Chapter 5

Design of Corbels and Nibs

5.0 NOTATION

- \( a_c \): Distance from centre of load to nearest face of column for a corbel
- \( a_l \): Distance from free edge of nib to nearest link in beam
- \( A_s \): Area of steel reinforcement in tension to resist bending
- \( A_{sh} \): Area of horizontal steel reinforcement to resist shear in corbel
- \( b \): Width of corbel
- \( d \): Effective depth from bottom of corbel to centre of tensile reinforcement
- \( d_b \): Depth of corbel at edge of loaded area
- \( f_s \): Tensile stress in steel
- \( f_y \): Characteristic yield strength of steel
- \( f_{cu} \): Characteristic cube strength of concrete at 28 days
- \( F_c \): Concrete strut force in compression
- \( F_t \): Steel tensile force
- \( F_{bt} \): Tensile force in reinforcement at start of a bend
- \( h \): Overall depth of corbel
- \( M \): Applied moment on a section
- \( p \): Percentage of tensile reinforcement
- \( r \): Internal radius of a bend in a bar
- \( S_h \): Spacing of horizontal links in a corbel
- \( T \): Tension force applied to corbel along with vertical load
- \( v \): Shear stress in concrete (N/mm²)
- \( v_c \): Design shear stress in concrete (N/mm²)
- \( v_c' \): Modified design shear stress to account for \( a_c \)
- \( V \): Vertical load on corbel
- \( x \): Distance of neutral axis from bottom of corbel
- \( z \): Depth of lever arm
- \( \beta \): Angle of inclination to horizontal of concrete strut in a corbel
- \( \varepsilon_s \): Strain in steel reinforcement
- \( \phi \): Diameter of reinforcing bar or equivalent diameter of a group of bars

5.1 LOAD COMBINATIONS

5.1.1 General rules See Section 2.2.1.
5.1.2 Exceptional loads  
See Section 2.2.4.

5.2 STEP-BY-STEP DESIGN PROCEDURE FOR CORBELS

Step 1  *Determine ultimate loads on the corbel.*
Follow load combination rules of Section 2.2.

Step 2  *Determination of corbel geometry*

Check the following:

1. Bearing stress on concrete under bearing plate \( \leq 0.8f_{cu} \).
2. Distance from end of loaded area to face of corbel should be as shown.
3. Depth at root of corbel should be such that shear stress \( V_{ibd} \) is less than \( 0.8\sqrt{f_{cu}} \) or \( 5 \text{ N/mm}^2 \), whichever is the lesser.
(4) Depth at outer edge of loaded area should be at least half the depth at the root.

(5) If $a_v$ is greater than $d$, the corbel should be designed as a cantilever beam.

**Step 3 Evaluation of internal forces**

![Diagram](image)

SK 5/3 Strut and tie diagram of a reinforced concrete corbel.

**FORCE DIAGRAM**

**STRAIN DIAGRAM**

**STRESS DIAGRAM**

$b =$ WIDTH OF CORBEL

Draw strut and tie diagram as shown and find the following parameters.

\[
v = \frac{V}{bd}
\]

Find $v/f_{cu} \text{ and } a_v/d$.

Find $z/d$ from Fig. 5.1.

Find $z$ and $x = (d - z)/0.45$

Find $F_t = T + V_a/z$

$F_t =$ tension in steel reinforcement

$T =$ applied horizontal load along with $V$

$z =$ depth of lever arm; $x =$ depth of neutral axis

\[
A_s = \frac{F_t}{0.87f_y} = \frac{0.5V}{0.87f_y} + \frac{T}{0.87f_y}
\]

Alternatively,

\[
F_t = F_c \cos \beta + T = \frac{V_{av}}{z} + T
\]

\[
F_c = \left( \frac{0.67f_{cu}}{1.5} \right) b 0.9x \cos \beta = 0.402f_{cu} b x \cos \beta
\]

\[
V = F_c \sin \beta
\]

\[
z = d - 0.45x
\]

By iteration, find $x$ after assuming $x$ in first trial. With final value of $x$, find $z$ and $F_t$. From $F_t$, find $A_s$. 

