Chapter 5
Design of Corbels and Nibs

5.0 NOTATION

a_v	Distance from centre of load to nearest face of column for a corbel
a_v	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/mm^2)
v_c	Design shear stress in concrete (N/mm^2)
v'_c	Modified design shear stress to account for a_v
V	Vertical load on corbel
x	Distance of neutral axis from bottom of corbel
z	Depth of lever arm
β	Angle of inclination to horizontal of concrete strut in a corbel
ε_s	Strain in steel reinforcement
ϕ	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.

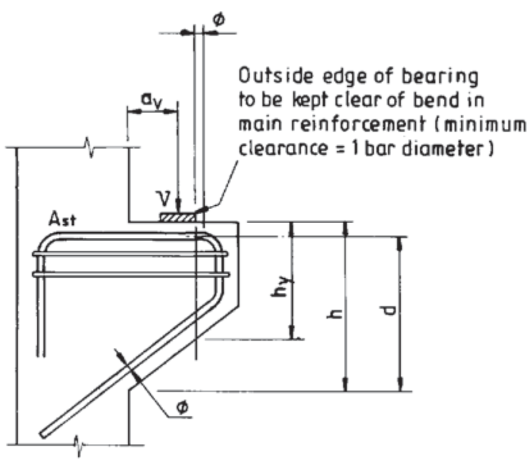
5.1.2 Exceptional loads

See Section 2.2.4.

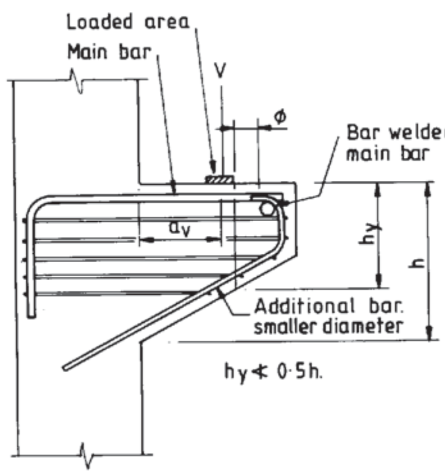
5.2 STEP-BY-STEP DESIGN PROCEDURE FOR CORBELS

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

Step 2 ***Determination of corbel geometry***



SK 5/1 Corbel geometry.



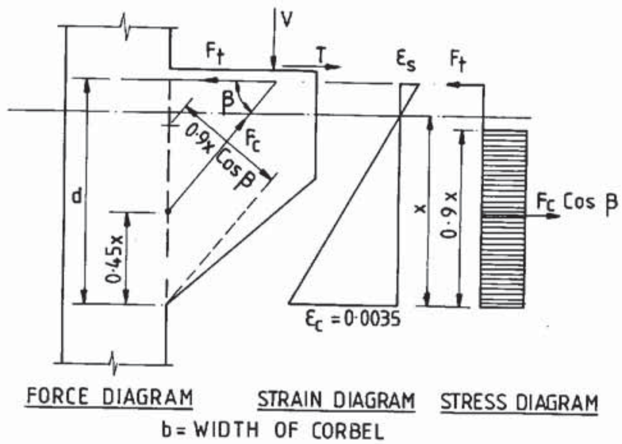
SK 5/2 Alternative corbel geometry.

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/bd is less than $0.8\sqrt{f_{cu}}$ or $5\,N/mm^2$, 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



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

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

$$v = \frac{V}{bd}$$

Find v/f_{cu} and a_v/d.
Find z/d from Fig. 5.1.
Find z and $x = (d - z)/0.45$
Find $F_t = T + Va_v/z$
F_t = tension in steel reinforcement
T = applied horizontal load along with V
z = depth of lever arm; x = depth of neutral axis

