times 10^3 / 460 \times 610 \text{ mm}^2 < 1960 \text{ mm}^2 \) ok.
### EXTERNAL COLUMN (Foundation → Roof)

**Axial Loading and Moment from Analysis**

<table>
<thead>
<tr>
<th>LOAD CASE</th>
<th>1</th>
<th>2</th>
<th>1</th>
<th>2</th>
<th>1</th>
<th>2</th>
<th>TOP</th>
<th>BOTTOM</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL</td>
<td>192</td>
<td>197</td>
<td>42</td>
<td>43</td>
<td>150</td>
<td>154</td>
<td>108</td>
<td>115</td>
</tr>
<tr>
<td>IMPOSED</td>
<td>99</td>
<td>99</td>
<td>9</td>
<td>9</td>
<td>99</td>
<td>99</td>
<td>98</td>
<td>105</td>
</tr>
<tr>
<td>DEAD</td>
<td>125</td>
<td>125</td>
<td>158</td>
<td>163</td>
<td>258</td>
<td>262</td>
<td>98</td>
<td>105</td>
</tr>
<tr>
<td>1st.</td>
<td>245</td>
<td>253</td>
<td>274</td>
<td>283</td>
<td>788</td>
<td>500</td>
<td>108</td>
<td>117</td>
</tr>
<tr>
<td>2nd.</td>
<td>247</td>
<td>255</td>
<td>116</td>
<td>120</td>
<td>131</td>
<td>135</td>
<td>98</td>
<td>105</td>
</tr>
<tr>
<td>3rd.</td>
<td>247</td>
<td>255</td>
<td>125</td>
<td>125</td>
<td>125</td>
<td>125</td>
<td>106</td>
<td>117</td>
</tr>
<tr>
<td>Edans</td>
<td>125</td>
<td>125</td>
<td>117</td>
<td>119</td>
<td>130</td>
<td>134</td>
<td>71</td>
<td>76</td>
</tr>
</tbody>
</table>

#### 3.8.1.6

**Eqn. 30**

**Effective height**

\[ h_e = \beta h \]

**Table 3.19**

- **N→S** \( \beta = 0.9 \) (end condition: top 1, bottom 3)
  - \( l_e = 0.9 \times 3.5 = 3.15 \text{ m} \)
  - \( \frac{l_e}{h} = \frac{3.15}{0.3} = 10.5 \)

- **E→W** \( \beta = 0.9 \) (end condition: top 1, bottom 3)
  - \( l_{ey} = 0.9 \times 3.65 = 3.285 \)
  - \( \frac{l_{ey}}{h} = \frac{3.285}{0.3} = 10.95 < 15 \)

#### 3.8.1.3

**BS6399 Part 1**

Just above 1st floor, \( N = 800 + 43 + 0.9 \times 240 = 1059 \text{ kN} \)

\[ M = 117 \text{ kNm} \]

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

Assume \( d = 300-50-13 = 237 \text{ mm} \)

\[ \frac{d}{h} = \frac{237}{300} = 0.79 \]

**BS6399 Part 1**

Below 1st floor, \( N = 1059 + 43 + 0.8 \times 359 = 1389 \text{ kN} \)

\[ M = 76 \text{ kNm} \]

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

\[ A_{sc} = 1350 \text{ mm}^2 \]
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.

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 (14d).
Length of short projection beyond crank = compression lap +, say, 75 mm for tolerance.

Reinforcement area at laps < 10% bh.

Bars project above first-floor slab level to provide a compression lap above the kicker. Bar projection = 35 \times 0.875/95 \times 25 \text{ mm} + 75 \text{ mm} for kicker = 875 \text{ mm}, i.e. compression lap = 800 mm.

As bar mark 1, but bars provide a tension lap above 1st floor kicker. Cover = 50 mm.
Clear distance between adjacent laps = 100 mm < 6 \times 25 \text{ mm}; i.e. use factor 1.4
Projection = 1.4 \times 35 \times 0.875/95 \times 25 + 75 = 1195 \text{ mm}, say 1200 mm, i.e. tension lap = 1125 mm.

Sum of bar sizes at tension lap = 4 \times 25 = 100 \text{ mm}. 100/300 \times 100 = 33\% < 40\% - OK.
This detail provides the maximum lever arm and is the preferred detail for column/beam intersections.

