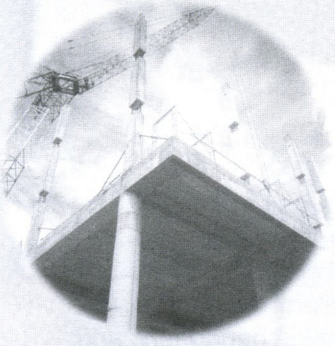


CHAPTER 3

DESIGN OF PRECAST CONCRETE CONNECTIONS

CHAPTER 3 DESIGN OF PRECAST CONCRETE CONNECTIONS

3.1 General

Connections form the most vital part of precast concrete construction. The ingenuity of engineers, researchers and manufacturers has, over the years, developed an extensive range of solutions, theoretical concepts and design equations for various types of connection. The knowledge of precast construction and design particularly those from UK, Continental Europe and North America is well disseminated through the publication of technical reports, research papers, design handbooks and product design information. However, an uninitiated design engineer will face considerable problems when looking for the necessary information to begin his design in precast construction. One of the aims of this part of the Handbook is, therefore, to bring together the wide and varied design methods used in the industry for the benefit of everyone. It is not possible, for obvious reasons, to present a complete overview of all the existing solutions and hence only current practices are included.

3.2 Design Criteria

Connections must meet a variety of design and performance criteria. A satisfactory connection design should, through careful detailing, include the following considerations:

1. a connection must be able to resist the ultimate design forces in a ductile manner
2. precast components are manufactured economically, easy to handle and simple to erect
3. tolerances for manufacturing and field erection must not jeopardise and adversely affect the intended structural behaviour of the components in service
4. the final appearance of the joint must satisfy fire, durability and visual requirements

3.3 Design Considerations

3.3.1 Strength

A connection must resist the forces to which it will be subjected during its lifetime. Some of these forces such as those caused by dead and live gravity loads, winds, earth and water pressures are obvious. Some are not so apparent and will often be overlooked. These are forces caused by the restraint of volume changes in the precast components and those required to maintain stability. Joint strength may be categorised by the type of forces that may be induced such as compressive, tensile, shear, flexural and torsion.

A connection may have a high degree of resistance to one type of force but with little or no resistance to another. While acknowledging the existence of these forces, the designer should not focus on a connection which would resist every conceivable forces. A better solution would be to utilise more than one type of joints to achieve the overall result.

3.3.2 Ductility

Ductility is the ability of a connection to undergo large deformation without failure. Deformation is measured between the first yield and ultimate failure of the structural materials used in the connection. Ductility in building frame is usually associated with moment resistance with the flexural tension capacity provided by the reinforcing bars or structural steel sections. The ultimate failure may be due to the rupture of reinforcing bars, crushing of concrete, the failure of connectors or steel embedded in the concrete.

3.3.3 Volume change

The combined shortening effect due to creep, shrinkage and temperature reductions induces tensile stresses in the precast components. The stresses must be accounted for in the connection design by either providing stress relieve details in unrestrained joint or by providing additional reinforcing steels to resist tensile forces in a restrained joint.

3.3.4 Durability

Exposed sections in a connection should be periodically inspected and maintained. Evidence of poor durability is usually exhibited by corrosion of exposed steel elements, cracking and spalling of concrete. Components which are exposed to weather should have the steel elements adequately encased in concrete or grout, painted, galvanised or using stainless steel sections.

3.3.5 Fire resistance

Connections which may be weakened by exposure to fire should be protected by concrete or grout, enclosed or sprayed with fire resistance materials. The connections should be protected to the same degree as that required for the components and the building frame.

3.4 Manufacturing Considerations

Maximum economy of precast concrete construction is achieved when connection details are kept as simple as possible and they are designed with adequate performance to facilitate easy field erection. The following manufacturing considerations should be noted :

1. Avoid congestion – where large quantity of reinforcing bars, embedded plates, inserts, blockouts, etc, are required, congestion is inevitable. When this occurs, study the area in question using large scale drawings. Use actual physical sizes and dimensions of steel bars, plates, inserts, bolts, etc, instead of line representations. Attention should be focused on areas which may not be reinforced as a result of minimum bending radii of reinforcing bars. It may be necessary to increase the precast components size to avoid congestion and to ensure that sufficient room is left for concrete to be placed without leaving any honeycombs.

