Chapter 10

Design of Connections

10.0 NOTATION

\[ a \]  Distance between two rows of bars resisting bending moment
\[ A_{st} \]  Area of steel in tension
\[ C \]  Embedment of pile into pile cap
\[ D \]  Diameter or width of pile
\[ f_b \]  Ultimate anchorage bond stress
\[ f_y \]  Characteristic yield strength of reinforcement
\[ f_{cu} \]  Design ultimate bond stress
\[ f_{cu} \]  Characteristic cube strength of concrete at 28 days
\[ F_s \]  Ultimate force in a bar or a group of bars
\[ F_t \]  Tie force (kN)
\[ H \]  Design ultimate horizontal load on pile
\[ K_t \]  Coefficient to determine transmission length of prestressing tendons
\[ l \]  Anchorage bond length
\[ l_r \]  Greatest distance between vertical load-bearing elements in direction of a tie
\[ h \]  Floor to ceiling height (m)
\[ l_t \]  Transmission length of prestressing tendons
\[ M \]  Applied bending moment on concrete section of pile
\[ n \]  Number of reinforcement bars in each row
\[ n_o \]  Number of stories in a building
\[ O \]  Perimeter of a bar of reinforcement in tension
\[ r \]  Internal radius of a bend in a bar
\[ v_c \]  Design concrete shear stress
\[ V \]  Shear force in concrete section
\[ z \]  Depth of lever arm

\[ \beta \]  Coefficient to determine design ultimate bond stress
\[ \phi \]  Diameter of bar
\[ \phi_e \]  Diameter of one bar or equivalent diameter of a group of bars

10.1 INTRODUCTION

To make a complete building or structure, the elements described and designed in the previous chapters will have to be connected together and also tied together to give horizontal stability. This chapter describes the principles of design of these connections and the tics.
There are basically two types of connections: rigid and free. The rigid connections will have full moment and other internal force transfer capability. The connections classed as free do not offer resistance to rotation to members at the connection. These connections should be capable of transferring shear and axial loads.

10.2 CONTENTS: TYPE OF CONNECTIONS

The following connections have been described in this chapter:

(1) Requirement of building ties as per Codes of Practice.
(2) Pile-to-foundation/pile cap connection.
(3) Column-to-foundation connection.
(4) Wall-to-foundation connection.
(5) Column-to-column connection.
(6) Wall-to-wall connection.
(7) Column-to-beam connection.
(8) Wall-to-beam connection.
(9) Wall-to-slab connection.
(10) Column-to-wall connection.
(11) Slab-to-beam connection.

The theory of anchorage and bond length requirements is described initially.

10.3 ANCHORAGE AND BOND

Local bond stress is dependent on shear, i.e. the rate of change of bending moment at any section.

Local bond stress = \( \frac{V}{\pi \Sigma o} \)

where  \( V \) = shear at section
\( \pi \) = lever arm of bending moment
\( \Sigma o \) = summation of perimeter of bars in tension.

The local bond stress at ultimate state need not be checked provided there is adequate anchorage of the bars in tension on both sides of a section.

The ultimate anchorage bond stress is assumed constant over the anchorage length of a bar.

\( f_u = \frac{F_s}{\pi \phi_e l} \)

where  \( f_u \) = ultimate anchorage bond stress
\( F_s \) = ultimate force in bar or group of bars bundled together
\( \phi_e \) = diameter of one bar or equivalent diameter of a bar, the area of which equals the total area of the bundle of bars
\( l \) = anchorage bond length.
SK 10/1 Development of bond stress in concrete.

The design ultimate bond stress depends on the characteristic strength of the concrete and is given by the following formula:

\[ f_{bu} = \beta \sqrt{f_{cu}} \]

Values of \( \beta \).