$$A_s = \frac{F_t}{0.87f_y} \geq \frac{0.5V}{0.87f_y} + \frac{T}{0.87f_y}$$

Alternatively,

$$F_t = F_c \cos\beta + T = \frac{Va_v}{z} + T$$

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

$$V = F_c \sin\beta$$

$$z = d - 0.45x$$

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***

$$p = \frac{100A_s}{bd}$$

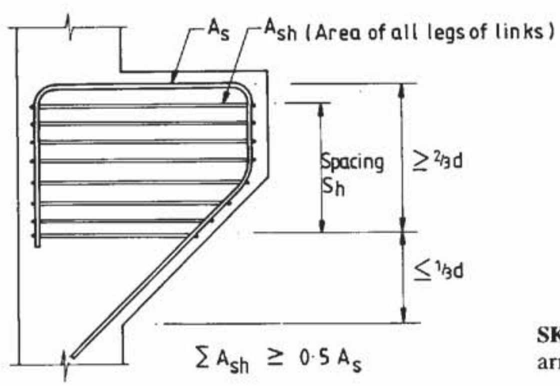
Find v_c from Figs 11.2 to 11.5 and multiply by $2d/a_v$ to get v_c' for corbel.

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



SK 5/4 Typical reinforcement arrangement in a corbel.

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

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 $0.5A_s$.

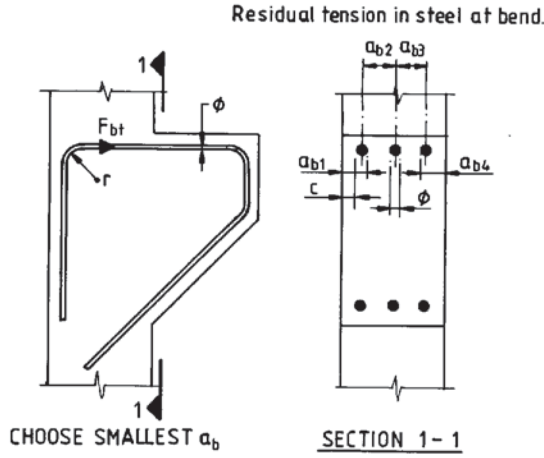
Step 5 ***Minimum tension reinforcement***
$A_s \geq 0.004bh$

Step 6 ***Maximum tension reinforcement***
$A_s \leq 0.040bh$

Step 7 ***Check bearing stress inside bend***
The following must be satisfied:

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

See step 22 of Section 2.3 for notation



SK 5/5 Bearing stress inside bend.

Step 8 ***Spacing of bars***

Minimum clear spacing horizontally = MSA + 5 ≥ dia. of bar

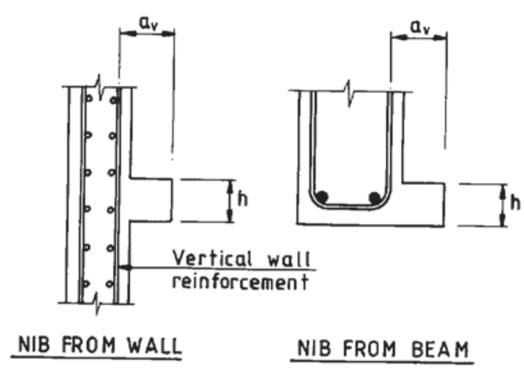
where MSA = maximum size of aggregate.

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

$$\text{Maximum clear spacing of bars in tension} \leq \frac{47\,000}{f_s} \leq 300$$

f_s = service stress in bar

5.3 STEP-BY-STEP DESIGN PROCEDURE FOR NIBS



SK 5/6 Typical arrangement of 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 $= \dfrac{\text{design ultimate support reaction}}{(\text{effective bearing length}) \times 0.4f_{cu}} \geq 40\,\text{mm}$

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

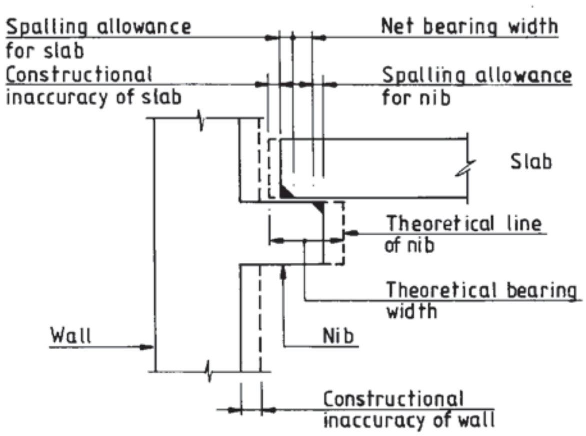
(5) Find allowance for inaccuracies, as per Table 5.3.