Step 4 Check shear

\[ p = \frac{100A_s}{bd} \]

Find \( v_c \) from Figs 11.2 to 11.5 and multiply by \( 2d/a_s \) to get \( v'_c \) for corbel.

If \( v < v'_c \), provide nominal shear reinforcement

Nominal reinforcement area = \( 0.5A_s \)

\( A_s \) is obtained in Step 3. Provide nominal links in upper two-thirds of effective depth \( d \).

If \( v > v'_c \), design shear reinforcement

\[ A_{sh} \geq \frac{bS_h(v - v_c)}{0.87f_y} \]

Provide \( A_{sh} \) in upper two-thirds of \( d \) at a spacing of \( S_h \).

Note: Total area of all legs of links in a vertical plane should be more than or equal to \( 0.5A_s \).

Step 5 Minimum tension reinforcement

\[ A_s \geq 0.004bh \]

Step 6 Maximum tension reinforcement

\[ A_s \leq 0.040bh \]

Step 7 Check bearing stress inside bend

The following must be satisfied:

\[ \text{bearing stress} = F_{br}r\phi \leq \frac{2f_{cu}}{1 + 2\left(\frac{\phi}{a_b}\right)} \]

See step 22 of Section 2.3 for notation.
Step 8  Spacing of bars
Minimum clear spacing horizontally = MSA + 5 \geq \text{dia. of bar}
where MSA = maximum size of aggregate.

Minimum clear spacing vertically = \frac{2 \text{MSA}}{3}

Maximum clear spacing of bars in tension \leq \frac{47000}{f_s} \leq 300

f_s = service stress in bar

5.3 STEP-BY-STEP DESIGN PROCEDURE FOR NIBS

Step 1  Determine cover to reinforcement
Determine cover required to reinforcement as per Tables 11.6 and 11.7.

Step 2  Determine ultimate loads on nib
Follow load combination rules of Section 2.2.
**Step 3  Determine nib geometry**

1. Bearing stress under load $\leq 0.4f_{cu}$ for dry bearing
   $\leq 0.6f_{cu}$ for bedded bearing.

2. Find effective bearing length which is the least of:
   (a) bearing length
   (b) one-half of bearing length plus 100 mm
   (c) 600 mm.

3. Find net bearing width $= \frac{\text{design ultimate support reaction}}{\text{effective bearing length}} \times 0.4f_{cu} \geq 40$ mm

4. Find allowance for spalling, as per Tables 5.1 and 5.2.

5. Find allowance for inaccuracies, as per Table 5.3.

6. Nominal bearing width $= (\text{net bearing width}) + (\text{allowances for spalling}) + (\text{allowances for inaccuracies})$

7. Nib projection $= (\text{nominal bearing width}) + 25$ mm
   Allow chamfer minimum 15 mm.

8. Overall depth of nib should be less than 300 mm.

9. Select diameter of reinforcement and find $a_c$ and $d$.

---

![Diagram](image)

**Step 4  Design of nib**

\[ M = V a_c \]

\[ V = \text{ultimate load per metre} \]

\[ K = \frac{M}{bd^2f_{cu}} \]

\[ b = 1 \text{ metre} \]
\[ z = d \left[ 0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)}\right] \leq 0.95d \]

\[ A_s = \frac{M}{0.87f_{zc}} \text{ per metre} \]

**Step 5** *Determine minimum reinforcement*
Minimum reinforcement = 0.0013bh

**Step 6** *Maximum spacing of bars*
Maximum allowable spacing = 3 × (effective depth) + (diameter of bar) ≤ 750 mm

Clear spacing \( \leq \frac{47000}{f_s} \leq 300 \text{ mm} \)

where \( f_s \) = service stress.

**Step 7** *Check shear*
\[ V = \text{ ultimate load per metre} \]
\[ \nu = \frac{V}{bd} \]
\[ b = 1 \text{ metre} \]

\[ P = \frac{100A_s}{bd} \]

Find \( \nu_c \) from Figs 11.2 to 11.5.

Find \( \nu_c' = \left(\frac{2d}{a_c}\right)^2 \nu_c \)

Check that \( \nu = \nu_c' \)

If not, increase depth of nib.

**Note:** If tensile reinforcement found in Step 3 is kept straight and exposed at end, shear stress \( \nu \) should be less than \( \nu_c' / 2 \).