<table>
<thead>
<tr>
<th></th>
<th>In tension</th>
<th>In compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain bars</td>
<td>0.28</td>
<td>0.35</td>
</tr>
<tr>
<td>Type 1: deformed</td>
<td>0.40</td>
<td>0.50</td>
</tr>
<tr>
<td>Type 2: deformed</td>
<td>0.50</td>
<td>0.63</td>
</tr>
<tr>
<td>Fabric</td>
<td>0.65</td>
<td>0.81</td>
</tr>
</tbody>
</table>

A partial safety factor \( \gamma_m = 1.4 \) is included.
The fabric reinforcement should be according to clause 3.12.8.5 of BS8110: Part 1: 1985.

10.3.1 Basic rules of anchorage and laps

**Anchorage of links**
The link is to pass round another bar of equal or greater dimension through an angle of 90° and continue for a minimum length of \( 8 \times \) diameter, or through an angle of 180° and continue for a minimum length of \( 4 \times \) diameter.

**Anchorage of column starter bars**
Column starter bars in compression need not be checked for anchorage. They should be taken down to the level of the bottom layer of reinforcement in the foundation.

Column starter bars in tension should be checked for anchorage.
SK 10/2 A 90° bend.

SK 10/3 A 180° bend.

Laps and joints

SK 10/4 Detailing rules at lap of column bars.

SK 10/5 Lapping of bars.
Laps and joints should be staggered. Welded joints should not be used for cyclic loading. Minimum lap length is 15 times bar size or 300 mm. Links to be used at laps of bars in beams and columns at a maximum spacing of 200 mm where both bars at a lap exceed 20 mm diameter and the cover is less than 1.5 times the diameter of the smaller bar.

10.3.2 Design of tension laps

SK 10/6 Case 1 – anchorage length.

SK 10/7 Case 2 – anchorage length.

SK 10/8 Case 3 – anchorage length.

SK 10/9 Case 4 – anchorage length.
Lap length = tension anchorage length normally
   = 1.4 \times \text{tension anchorage length for Cases 1, 2 and 3}
   = 2 \times \text{tension anchorage length for Case 4}

**Case 1**
Bars lapped are at the top of a section and cover is less than 2 times the size of lapped bars.

**Case 2**
Bars lapped are at a corner of a section and cover to either face is less than 2 times the size of lapped bar.

**Case 3**
The distance between adjacent laps is less than 75 mm or 6 times bar diameter, whichever is the greater.

**Case 4**
Corner bars at the top of a section with less than 2 times diameter of bar cover to either face.
Lapped bars at the top of a section with distance between them less than 75 mm or 6 times bar diameter.

10.3.3 Design of compression laps

Lap length = 1.25 \times \text{compression anchorage length of smaller bar at lap}

*Effective anchorage length of a hook or a bend*

![Diagram of effective anchorage length of a hook or a bend](image)

SK 10/10 Effective anchorage bond lengths.
180° hook
Effective anchorage length = \(8r(\leq 24\phi) + l - 4\phi\)
\[= \text{actual length of bar from tangent point whichever is larger}\]

90° bend
Effective anchorage length = \(4r(\leq 12\phi) + l - 4\phi\)
\[= \text{actual length of bar from tangent point whichever is larger}\]

10.3.4 Curtailment and anchorage of bars

Minimum anchorage length = \(d\) or \(12\phi\)

In tension zone of a flexural member, take a bar to:

- a full tension anchorage length beyond a point where it is not required, or
- a point where the shear capacity of the section is twice the shear force at the point, or
- a point where the available bars continuing beyond provide a moment of resistance twice the bending moment at the point.

The curtailment of bars should be staggered.

Anchorage of bars at a simply supported end

(1) An effective anchorage length of \(12\phi\) beyond centreline of support. Hook or bend should not begin before centreline. Effective anchorage lengths of a hook or a bend may be considered.

SK 10/11 Required effective anchorage at simply supported end where centreline of support is less than or equal to \(d/2\) away from face of support.
SK 10/12 Required effective anchorage at simply supported end where centreline of support is more than \(d/2\) away from face of support.

(2) An effective anchorage length of 12\(\phi\) beyond \(d/2\) from face of support. Hook or bend should not begin before \(d/2\) from face of support. Effective anchorage lengths of a hook or a bend may be considered.