(6) Nominal bearing width = (net bearing width) + (allowances for spalling) + (allowances for inaccuracies)

(7) Nib projection = (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_v and d.



SK 5/7 Typical calculation for net bearing width of nib.

Step 4 Design of nib

$M = Va_v$

V = ultimate load per metre

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

b = 1 metre

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

$$A_s = \frac{M}{0.87f_y z} \text{ per metre}$$

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

$$\text{Clear spacing} \leq \frac{47\,000}{f_s} \leq 300\,\text{mm}$$

where f_s = service stress.

Step 7 Check shear
V = ultimate load per metre

$$v = \frac{V}{bd}$$

b = 1 metre

$$p = \frac{100A_s}{bd}$$

Find v_c from Figs 11.2 to 11.5.

$$\text{Find } v'_c = \left(\frac{2d}{a_v}\right)v_c$$

Check that $v \leq v'_c$

If not, increase depth of nib.

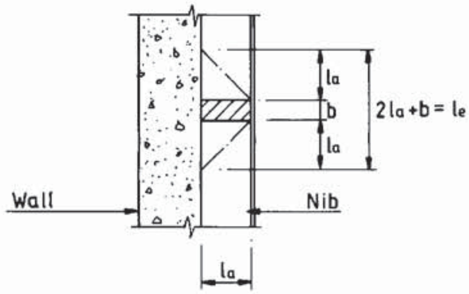
Note: If tensile reinforcement found in Step 3 is kept straight and exposed at end, shear stress v should be less than $v'_c/2$.

Step 8 Extra vertical reinforcement in beam

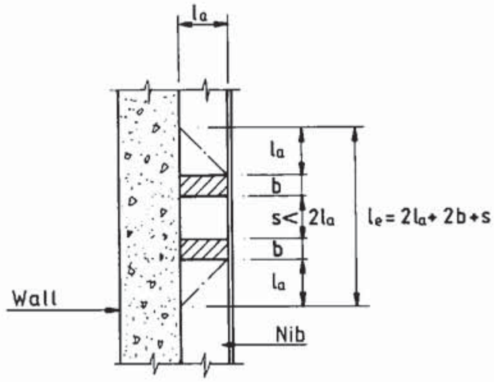
$$A_{sv} = \frac{V}{0.87f_y} \quad \text{per metre length of beam}$$

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



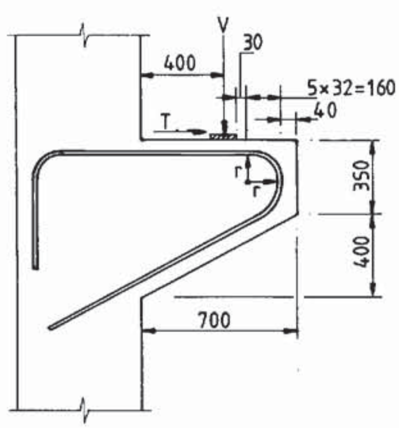
SK 5/8 Plan of wall and nib showing effective length of nib for a line load of width b.



SK 5/9 Plan of wall and nib showing effective length with multiple line loads.

5.4 WORKED EXAMPLES

Example 5.1 Design of a corbel



SK 5/10 Elevation of 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 = $0.8f_{cu} = 32\,\text{N/mm}^2$

Length of bearing plate = 300 mm

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

Actual width of bearing plate = 100 mm = $l_w > 83$ mm OK

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

Use $h = 750$ mm at column face.

$d = 750 - 30 - 16 = 704$ mm

Maximum allowable shear stress at column face = $5\,\text{N/mm}^2$

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

$$\begin{aligned} v &= \frac{V}{bd} \\ &= \frac{800 \times 10^3}{400 \times 704} = 2.84\,\text{N/mm}^2 < 0.8\sqrt{f_{cu}} = 5.05\,\text{N/mm}^2 \end{aligned}$$