**Step 8** *Extra vertical reinforcement in beam*
\[ A_{sv} = \frac{V}{0.87f_y} \text{ per metre length of beam} \]

**Step 9** *Isolated loads on continuous nib*
To find effective width of load dispersal, assume a 45° angle of line of failure crack as shown.

\( l_e \) = effective width for isolated load on continuous nibs
5.4 WORKED EXAMPLES

Example 5.1 Design of a corbel

Step 1 Determine ultimate loads on the corbel
Ultimate vertical load = \( V = 800 \text{kN} \)
Ultimate horizontal load = \( T = 80 \text{kN} \)
(Ignore small eccentricity of horizontal load from tension steel.)
Line of action of load at 400 mm from face of column.
Size of column = 600 mm × 400 mm
Corbel about the major axis of column.
Width of corbel = 400 mm

**Step 2**  
**Determination of corbel geometry**
- $f_{cu} = 40 \text{ N/mm}^2$
- $f_y = 460 \text{ N/mm}^2$
- Minimum cover to reinforcement = 30 mm
- Assumed diameter of main reinforcement = 32 mm
- Assumed diameter of horizontal links = 10 mm
- Bearing plate used.
- Maximum bearing stress = $0.8 f_{cu} = 32 \text{ N/mm}^2$
- Length of bearing plate = 300 mm

Minimum bearing width = \[
\frac{V}{32 \times 300} = \frac{800 \times 10^3}{32 \times 300} = 83 \text{ mm}
\]

Actual width of bearing plate = 100 mm = $l_w > 83 \text{ mm}$  OK

\[l = \text{length of corbel} = a_v + \frac{1}{2} l_w + \text{length of bend of bar + min. cover +}
\]
\[
\text{dia. of link + min. cover}
\]
\[= 400 + 50 + 5 \times 32 + 30 + 10 + 30
\]
\[= 680 \text{ mm} \quad \text{say 700 mm}
\]

Use $h = 750 \text{ mm}$ at column face.

\[d = 750 - 30 - 16 = 704 \text{ mm}
\]

Maximum allowable shear stress at column face = 5 N/mm²

\[d > \frac{V}{5b} = \frac{800 \times 10^3}{5 \times 400} = 400 \text{ mm}
\]

\[
v = \frac{V}{bd}
\]
\[= \frac{800 \times 10^3}{400 \times 704} = 2.84 \text{ N/mm}^2 < 0.8 f_{cu} = 5.05 \text{ N/mm}^2
\]

**Step 3**  
**Evaluation of forces**

**First trial**
From strut and tie diagram (Step 3 in Section 5.2),
\[ F_t = F_c \cos \beta + T = \frac{V a_s}{z} + T \]

\[ F_c = \left( \frac{0.67 f_{cu}}{1.5} \right) b \ 0.9x \cos \beta = 0.402 f_{cu} b x \cos \beta \]

\[ V = F_c \sin \beta \]

\[ z = d - 0.45x \]

Assume \( x = 0.4d = 282 \text{ mm} \), say.

\[ z = d - 0.45x \]
\[ = 704 - (0.45 \times 282) \]
\[ = 577 \text{ mm} \]

\[ \cot \beta = \frac{a_s}{z} = \frac{400}{577} = 0.6932 \]

\[ \sin \beta = 0.8218 \quad \cos \beta = 0.5697 \]

\[ F_c = \frac{V}{\sin \beta} = 973.5 \text{ kN} \]

\[ x = \frac{F_c}{0.402 f_{cu} b \cos \beta} \]
\[ = \frac{973.5 \times 10^3}{0.402 \times 40 \times 400 \times 0.5697} \]
\[ = 265.7 \text{ mm} \]

Second trial

\[ x = 265 \text{ mm} \]

\[ z = 584.7 \text{ mm} \]

\[ \cot \beta = 0.6841 \]

\[ \sin \beta = 0.8254 \]

\[ \cos \beta = 0.5646 \]

\[ F_c = 969.2 \text{ kN} \]

\[ x = 266.9 \text{ mm} \quad \text{OK} \]