SK 10/13 Required effective anchorage at end of slab where shear stress is less than 0.5\(V_c\).

(3) For slabs, if shear stress at face of support is less than half \(V_c\), then project a straight length of bar beyond centreline of support equal to one third of support width or 30 mm, whichever is greater.

Anchorage bond lengths in multiples of bar sizes for Type 2 deformed bars (\(f_y = 460\,\text{N/mm}^2\)).

<table>
<thead>
<tr>
<th>Grades of concrete</th>
<th>(-f_{cu}) (N/mm(^2))</th>
<th>C25</th>
<th>C30</th>
<th>C35</th>
<th>C40</th>
<th>C45</th>
<th>C50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension anchorage and lap</td>
<td>40</td>
<td>37</td>
<td>34</td>
<td>32</td>
<td>30</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td>Compression anchorage</td>
<td>32</td>
<td>29</td>
<td>27</td>
<td>26</td>
<td>24</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>Compression lap</td>
<td>40</td>
<td>37</td>
<td>34</td>
<td>32</td>
<td>30</td>
<td>28</td>
<td></td>
</tr>
</tbody>
</table>
Anchorage bond lengths in multiples of bar sizes for plain grade 250 N/mm² bars

<table>
<thead>
<tr>
<th>Grades of concrete – $f_{cu}$ (N/mm²)</th>
<th>C25</th>
<th>C30</th>
<th>C35</th>
<th>C40</th>
<th>C45</th>
<th>C50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension anchorage and lap</td>
<td>39</td>
<td>36</td>
<td>33</td>
<td>31</td>
<td>29</td>
<td>27</td>
</tr>
<tr>
<td>Compression anchorage</td>
<td>32</td>
<td>29</td>
<td>27</td>
<td>25</td>
<td>23</td>
<td>22</td>
</tr>
<tr>
<td>Compression lap</td>
<td>39</td>
<td>36</td>
<td>33</td>
<td>31</td>
<td>29</td>
<td>27</td>
</tr>
</tbody>
</table>

Note: The tension anchorage bond lengths will be multiplied by either 1.4 or 2.0, depending on the location of the bar as described in Section 10.3.2.

10.4 BUILDING TIES

The following ties will be considered:

- Peripheral ties.
- Internal ties.
- Horizontal column and wall ties.
- Vertical ties.

Ties are continuous fully anchored and properly lapped welded or mechanically connected tension reinforcement.

The reinforcement required to act as continuous ties is additional to other designed reinforcement. Available excess design reinforcement if properly tied and continuous and capable of carrying the prescribed tie forces may be considered.

10.4.1 Peripheral ties

A continuous tie should be provided at each floor level and roof level within 1.2 m of edge of building or within perimeter wall. This tie should be capable of resisting a tensile force equal to $F_t$ kN.

\[ F_t = 20 + 4n_o \quad \text{or} \quad 60 \quad \text{whichever is less} \]

where $n_o = \text{number of storeys in the structure}$.

Required area of steel for peripheral tie = $A_{st} = \frac{F_t}{0.87f_y}$

This means that the maximum area of steel for peripheral tie at each floor and roof level is given by:

\[ \frac{60 \times 10^3}{0.87 \times 460} = 150 \text{mm}^2 \]

or two 10 mm dia. bars ($f_y = 460 \text{ N/mm}^2$) fully lapped and anchored.
10.4.2 Internal ties

These ties are at floor and roof levels in two orthogonal directions and anchored to peripheral ties or columns or perimeter walls. The spacings of these ties will not be greater than $1.5l_i$, where $l_i$ is greatest.
distance between centres of vertical load-bearing elements in direction of tie.
The ties should be capable of resisting a tensile force equal to the greater of:

$$0.0267 (g_k + q_k) F_t$$ or $$F_t$$

where $$(g_k + q_k)$$ is the sum of the average characteristic dead and imposed
floor loads (kN/m²).