Step 3 Evaluation of forces

First trial

From strut and tie diagram (Step 3 in Section 5.2),

$$F_t = F_c \cos\beta + T = \frac{Va_v}{z} + T$$

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

$$V = F_c \sin\beta$$

$$z = d - 0.45x$$

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

$$\begin{aligned} z &= d - 0.45x \\ &= 704 - (0.45 \times 282) \\ &= 577\,\text{mm} \end{aligned}$$

$$\cot\beta = \frac{a_v}{z} = \frac{400}{577} = 0.6932$$

$$\sin\beta = 0.8218 \qquad \cos\beta = 0.5697$$

$$F_c = \frac{V}{\sin\beta} = 973.5\,\text{kN}$$

$$\begin{aligned} x &= \frac{F_c}{0.402 f_{cu} b \cos\beta} \\ &= \frac{973.5 \times 10^3}{0.402 \times 40 \times 400 \times 0.5697} \\ &= 265.7\,\text{mm} \end{aligned}$$

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 $z = 585\,\text{mm}$

$$\begin{aligned} F_t &= \frac{Va_v}{z} + T \\ &= \left(\frac{800 \times 10^3 \times 400}{585}\right) + 80 \times 10^3 = 627 \times 10^3\,\text{N} \end{aligned}$$

$$\begin{aligned} \varepsilon_s &= 0.0035 \times \left(\frac{704 - 265}{265}\right) \\ &= 5.798 \times 10^{-3} > 0.002 \end{aligned}$$

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 ($1608\,\text{mm}^2$).
Alternatively by use of the chart in Fig. 5.1,

$$\frac{v}{f_{cu}} = \frac{2.84}{40} = 0.071$$

$$\frac{a_v}{d} = \frac{400}{704} = 0.568$$

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.

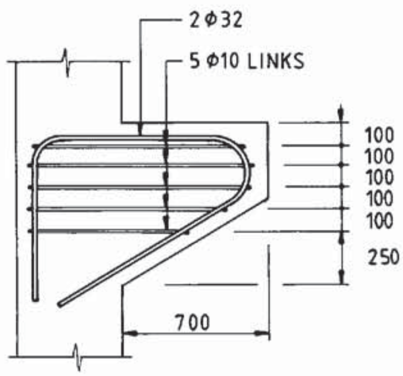
$$\frac{2}{3}d = \frac{2}{3} \times 704 = 470\,\text{mm}$$

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 ($785\,mm^2$).



SK 5/11 Elevation of designed corbel.

Step 5 ***Minimum tension reinforcement***

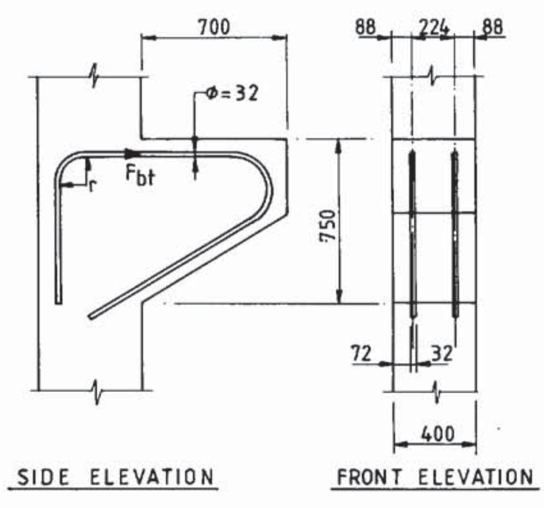
$A_s > 0.004bh = 1200\,mm^2$ satisfied

Step 6 ***Maximum tension reinforcement***

$A_s < 0.040bh = 12\,000\,mm^2$

Not exceeded.

Step 7 ***Check bearing stress inside bend***



SK 5/12 Bearing stress inside bend.

$$F_{bt} = \left(\frac{F_t}{\text{no. of bars}}\right)\left(\frac{A_{s\ req}}{A_{s\ prov}}\right)$$
$$= \frac{627}{2} \times \frac{1567}{1608}$$
$$= 305.5\,\text{kN}$$

Ultimate anchorage bond stress

$f_{bu} = 0.5\sqrt{f_{cu}}$ (for Type 2 deformed bar as obtained from Table 3.28 of BS 8110: Part 1)
$= 0.5\sqrt{40} = 3.16\,\text{N/mm}^2$

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

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

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

r = internal radius of bend
$= 4 \times 32$ (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