Final \( z = 585 \text{ mm} \)

\[ F_t = \frac{V a_s}{z} + T \]
\[ = \left( \frac{800 \times 10^3 \times 400}{585} \right) + 80 \times 10^3 = 627 \times 10^3 \text{ N} \]

\[ e_s = 0.0035 \times \left( \frac{704 - 265}{265} \right) \]
\[ = 5.798 \times 10^{-3} > 0.002 \]
So the steel will be at the yield stress level

\[ f_y = 460 \text{ N/mm}^2 \]

\[ A_s = \frac{F_t}{0.87f_y} \geq \left( \frac{0.5V}{0.87f_y} \right) + \left( \frac{T}{0.87f_y} \right) = 1200 \text{ mm}^2 \]

\[ = \frac{627 \times 10^3}{0.87 \times 460} \]

\[ = 1567 \text{ mm}^2 > 1200 \text{ mm}^2 \quad \text{OK} \]

Use 2 no. 32 dia. bars as main tension reinforcement (1608 mm²).

Alternatively by use of the chart in Fig. 5.1,

\[ \frac{v}{f_{cu}} = \frac{2.84}{40} = 0.071 \]

\[ a_v = \frac{400}{704} = 0.568 \]

From Fig. 5.1,

\[ \frac{z}{d} = 0.83 \]

\[ z = 704 \times 0.83 = 584 \text{ mm} \]

**Note:** The chart gives the same \( z \) as is obtained by iteration. Having found \( z \) from the chart, find \( F_t \) and \( A_s \).

**Step 4 Check shear**

\[ p = \frac{100A_s}{bd} \]

\[ = \frac{100 \times 1608}{400 \times 704} \]

\[ = 0.57 \]

From Fig. 11.5,

\[ v_c = 0.608 \text{ N/mm}^2 \]

\[ v'_c = \left( \frac{2d}{a_v} \right) v_c = \frac{2 \times 704 \times 0.608}{400} = 2.14 \text{ N/mm}^2 < 2.84 \text{ N/mm}^2 \]

Shear reinforcement is required.

Horizontal links are provided. Assume \( S_h = 200 \text{ mm} \).

\[ A_{sh} \geq \frac{bS_h(v - v'_c)}{0.87f_y} = \frac{400 \times 200 \times (2.84 - 2.14)}{0.87 \times 460} = 140 \text{ mm}^2 \]

Required: 2-legged 10 mm diameter links at 200 centres for the upper two-thirds of \( d \).

\[ \frac{2}{3}d = \frac{2}{3} \times 704 = 470 \text{ mm} \]
Required: 3 sets of links of 10 mm diameter at 200 mm centres.
Total area of legs = 471 mm$^2 < 0.5 \times 1567 = 783.5$ mm$^2$
Main tension steel required = 1567 mm$^2$
Use 5 sets of links 10 mm diameter at 100 mm centres (785 mm$^2$).

*Step 5 Minimum tension reinforcement*
\[ A_s > 0.004bh = 1200 \text{ mm}^2 \] satisfied

*Step 6 Maximum tension reinforcement*
\[ A_s < 0.040bh = 12000 \text{ mm}^2 \]
Not exceeded.

*Step 7 Check bearing stress inside bend*

**SK 5/12** Bearing stress inside bend.
\[ F_{bt} = \left( \frac{F_t}{\text{no. of bars}} \right) \left( \frac{A_{s \text{ req.}}}{A_{s \text{ prov.}}} \right) \]
\[ = \frac{627}{2} \times \frac{1567}{1608} \]
\[ = 305.5 \text{ kN} \]

Ultimate anchorage bond stress
\[ f_{bu} = 0.5 \sqrt{f_{cu}} \] (for Type 2 deformed bar as obtained from Table 3.28 of BS8110: Part 1)
\[ = 0.5 \sqrt{40} = 3.16 \text{ N/mm}^2 \]

Anchorage bond length required
\[ = \frac{F_{bt}}{\pi \phi f_{bu}} \]
\[ = \frac{305.5 \times 10^3}{\pi \times 32 \times 3.16} \]
\[ = 962 \text{ mm} \]

In the column, the straight length of bar before start of bend is taken as approximately equal to 350 mm which is say one-third of the required anchorage length. Hence