Similar to mark 2 links, but extending to underside of main beam. Cover to vertical bars = 50 mm.

These U-bars are provided to restrain the vertical bars in the external face of the column.
## Foundation

**reinforced pad footing**

### BS 8110 ref.

<table>
<thead>
<tr>
<th>CALCULATIONS</th>
<th>OUTPUT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DURABILITY</strong></td>
<td>Nominal cover 40mm. b1m, 75mm ends</td>
</tr>
<tr>
<td><strong>LOADING</strong></td>
<td></td>
</tr>
<tr>
<td>Dead</td>
<td>1273</td>
</tr>
<tr>
<td>Imposed</td>
<td>1273/1.4 = 909</td>
</tr>
<tr>
<td><strong>Allow 10kN/m² extra over soil displaced by concrete and for ground floor loading.</strong></td>
<td></td>
</tr>
<tr>
<td>Pad area required</td>
<td>= 1358/ (200 - 10)</td>
</tr>
<tr>
<td>Adopt 2.75 x 2.75 x 0.6 thick ftg, area provided</td>
<td>= 7.5 &gt; 7.15m²</td>
</tr>
<tr>
<td><strong>REINFORCEMENT</strong></td>
<td></td>
</tr>
<tr>
<td>Moment at face of column</td>
<td>= 263 x 2.75 x 1.2²</td>
</tr>
<tr>
<td>Average d</td>
<td>= 0.00 - 40 - 25</td>
</tr>
<tr>
<td>( f_{ce} = 5350 )</td>
<td>= 543 x 10⁶</td>
</tr>
<tr>
<td>( \frac{2}{d} = 0.95 ), ( A_s = \frac{543 x 10^6}{0.93 x 460 x 0.95 x 535} )</td>
<td>= 2445 mm²</td>
</tr>
<tr>
<td><strong>ULTIMATE SHEAR</strong></td>
<td></td>
</tr>
<tr>
<td>Minimum ( V_c = 0.34 (40/25)^{1/3} = 0.4 N/mm² )</td>
<td></td>
</tr>
<tr>
<td><strong>Condition 1:</strong></td>
<td></td>
</tr>
<tr>
<td>Shear force ( V @ d ) from col. face</td>
<td>= ( \frac{1991 x (1375 - 150 - 535)}{1375} ) = 76.20 kN</td>
</tr>
<tr>
<td><strong>Stress</strong> ( v = \frac{V}{A} )</td>
<td>= 500 x 10³</td>
</tr>
<tr>
<td>Shear force ( V @ 2d ) from col. face</td>
<td>= ( \frac{1991 x (1375 - 150 - 2 x 535)}{1375} ) = 112 kN</td>
</tr>
<tr>
<td><strong>Stress</strong> ( v = \frac{V}{A} )</td>
<td>= 125 x 10³</td>
</tr>
<tr>
<td><strong>Condition 2:</strong> (punching shear, usually the more critical)</td>
<td></td>
</tr>
<tr>
<td>( V_{max} = \frac{V}{A} )</td>
<td>= ( \frac{1991 x 10³}{4 x 300 x 535} ) = 3.1 &lt; 5.0 N/mm²</td>
</tr>
<tr>
<td>Critical perimeter</td>
<td>= 4 (300 + 3 x 535)</td>
</tr>
<tr>
<td>Area within</td>
<td>= (0.3 + 3 x 0.535)²</td>
</tr>
<tr>
<td>Shear on</td>
<td>= 263 (2.75² x 3.63)</td>
</tr>
<tr>
<td>( V = \frac{1034 x 10³}{7620 x 535} ) = 0.25 N/mm²</td>
<td></td>
</tr>
<tr>
<td><strong>CRACKING</strong></td>
<td></td>
</tr>
<tr>
<td>Clear distance</td>
<td>= 330 &lt; 750 mm</td>
</tr>
<tr>
<td>100 ( A_s ) / ( 2750 x 535 )</td>
<td>= 100 x 2445</td>
</tr>
<tr>
<td><strong>Crack widths ok.</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Space bars uniformly across section, say @ 350crs each way.</strong></td>
<td></td>
</tr>
</tbody>
</table>
Commentary on bar arrangement