$\therefore\ a_b = 104\,\text{mm}$

$$\frac{F_{bt}}{r\phi} = \frac{203.7 \times 10^3}{128 \times 32} = 49.73\,\text{N/mm}^2$$

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

Standard radius bend will be adequate.
Calculation of anchorage bond length:

Anchorage value standard bend = 12 × 32 = 384 mm (includes 4 diameter straight)

Straight before bend = 350 mm

Bar should project vertically into column after standard bend by minimum of

$962 - 384 - 350 + (4 \times 32) = 356\,\text{mm}$

Step 8 Spacing of bars

Minimum horizontal spacing $= 20 + 5 = 25\,\text{mm}$

Maximum clear spacing of bars in tension $< \dfrac{47\,000}{f_s} < 300$

f_s = service stress (from crack width calculations in Step 9)
$= 226.4\,\text{N/mm}^2$

$$\frac{47\,000}{f_s} = \frac{47\,000}{226.4} = 208\,\text{mm}$$

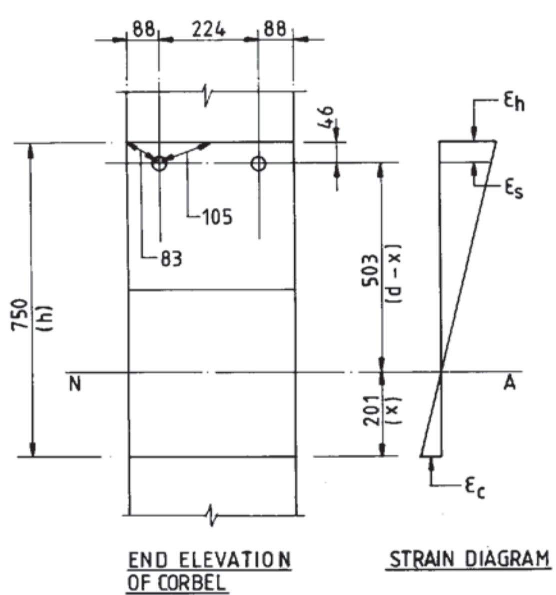
Actual clear spacing $= 224 - 32 = 192\,\text{mm} < 208\,\text{mm}$ OK

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 BS 8110: 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 BS 8110: 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.



SK 5/13 Crack width calculations.

Service horizontal load = 50 kN

Service vertical load = 500 kN

Moment at face of column = $500 \times a_v = 200$ kNm

See Section 1.13 and assume $A_s' = 0$

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

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

$$= 201\,\text{mm}$$

$$z = d - \frac{x}{3}$$

$$= 704 - \frac{201}{3}$$

$$= 637\,\text{mm}$$

$$f_{sb} = \frac{M}{A_s z}$$

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

$$= 195.3\,\text{N/mm}^2 \quad \text{due to flexure}$$

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

$$f_s = f_{sb} + f_{sh}$$
$$= 195.3 + 31.1$$
$$= 226.4\,\text{N/mm}^2$$

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

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

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

$$= 1.235 \times 10^{-3}$$

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

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

$$= 0.9866 \times 10^{-3}$$

$$a_{c1} = \sqrt{(88^2 + 46^2)} - 16 = 83.3\,\text{mm}$$

$$a_{c2} = \sqrt{(112^2 + 46^2)} - 16 = 105\,\text{mm}$$

$$a_{cr} = 105\,\text{mm}$$

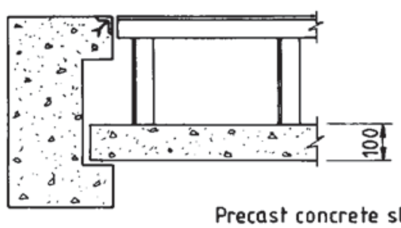
$$W_{cr} = \frac{3a_{cr}\,\varepsilon_m}{1 + 2\left(\dfrac{a_{cr} - c_{min}}{h - x}\right)}$$

$$= \frac{3 \times 105 \times 0.9866 \times 10^{-3}}{1 + 2\left(\dfrac{105 - 30}{750 - 201}\right)}$$

$$= 0.244\,\text{mm} < 0.3\,\text{mm}$$

Crack width criterion is satisfied.

Example 5.2 Design of concrete nib



SK 5/14 General arrangement of nib.