Tension in bar at start of bend
\[ = \frac{2}{3} F_{bt} \]
\[ = \frac{2}{3} \times 305.5 = 203.7 \text{ kN} \]

\( r = \) internal radius of bend
\[ = 4 \times 32 \text{ (minimum)} \]
\[ = 128 \text{ mm standard} \]

\( \phi = 32 \text{ mm} \)
\( a_b = \) cover + bar diameter for corner bar
\[ = 72 + 32 = 104 \text{ mm} \]

Centre-to-centre distance of bars = 224 mm > 104 mm
\[ \therefore a_b = 104 \text{ mm} \]

\[ F_{bt} = \frac{203.7 \times 10^3}{128 \times 32} = 49.73 \text{ N/mm}^2 \]

\[ \frac{2f_{cu}}{1 + \left( \frac{2\phi}{a_b} \right)} = \frac{2 \times 40}{1 + 2 \left( \frac{32}{104} \right)} = 49.52 \text{ N/mm}^2 < 49.73 \text{ N/mm}^2 \]

Standard radius bend will be adequate.
Calculation of anchorage bond length:
Anchorage value standard bend = 12 \times 32 = 384 mm (includes 4 diameter straight)

Straight before bend = 350 mm
Bar should project vertically into column after standard bend by minimum of
962 - 384 - 350 + (4 x 32) = 356 mm

Step 8 Spacing of bars

Minimum horizontal spacing = 20 + 5 = 25 mm

Maximum clear spacing of bars in tension < \frac{47000}{f_s} < 300

\( f_s = \text{service stress} \) (from crack width calculations in Step 9)

\( = 226.4 \text{ N/mm}^2 \)

\[ \frac{47000}{f_s} = \frac{47000}{226.4} = 208 \text{ mm} \]

Actual clear spacing = 224 - 32 = 192 mm < 208 mm  OK

Clear distance between the corner of corbel and the nearest tension bar should not be greater than 80 mm as per clause 3.12.11.2.5 of BS8110: Part 1: 1985. Actual clear distance is 72 mm.

Note: No crack width calculation is required if maximum spacing of bars in tension does not exceed the recommendations of clause 3.12.11.2 of BS8110: Part 1.

Step 9 Crack width calculations

Note: This step is optional and is included to show the method of calculation of crack width for a corbel.

END ELEVATION OF CORBEL

SK 5/13 Crack width calculations.
Service horizontal load = 50 kN

Service vertical load = 500 kN

Moment at face of column = \(500 \times a_s = 200\) kNm

See Section 1.13 and assume \(A_s' = 0\)

\[
x = \frac{m A_s}{b} \left[ \left( 1 + \frac{2bd}{A_{km}} \right)^{\frac{1}{3}} - 1 \right]
\]

\[
= \frac{10 \times 1608}{400} \left[ \left( 1 + \frac{2 \times 400 \times 704}{1608 \times 10} \right)^{\frac{1}{3}} - 1 \right]
\]

\[
= 201\text{ mm}
\]

\[
z = d - \frac{x}{3}
\]

\[
= 704 - \frac{201}{3}
\]

\[
= 637\text{ mm}
\]

\[
f_{sb} = \frac{M}{A_s z}
\]

\[
= \frac{200 \times 10^6}{1608 \times 637}
\]

\[
= 195.3\text{ N/mm}^2 \text{ due to flexure}
\]

\[
f_{sh} = \frac{50 \times 10^3}{1608} = 31.1\text{ N/mm}^2 \text{ due to horizontal load}
\]

\[
f_s = f_{sb} + f_{sh}
\]

\[
= 195.3 + 31.1
\]

\[
= 226.4\text{ N/mm}^2
\]

\[
\varepsilon_s = \frac{f_s}{E_s} = \frac{226.4}{200 \times 10^3} = 1.132 \times 10^{-3}
\]

\[
\varepsilon_h = \left( \frac{h - x}{d - x} \right) \varepsilon_s
\]

\[
= \left( \frac{750 - 201}{704 - 201} \right) \times 1.132 \times 10^{-3}
\]

\[
= 1.235 \times 10^{-3}
\]

\[
\varepsilon_{mh} = \varepsilon_h - \frac{b(h - x)^2}{3E_s A_s (d - x)}
\]