<table>
<thead>
<tr>
<th>BS 8110 ref</th>
<th>Bar marks</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.12.8.1</td>
<td>1</td>
<td>Straight bars extend full width of base, less end covers. Bars should project a minimum tension bond length beyond the column face = 35 × 20 = 700 mm &lt; 1150 mm = OK.</td>
</tr>
<tr>
<td>Table 3.27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.3.1.4</td>
<td>2</td>
<td>The underside of base is concrete blinded, cover = 40 mm. Column starter bars are wired to bottom mat. Minimum projection above the top of base is a compression lap + kicker = 35 × 0.87 + 95 × 25 + 75 = 875 mm, i.e. lap = 800 mm (see p. 21).</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Links are provided to stabilize and locate the starter bars during construction. These are the same size as the column links above.</td>
</tr>
</tbody>
</table>
# Shear Wall

**External Plain Concrete Wall**

### Design as Braced Plain Concrete Wall (BS 8110)

<table>
<thead>
<tr>
<th>BS 8110 ref.</th>
<th>CALCULATIONS</th>
<th>OUTPUT</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.9.4.3</td>
<td>Design as braced plain concrete wall</td>
<td>175 mm thick</td>
</tr>
<tr>
<td>1.2.4</td>
<td><strong>Eff. Ht.</strong> JDN = 175 ft.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$h_e = 0.875 \times 3825 = 3350$ mm; $h = 175$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e = 3350 = 19.2 &gt; 12$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1:2 &gt; 1 hour</td>
<td></td>
</tr>
</tbody>
</table>

### Durability and Fire Resistance

Nominal cover: (a) severe exposure = 40 mm. (b) mild = 20 mm.

- Fire resistance of 175 plain wall = 12 > 1 hour
- External cover = 40 mm
- Internal = 20 mm
- 1 hr. fire resistance ok.

### Loading e Top of Foundations

Dead load from: 1st, 2nd, 3rd fl. + roof = 0.5(3x23.5 + 28.5) = 49.5 kN/m.

- Self weight = 0.175x24x15.5 = 65.1 kN/m

- Characteristic dead load = 114.6 kN/m

- Imposed (slabs) = 2.5(1.5 + 3x4x0.8) = 27.8 kN/m

- $q_k = 114.6$ kN/m |
- $q_i = 27.8$ kN/m |

### Wind Loading

$V_s = V_6 S_a = 22$ m/sec, $V_e = V_6 S_e = 37.7$ m/sec

- Dynamic pressure $q_s = 0.613 V_s^2 = 0.613 \times 37.7 = 0.87$ kN/m²

- Total wind load = $0.85 \times (C_{p e} + C_{p c} + C_{p m})$ A

- $M$ end wall = $0.5 \times 4234.8 = 1694$ kN/m

- $f = -14.3^2 = 49.7$ kN/m²

- $q_k = 29.9$ kN/m² |
- $q_i = 19.9$ kN/m² |

### Vertical Loading Intensity (U.S. Design Loads)

**Table 2.1**

- Load combination 1: $f_1 = 1.4 \times 114.6 + 1.6 \times 27.8 = 204.9$ kN/m

- $f_2 = 1.4 \times 114.6 + 1.4 \times 49.7 = 220$ kN/m

- $f_3 = 1.0 \times 114.6 + 1.4 \times 49.7 = 45$ kN/m

- $f_4 = 1.2 \times (114.6 + 27.8 + 49.7) = 295.5$ kN/m

- $e_x = \frac{1}{20}$, $e_a = \frac{1}{2500}$

- $h <= 0.3 \times (h_x + h_y) = 20.3 < 0.72 h_y, e_y <= 0.72 h_y, e_y <= 1344$ kN/m²

- Actual U.S. Design Loads $< q_k$ in all cases

### Shear

- Horizontal design shear force $V = 1.4 \times 4234.8 = 29.64$ kN

- Min. vertical - load $N = 1.0 \times 114.6 \times 143 = 1639$ kN

- $V = 29640 < 0.25 \times 175 \times 14300 = 0.12 \times 0.45$ kN/mm²

- ok.