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/m^2

Imposed load on floor = 5.0 kN/m^2

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 = $2.5\,kN/m^2$

False floor + finish = $2.5\,kN/m^2$

Total dead load = $5\,kN/m^2$

Imposed load = $5\,kN/m^2$

Ultimate load = $1.4 \times 5 + 1.6 \times 5 = 15\,kN/m^2$

Reaction at either end of precast floor unit (400 mm) $= 4.5 \times 15 \times 0.5 \times 0.4$
$= 13.5\,kN$

Step 3 Determine nib geometry

Allowable bearing stress $= 0.4f_{cu} = 0.4 \times 40 = 16\,N/mm^2$

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

$$\text{Net bearing width} = \frac{\text{ultimate support reaction}}{(\text{effective bearing length}) \times 0.4f_{cu}} \geq 40$$
$$= \frac{13.5 \times 10^3}{300 \times 16} = 2.8\,mm$$

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 $= 4.5\,m - 2 \times 10\,mm$ (clearance)
$= 4.48\,m$

Minimum depth of nib $= 2 \times$ (minimum cover) $+ 8 \times$ (diameter of bar)
$= 2 \times 20 + 8 \times 8 = 104\,mm < 300\,mm$

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

$a_v = 110 - 15$ (chamfer) + 20 (cover) + 5 (half dia. of link)
$= 120\,mm$

$d = 105 - 20 - 4 = 81\,mm$

$$M = \text{bending moment per metre}$$
$$= (\text{load per metre run}) \times a_v$$
$$= 43.5 \times 0.5 \times 15 \times 0.12$$
$$= 4.05\,\text{kNm/m}$$

$$K = \frac{M}{f_{cu}bd^2}$$
$$= \frac{4.05 \times 10^6}{40 \times 1000 \times 81^2}$$
$$= 0.0154$$

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

$$A_s = \frac{M}{0.87f_yz} = \frac{4.05 \times 10^6}{0.87 \times 460 \times 77} = 131\,\text{mm}^2/\text{m}$$

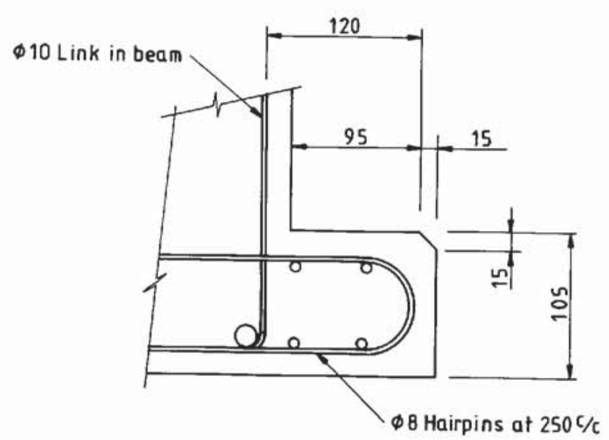
Step 5 Determine minimum reinforcement

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 × effective depth + bar dia.
$= 3 \times 81 + 8$
$= 251$ mm centres

Use 8 mm dia. bars at 250 centres ($201\,\text{mm}^2/\text{m}$).
(See Example 2.3, Step 25 for refinement.)



SK 5/15 Typical reinforcement in nib.

Step 7 Check shear

V = 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,

$$v_c = 0.62\,\text{N/mm}^2$$

$$v_c' = \frac{v_c 2d}{a_v}$$

$$= \frac{0.62 \times 2 \times 81}{120}$$

$$= 0.84\,\text{N/mm}^2 > 0.42\,\text{N/mm}^2$$

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.