### 10.4.3 Horizontal column and wall ties

**SK 10/17** Anchorage of ties.

If peripheral tie is located in wall, then internal ties should be anchored to
peripheral tie. No other wall tie is required.
Each external column should be tied back horizontally at each floor or
roof level. The tie force will be the greater of (a) or (b) below.

**SK 10/18** External column – elevation showing tie back.

**SK 10/19** Corner column – plan view showing tie back.
(a) $2F_t$ or $(l/2.5) F_t$ if less, where $l_v$ = floor-to-ceiling height (m)
(b) 3% of total design ultimate axial load carried by column

If peripheral tie is not located in wall, then every metre of wall should be tied back at each floor or roof level. The tie force will be either (a) or (b), as above.

The corner column will have horizontal ties at each floor level or roof level in each of two directions, capable of developing a tie force equal to either (a) or (b), as above.

### 10.4.4 Vertical ties

Each column and each wall should be tied continuously from the lowest to the highest level. The tie force in tension will be the maximum design ultimate dead and live load imposed on column or wall from any one storey.

### 10.5 CONNECTIONS

The most commonly occurring structural connections are illustrated in this section with guidance on the preferred detailing methods.

#### 10.5.1 Pile-to-pile cap/foundation raft/ground beam

##### 10.5.1.1 Bored and cast in-situ concrete pile

![Diagram of bored in-situ pile](SK 10/20 Pile-to-foundation connection)

**Case 1**
Mainly vertical loads.
Small horizontal load.
No bending moment in pile at connection.
No tension loads in pile.
Pile embedment \( C \) into pile cap, or raft, or ground beam, up to bottom layer of reinforcement.

*Check bearing stress on concrete (pile and pile cap) due to horizontal load.*

\[
\text{Bearing stress} = \frac{H}{DC} \leq 0.6f_w
\]

where \( H \) = design ultimate horizontal load on pile  
\( D \) = diameter or width of pile  
\( C \) = embedment of pile into pile cap.

*Check anchorage of pile reinforcement.*  
\( l \) = compression anchorage length

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**SK 10/21** Pile-to-foundation connection.

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**SK 10/22** Pile-to-foundation connection.

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**SK 10/23** Pile-to-foundation connection.
Case 2
Vertical compression load.
Vertical tension load.
Piles in swelling clay.
Bending moment in pile at connection.
Horizontal load on pile.

Check bearing stress on concrete as in Case 1.
Check anchorage of pile reinforcement.

\[ l_1 = \text{compression anchorage length} \]
\[ l_2 = \text{tension anchorage length (the effective anchorage length of the bend may be considered)} \]

If bearing stress is higher than allowed, then embedment may be increased by adopting solution in Case 2C.

10.5.1.2 Precast reinforced concrete pile

Case 1 (same condition as in Section 10.5.1.1, Case 1)

Reinforcement to be exposed by breaking out a length equal to \( l \) above pile cut off.

\[ l = \text{compression anchorage length} \]

Check bearing stress, as in Section 10.5.1.1, Case 1.

Case 2 (same condition as in Section 10.5.1.1, Case 2)
Reinforcement to be exposed by breaking out a length equal to \( l \) above pile cut off.

\[ l = \text{tension anchorage length} \]
Check bearing stress, as in Section 10.5.1.1, Case 2.

If bearing stress is higher than allowed then embedment $C$ may be increased by adopting solution in Case 2B.

10.5.1.3 Precast prestressed concrete pile

Case 1 (same condition as in Section 10.5.1.1, Case 1)

Check bearing stress, as in Section 10.5.1.1, Case 1.

Case 2 (same condition as in Section 10.5.1.1, Case 2)

$l = \text{transmission length of prestressing tendons}

= \frac{K_\phi}{\sqrt{f_{cu}}}$
where \( \phi = \) nominal diameter of tendon
\[ K_s = \begin{cases} 
600 & \text{for plain or indented wire} \\
400 & \text{for crimped wire with wave height not less than 0.15 mm} \\
240 & \text{for 7-wire strand or super-strand} \\
360 & \text{for 7-wire drawn strand.}
\end{cases} \]

The top-hat reinforcement detailing at pile connection may be adopted for large vertical load and significant bending moment in pile.