### Tie Provision @ 1st Floor

- Peripheral tie: $A_s = \frac{36 \times 10^3}{460} = 78$ mm²

- $T_{10} \times 200$ EF, Horiz. (393 mm²) | (78.5 mm²)

### Horizontal Reinforcement in wall 0.5 m above and below slab adequate

- Wall tie: $A_s = \frac{5 \times 25 \times 35 \times 14.0}{2.5} \text{ but } 0.03 \times 2049 = 0.72 A_s = 47.9 \times 10^3 = 104$ mm²

- $A_s = 460$

### Footing

- Max. pressure due to dead + imp. + wind = $114.6 + 27.8 + 49.7 = 192$ kN/m²

- $f = 900$ wide, pressure = $\frac{192}{90} = 2.13$ kN/m² $< 2.5 < 0.190$

- $B = 230 \times 0.5 = 115.1$ kN/m, $A_s = 0.95 \times 460 = 182$ mm²

- $d = 200$

- $M = 250 \times 0.5 \times 0.725 \times 35 \times 14.3 = 182$ mm²

- $A_s = 460$

- Min. area = $0.12 \times 10^3 \times 250 = 325$ mm²

<table>
<thead>
<tr>
<th>Footing</th>
<th>BS 8004 1.3.2.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom</td>
<td>$T_{12} \times 300$ EF, Horiz. (377 mm²)</td>
</tr>
</tbody>
</table>

---

24
Commentary on bar arrangement

<table>
<thead>
<tr>
<th>BS 8110 ref</th>
<th>Bar marks</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>Wall starters match vertical reinforcement. Minimum projection of horizontal legs beyond the wall face is a design tension bond length = $35 \times 182/377 \times 12 = 203 \text{ mm} &lt; 287 \text{ mm}$. This provides the footing reinforcement. Minimum projection above top of base is a compression lap + kicker = $35 \times 12 + 75 = 495 \text{ mm}$, say 525 mm, i.e. lap = 450 mm.</td>
</tr>
<tr>
<td>Table 3.27</td>
<td>2</td>
<td>Minimum longitudinal reinforcement provided.</td>
</tr>
<tr>
<td>3.3.1.4</td>
<td></td>
<td>Underside of footing is concrete blinded, cover = 40 mm.</td>
</tr>
<tr>
<td>Table 3.25</td>
<td>2</td>
<td>Minimum horizontal reinforcement. Area = $0.25 % \times 1000 \times 175 = 438 \text{ mm}^2/\text{m}$. (T12 @ 200 EF = 754 mm/$\text{m}$. ) 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.</td>
</tr>
<tr>
<td>3.9.4.19</td>
<td>3</td>
<td>Minimum vertical reinforcement. Area = $0.25 % \times 1000 \times 175 = 438 \text{ mm}^2/\text{m}$. (T12 @ 200 EF = 786 mm/$\text{m}$. ) Provide at least a tension lap = $35 \times 10 = 350 \text{ mm}$, 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 = $35 \times 10 = 350 \text{ mm}$, say 450 mm.</td>
</tr>
<tr>
<td>4.5.6</td>
<td></td>
<td>Minimum horizontal reinforcement. Area = $438 \text{ mm}^2/\text{m}$. (T12 @ 200 EF = 786 mm/$\text{m}$. ) Provide at least a tension lap = $35 \times 10 = 350 \text{ mm}$, 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 = $35 \times 10 = 350 \text{ mm}$, say 450 mm.</td>
</tr>
<tr>
<td>Table 3.27</td>
<td>7,8</td>
<td>Peripheral tie at first floor. L-bars at either end provide continuity with edge beams. Laps, say 450 mm.</td>
</tr>
<tr>
<td>3.12.3.4</td>
<td>9</td>
<td>Wall spacers maintain location of each face of reinforcement.</td>
</tr>
</tbody>
</table>
**Staircase**