$$A_{sv} = \frac{V}{0.87f_y}$$

$$= \frac{33.75 \times 10^3}{0.87 \times 460}$$

$$= 84\,\text{mm}^2/\text{m}$$

5.5 FIGURES AND TABLES FOR CHAPTER 5

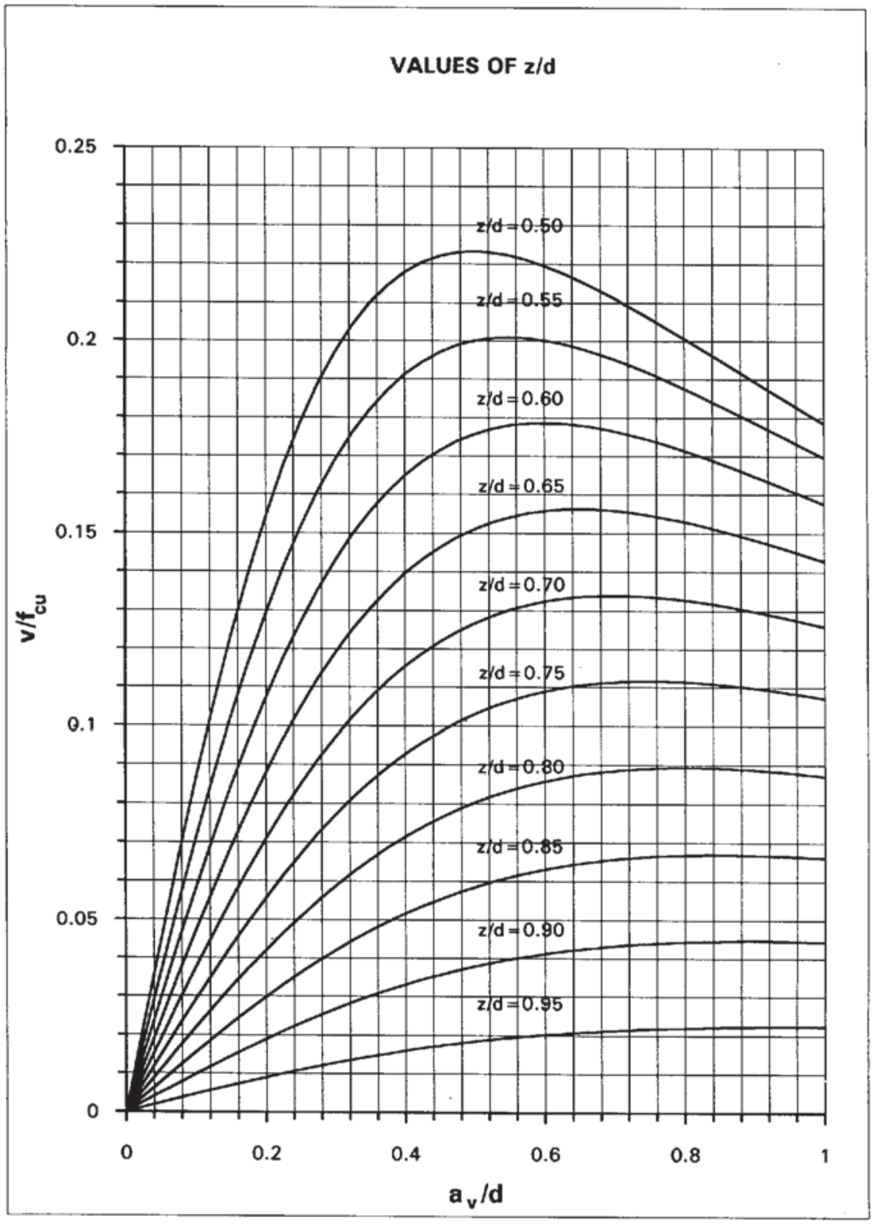


Fig. 5.1 Chart for determining z/d.

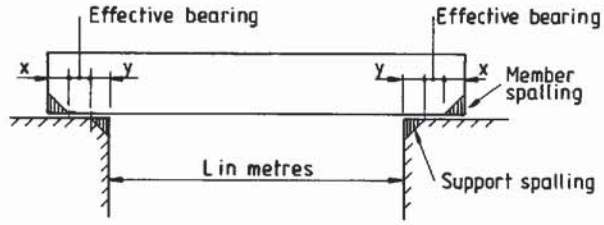


Table 5.1 Allowance for effects of spalling at supports.

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

Table 5.2 Allowance for effects of spalling at supported members.

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

Table 5.3 Allowance for construction inaccuracies.

Material of support	Construction inaccuracy (mm)
Steel or precast concrete support	15 or $3L$, whichever is greater
Masonry supports	20 or $4L$, whichever is greater
In-situ concrete supports	25 or $5L$, whichever is greater