\[
= 1.235 \times 10^{-3} - \frac{400(750 - 201)^2}{3 \times 200 \times 10^3 \times 1608 \times (704 - 201)}
\]

\[
= 0.9866 \times 10^{-3}
\]

\[
a_{cl} = \sqrt{(88^2 + 46^2)} - 16 = 83.3\text{ mm}
\]
\[ a_{c2} = \sqrt{(112^2 + 46^2)} - 16 = 105\,\text{mm} \]
\[ a_{cr} = 105\,\text{mm} \]

\[ W_{cr} = \frac{3a_{cr} \varepsilon_m}{1 + 2\left(\frac{a_{cr} - \varepsilon_{min}}{h - x}\right)} \]
\[ = \frac{3 \times 105 \times 0.9866 \times 10^{-3}}{1 + 2\left(\frac{105 - 30}{750 - 201}\right)} \]
\[ = 0.244\,\text{mm} < 0.3\,\text{mm} \]

Crack width criterion is satisfied.

**Example 5.2** Design of concrete nib

![Diagram of concrete nib](image)

Precast concrete slab

Reinforced concrete in-situ floor beams with nibs to carry precast floor units.

Clear gap between beams = 4.5 m
Width of floor units = 400 mm
Depth of floor units = 100 mm
False floor + finish on units = 2.5 kN/m²
Imposed load on floor = 5.0 kN/m²
Grade of concrete for beam = C40
Assume dry bearing.

**Step 1** Determine cover to reinforcement
Exposure = mild
Fire resistance = 1 hour
Grade of concrete = C40
Maximum size of aggregate = 20 mm
Minimum thickness of floor = 95 mm
Nominal cover = 20 mm
Step 2  **Determine loading**
Self-weight of precast unit = 2.5 kN/m²
False floor + finish = 2.5 kN/m²
Total dead load = 5 kN/m²
Imposed load = 5 kN/m²
Ultimate load = 1.4 × 5 + 1.6 × 5 = 15 kN/m²
Reaction at either end of precast floor unit (400 mm) = 4.5 × 15 × 0.5 × 0.4
= 13.5 kN

Step 3  **Determine nib geometry**
Allowable bearing stress = 0.4f_{cu} = 0.4 × 40 = 16 N/mm²
Effective bearing length is the least of:
(a) bearing length = 400 mm
(b) one-half bearing length + 100 = 300 mm
(c) 600 mm.
Effective bearing length = 300 mm

Net bearing width = \frac{\text{ultimate support reaction}}{(\text{effective bearing length}) × 0.4f_{cu}} ≥ 40
= \frac{13.5 × 10³}{300 × 16} = 2.8 mm

Net bearing width = 40 mm
Allowance for spalling (from Tables 5.1 and 5.2) = 20 + 0 = 20 mm
Allowance for inaccuracies (from Table 5.3) = 25 mm
Nominal bearing width = 40 + 20 + 25 = 85 mm
Nib projection = 85 mm + 15 mm (chamfer) + 10 mm (clearance) = 110 mm
Nominal length of precast units = 4.5 m - 2 × 10 mm (clearance)
= 4.48 m

Minimum depth of nib = 2 × (minimum cover) + 8 × (diameter of bar)
= 2 × 20 + 8 × 8 = 104 mm < 300 mm
Minimum depth of nib 105 mm, say.

**Note:** The depth of the nib can be reduced if 6 mm diameter mild steel bars are used or welded anchor bars are used at straight ends of flexural bars.

Step 4  **Design of nib**
\[ a_c = 110 - 15 \text{ (chamfer)} + 20 \text{ (cover)} + 5 \text{ (half dia. of link)} \]
= 120 mm
\[ d = 105 - 20 - 4 = 81 \text{ mm} \]
\[ M = \text{bending moment per metre} \\
= (\text{load per metre run}) \times a_v \\
= 43.5 \times 0.5 \times 15 \times 0.12 \\
= 4.05 \text{kNm/m} \\
\]

\[ K = \frac{M}{f_{\text{cd}} d^2} \\
= \frac{4.05 \times 10^6}{40 \times 1000 \times 81^2} \\
= 0.0154 \\
\]

\[ z = d \left[ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right] \leq 0.95d \\
= 0.95d = 77 \text{mm} \\
\]

\[ A_s = \frac{M}{0.87 f_{\text{yz}}} = \frac{4.05 \times 10^6}{0.87 \times 460 \times 77} = 131 \text{mm}^2/\text{m} \\
\]