**end-span continuous slab**

<table>
<thead>
<tr>
<th>BS 8110 ref.</th>
<th>CALCULATIONS</th>
<th>OUTPUT</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.3 Tables</td>
<td><strong>DURABILITY and FIRE RESISTANCE</strong> (as floor slab, page B)</td>
<td>cover = 20mm 1hr. fire resistance</td>
</tr>
<tr>
<td>3.10.1.1</td>
<td><strong>LOADING</strong></td>
<td></td>
</tr>
<tr>
<td>3.10.1.2</td>
<td>Average slab thickness on plan = 250 mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Self-weight = 0.25 x 24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Finishes = 0.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Characteristic dead load = $6.8 \text{kN/m}^2$</td>
<td>$g_k = 6.8 \text{kN/m}^2$</td>
</tr>
<tr>
<td></td>
<td>Characteristic imposed load (page 6) = $4.0 \text{kN/m}^2$</td>
<td>$g_k = 4.0 \text{kN/m}^2$</td>
</tr>
<tr>
<td></td>
<td>Design load = $(1.4 \times 6.8 + 1.6 \times 4.0) 5.0$ = $77.5 \text{kN/width}$</td>
<td>$F = 77.5 \text{kN/width}$</td>
</tr>
<tr>
<td>3.4.3</td>
<td><strong>ULTIMATE B.M.S</strong></td>
<td></td>
</tr>
<tr>
<td>Table 3.5</td>
<td>1st interior support = 0.11 Fl = 0.11 x 77.5 x 5.06 = 43.1 kN/m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Near mid-end span = 0.09 Fl = 35.3 kN/m</td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>REINFORCEMENT</strong></td>
<td></td>
</tr>
<tr>
<td>3.4.4</td>
<td>1st interior support, $\frac{M}{f_{cu} b_d^2} = 0.049$, $\phi = 0.94$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_s = \frac{M}{0.95 f_{cu} b_d^2} = 43.1 \times 10^6$</td>
<td>$d = 149 \text{mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Top T12 @ 125 (905 mm$^2$/m)</td>
</tr>
<tr>
<td></td>
<td>Near mid-end span, $\frac{M}{f_{cu} b_d^2} = 0.040$, $\phi = 0.95$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_s = \frac{M}{0.95 f_{cu} b_d^2} = 35.3 \times 10^6$</td>
<td>Bottom T12 @ 150 (754 mm$^2$/m)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Check for Shear:</strong> $V = \frac{0.6 \times 77.5 \times 10^3}{10^3 \times 149}$ = 0.31 N/mm$^2$ &lt; $V_c$ ok.</td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>DEFLECTION</strong></td>
<td></td>
</tr>
<tr>
<td>3.4.6</td>
<td>Basic span/eff. depth ratio = 26 max.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{M}{b_d^2} = \frac{35.3 \times 10^6}{10^3 \times 149} = 1.59$, $f_s = \frac{2 \times 460 \times 571}{3 \times 154} = 232.2 \text{N/mm}^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Modification factor for service stress 232.2 N/mm$^2$ = 1.37</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Allowable span/eff. depth ratio $= 26 \times 1.37 = 35.62$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Actual $\begin{vmatrix} \frac{5060}{149} \end{vmatrix} = 34.0$</td>
<td>$\phi/d$ ratio ok.</td>
</tr>
<tr>
<td>3.12.11.27</td>
<td><strong>CRACKING</strong> Spacing between bars &lt; 3 x 149 = 4.47 mm.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$h = 175 &lt; 200 \text{mm}$. no further decks required</td>
<td>Spacing &lt; 3d. ok. ok.</td>
</tr>
<tr>
<td>3.12.3.4</td>
<td><strong>TIE PROVISION</strong> East -&gt; West Internal Tie Min. area continuity reinf. = 91 mm$^2$/m (as floor slab, p.8)</td>
<td>Bottom 4T12 (456 mm$^2$)</td>
</tr>
<tr>
<td></td>
<td>Total area req'd for width of staircase = 91 x 3 = 273 mm$^2$</td>
<td>min. 300mm. laps</td>
</tr>
<tr>
<td></td>
<td>Provide 2T12 tie bars each side in adjacent slab.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>LAP, as floor slab, p.8.</td>
<td></td>
</tr>
</tbody>
</table>
Main tension reinforcement. Lap lengths and anchorage bond lengths = 35 x 12 = 420 mm, say 450 mm. Laps are located to facilitate likely construction sequences. Similar for bar marks 12, 13 and 15.

Secondary reinforcement. Minimum area = 0.0013 x 1000 x 175 = 228 mm²/m. Use T10 @ 300 = 262 mm/m.

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.

U-bars provide 50% mid-span reinforcement in both top and bottom at end support = 0.5 x 571 = 286 mm/m. Use T10 @ 150 = 524 mm/m to match spacing of span bars. Laps, say 450 mm.

Optional reinforcement. Minimum area = 228 mm². Similar for bar mark 16.
Rectangular columns

\begin{align*}
\text{fcu} & = 40 \\
\text{fy} & = 460 \\
\frac{d}{h} & = 0.80
\end{align*}
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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