### 10.5.1.4 Steel H-pile or steel tubular pile

**Case 1** (same conditions as in Section 10.5.1.1, Case 1)
Check bearing stress, as in Section 10.5.1.1 Case 1, using width of flange or depth of section, whichever is smaller.
Check bearing stress on concrete on top of mild steel plate using maximum ultimate vertical load on pile.
SK 10/28 Connection of steel pile to foundation.

**Case 2** *(same conditions as in Section 10.5.1.1, Case 2)*

Check bearing stress, as in Section 10.5.1.1 Case 1, using width of flange or depth of section, whichever is smaller.

Check bearing stress on concrete on top of mild steel plate using maximum ultimate vertical load on pile.

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SK 10/29 Moment connection of steel pile to foundation.

\[ l = \text{tension anchorage length of the type of bar used (the effective anchorage length of a hook or a bend may be used)} \]

\[ \phi = \text{diameter of bar} = 1.21 \left( \frac{M}{af_n} \right)^{\frac{1}{2}} \]

\[ M = \text{maximum ultimate bending moment in pile at connection} \]

\[ a = \text{distance between bars perpendicular to axis of rotation or moment, or distance between two rows} \]

\[ n = \text{number of bars in each row} \]
Note: During the driving operation it is difficult to control the orientation of the flanges of H-piles to match the axis of bending moment. In practice, the anchoring bars should be used in such a fashion that the bending moment capacity of these anchor bars are equal in both the orthogonal directions. The mild steel plate should be checked for strength to transfer the anchoring tension of the bars to the web of the H-pile.

For a circular steel pile using 6 no. anchor bar on a circle of diameter $D$ which is diameter of circular pile, diameter of each anchor bar is given by:

$$\phi = 0.92 \left( \frac{M}{Df_y} \right)^{1/4}$$

10.5.2 Column-to-foundation/pile cap/raft

Case 1: Column bars always in compression

SK 10/30 Plan of circular pile showing location of anchor bars for moment connection as in Case 2.

SK 10/31 Connection of column to foundation – no tension in bars.
No check necessary for anchorage bond length. The column bars should be taken down to bottom reinforcement mat of foundation.

Case 2: Significant tension in column bars due to foundation fixity bending moment and/or direct axial tension load

\[ l = \text{tension anchorage length (effective anchorage length of a bend may be used)} \]

10.5.3 Wall-to-foundation/pile cap/raft

Case 1 (same condition as in Section 10.5.2, Case 1) Use same principle as in 10.5.2, Case 1.

Case 2 (same condition as in Section 10.5.2, Case 2) Use same principle as in Section 10.5.2, Case 2.

10.5.4 Column-to-column connection

Case 1: Column bars always in compression

\[ l = \text{compression lap length} \]
The links at the lap will be at a maximum spacing of 200 mm if bars lapped are greater than 20 mm in diameter and cover is less than 1.5 times bar diameter.

**Case 2: Significant tension in column bars due to bending moment or axial tension**

\[ I = \text{tension anchorage length if cover is at least 2 times diameter of lapped bars} \]
\[ l = 1.4 \times \text{tension anchorage length for corner bars where cover to either face is less than 2 times diameter of lapped bars} \]

\[ l = 1.4 \times \text{tension anchorage length if adjacent laps are less than 75 mm or 6 times bar diameter away} \]

10.5.5 Wall-to-wall connection

**SK 10/34** Type 1 – plan of wall at corner.

**SK 10/35** Type 1 – plan of wall at intersection.

**SK 10/36** Type 2 – plan of wall at corner.

**SK 10/37** Type 3 – plan of wall at corner.

*Type 1 and Type 2 connections are efficient for significant reversible bending moment at connection. If horizontal bars are designed to carry significant tension at junction, then lap length is given by:

\[ l = \text{tension anchorage lap length} \]
Type 3 connection may be used where nominal horizontal reinforcement is required to prevent cracking and to contain vertical reinforcement. The lap length is given by:

\[ l = 15 \times \text{bar diameter or } 300 \text{mm, whichever is greater} \]

**Note:** If the loading causes the corner of the wall to open up then Type 2 connection becomes most efficient. Moreover, with Type 2 detailing the horizontal bars could be of different diameters at the inside and the outside faces.