**Step 5** Determine minimum reinforcement
Minimum reinforcement = 0.0013bh
= 0.0013 \times 1000 \times 105
= 137 \text{mm}^2/\text{m}

**Step 6** Maximum spacing of bars
Maximum spacing = 3 \times \text{effective depth} + \text{bar dia.}
= 3 \times 81 + 8
= 251 \text{mm centres}

Use 8 mm dia. bars at 250 centres (201 mm²/m).
(See Example 2.3, Step 25 for refinement.)

\[ \Phi 10 \text{ Link in beam} \\
\]

SK 5/15 Typical reinforcement in nib.
Step 7  Check shear

\[ V = \text{ultimate load per metre length} \]
\[ = 4.5 \times 0.5 \times 15 \]
\[ = 33.75 \text{kN/m} \]
\[ v = \frac{V}{bd} = \frac{33.75 \times 10^3}{1000 \times 81} \]
\[ = 0.42 \text{N/mm}^2 \]
\[ p = \frac{100A_s}{bd} \]
\[ = \frac{100 \times 201}{1000 \times 81} \]
\[ = 0.25 \]

From Fig. 11.5,
\[ \nu_c = 0.62 \text{N/mm}^2 \]
\[ \nu'_c = \frac{\nu_c 2d}{a_v} \]
\[ = \frac{0.62 \times 2 \times 81}{120} \]
\[ = 0.84 \text{N/mm}^2 > 0.42 \text{N/mm}^2 \]

Step 8  Extra vertical reinforcement in beam

In addition to links, an area of reinforcement is required in the beam to carry the load from the nib.

\[ A_{sv} = \frac{V}{0.87f_y} \]
\[ = \frac{33.75 \times 10^3}{0.87 \times 460} \]
\[ = 84 \text{mm}^2/\text{m} \]
Fig. 5.1 Chart for determining $z/d$. 

VALUES OF $z/d$

$\frac{v}{f'_{cd}}$

$\frac{a_y}{d}$
Table 5.1 Allowance for effects of spalling at supports.

<table>
<thead>
<tr>
<th>Material of support</th>
<th>Distance $y$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>0</td>
</tr>
<tr>
<td>Concrete Grade 30 or over</td>
<td>15</td>
</tr>
<tr>
<td>Brickwork or masonry</td>
<td>25</td>
</tr>
<tr>
<td>Concrete below Grade 30</td>
<td>25</td>
</tr>
<tr>
<td>Reinforced concrete nib less than 300 mm deep</td>
<td>Nominal cover to reinforcement</td>
</tr>
<tr>
<td>Reinforced concrete nib less than 300 mm deep with vertical loop reinforcement exceeding 12 mm in diameter</td>
<td>Nominal cover plus inner radius of bend</td>
</tr>
</tbody>
</table>

Table 5.2 Allowance for effects of spalling at supported members.

<table>
<thead>
<tr>
<th>Reinforcement at bearing of supported member</th>
<th>Distance $x$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight bars, horizontal loop or vertical loop reinforcement not exceeding 12 mm diameter</td>
<td>10 or end cover, whichever is greater</td>
</tr>
<tr>
<td>Tendons or straight bars exposed at end of member</td>
<td>0</td>
</tr>
<tr>
<td>Vertical loop reinforcement of bar diameter exceeding 12 mm</td>
<td>End cover plus inner radius of bend of bar</td>
</tr>
</tbody>
</table>

Table 5.3 Allowance for construction inaccuracies.

<table>
<thead>
<tr>
<th>Material of support</th>
<th>Construction inaccuracy (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel or precast concrete support</td>
<td>15 or $3L$, whichever is greater</td>
</tr>
<tr>
<td>Masonry supports</td>
<td>20 or $4L$, whichever is greater</td>
</tr>
<tr>
<td>In-situ concrete supports</td>
<td>25 or $5L$, whichever is greater</td>
</tr>
</tbody>
</table>