### 10.5.6 Column-to-beam connection

#### 10.5.6.1 External columns

**Case 1: Beam assumed simply supported**

![Diagram of column-to-beam connection](image)

\[ l = \text{tension lap length} \]

See Section 10.3.2 for design of tension lap length.

**Case 2: Beam assumed fixed to column**

\[ l_1 = \text{length required by design calculations } \geq 0.15 \text{ span } \geq \text{tension lap length} \]

\[ r = \text{radius of bend (special radius may be necessary)} \]

Check bearing stress inside bend (see Step 22 of Section 2.3).

\[ l_2 = \text{tension lap length (see Section 10.3.2)} \]

\[ l_3 = \text{tension anchorage length (to be checked if available within depth)} \]
SK 10/39 Connection of beam to column – Case 2.

**Case 3: Beam assumed fixed to column (no reversal of moment)**

SK 10/40 Connection of beam to column – Case 3.

If \( l_3 \) in Case 2 is less than tension anchorage length, use detail in Case 3.

\[ l_1 = \text{tension anchorage length} \]

\[ r = \text{radius of bend} \]

Check bearing stress inside bend (see Step 22 of Section 2.3).

\[ l_2 = \text{length required by design calculations} \geq 0.15 \text{ span} \geq \text{tension lap length} \]

\[ l_3 = 12 \times \text{bar diameter of effective anchorage} \]

\[ l_4 = \text{tension lap length (see Section 10.3.2)} \]
Case 4: Beam assumed fixed to column (ductile connection for reversible moment)

Where bending moments at connection are very large and reversible, e.g. at knee of a portal frame, use detail in Case 4.

- $l_1 =$ tension lap length (see Section 10.3.2)
- $l_2 =$ designed length $\geq 0.15 \text{ span } \geq$ tension lap length
- $l_3 =$ tension anchorage length

May be provided with a hook at end to get full effective anchorage length.

- $r =$ radius of bend
- Check bearing stress inside bend (see Step 22 of Section 2.3).
- $l_4 =$ tension anchorage length
- $l_5 =$ tension anchorage length
- $A_s =$ same area of steel as beam design bottom steel at column

10.5.6.2 Internal columns
Connection uses straight splice bars at intersection.
Splice bars for secondary beam may be placed inside splice bars of main beam.

- $l_1 =$ tension lap length (see Section 10.3.2)
- $l_2 =$ designed length $\geq$ tension anchorage length
10.5.7 Wall-to-beam connection

The same principles apply as in Section 10.5.6.

10.5.8 Wall-to-slab connection

10.5.8.1 Slab simply supported on wall

\[ l_1 = 4 \times \text{thickness of slab or 600 mm or 0.1 \times span, whichever is the greatest} \]

U-bars are same diameter as bottom bars.

\[ l_2 = \text{tension lap or 500 mm, whichever is greater} \]

10.5.8.2 Slab restrained by wall-moment connection
Case 1: Small diameter bars

$l_1$ = the greatest of designed length, tension anchorage length, $4 \times$ thickness of slab, 600 mm, or $0.1 \times$ span

$l_2$ = tension lap length

$l_3$ = tension anchorage length allowing for bends

Check bearing stress inside bend.

Case 2: Large diameter bars

$l_1$ = same as in Case 1

$l_2$ = tension lap length

$l_3$ = tension anchorage length allowing for a bend

Check bearing stress inside bend.
Note: Case 2 detail is used when tension anchorage length cannot be accommodated within bend of U-bar in Case 1.

10.5.9 Column-to-wall connection

Case 1: No significant tension in column bars

Case 2: Significant tension in column bars

10.5.10 Slab-to-beam connection

The same principles apply as in Section 10.5.8.1.