2. Avoid penetration of forms – projections such as corbels, nibs, or starter bars which require cutting through the forms are difficult and costly to place. Where possible, these projections should be located at the top surface of the components when casting concrete.

3. Minimise embedded items – embedded items such as inserts, connectors, plates, etc, which require precise positioning and secure fixing are labour and time consuming. They should be kept to a minimum.

4. Avoid operations after stripping the form – operations which are required after stripping the form such as special cleaning, finishing, welding on projecting steel elements should be avoided whenever possible. These operations require additional handling, added labour (often with skilled trade) and extra working space.

5. Allow manufacturing tolerances – dimensional tolerances which are more stringent than industrial standards are difficult to achieve. Connections requiring close fitting parts with little provision for field adjustments should be avoided.

6. Use standard items – wherever possible, embedded items of inserts, plates, bolts, steel sections etc. should be standardised. The items should be available from more than one supplier. Standardisation of items will reduce errors made and will enhance and improve productivity.

7. Use repetitive details – utilise similar connection details as much as possible even if they may result in slight over-sizing. Repeated use of similar details will involve fewer modifications and form setup.

8. Allow alternatives – manufacturers should be allowed to propose alternative methods and detailing provided the design requirements are met. Allowing alternative solutions would generally provide more economical and better performing connections.

3.5 Construction Considerations

Hoisting equipment and lifting operation may constitute a substantial cost item in precast construction. The design of connections should enable the components to be lifted, set and unhooked in the shortest possible time. This will mean that temporary shoring, staying guy ropes, bearing pads, levelling shims and other loose attachments such as angles, bolts, nuts and plates shall be in place or made available at a short notice prior to lifting. The following list of items should be considered in the design to improve on the erection of precast components:

1. Allow for field adjustment – design of connections should avoid details for perfect fit in the field. Certain amount of field adjustment should be allowed by means of oversize holes for bolts and dowels, shimming, welding and grouting.
2. Accessibility for works – connections should be detailed and planned to allow accessibility for working and sufficient room for equipment . For example, wrenches require minimum swinging arc for bolt tightening. Working under the deck in an overhead position should be avoided.
3. Repetitive and standard details – connection details should be standardised and repeated as much as possible. Connections requiring special skills such as welding, prestressing should be reduced so as to make the construction more economical.
4. Robust projections – projections from precast components such as reinforcing bars, bolts, steel sections should be robust and rugged in order to withstand damages due to handling. Threads on projecting bolts should be taped and greased to protect against damage and rust.
5. Allow alternatives – precast manufacturers and erectors should be allowed to propose alternative methods and details not anticipated by the designer. This would often result in more economical and easy erection of precast components.

3.6 Types of Joints

3.6.1 Compressive joints

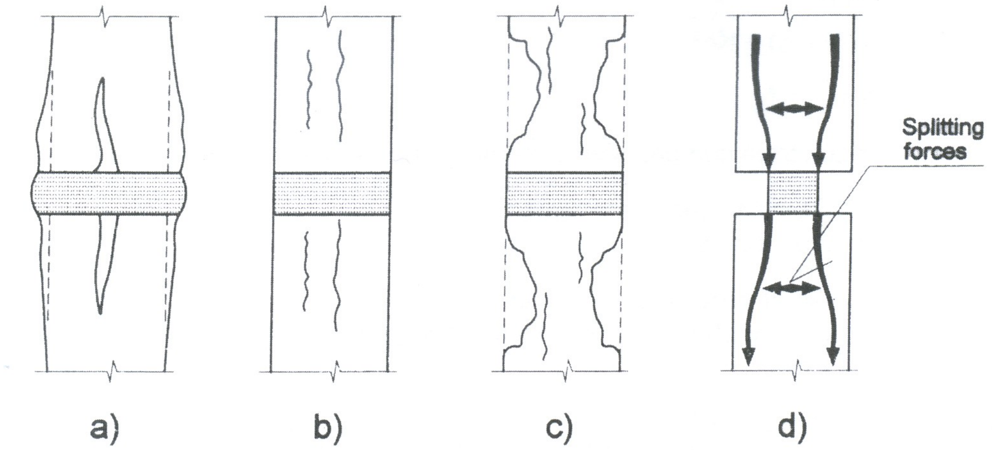
Compressive forces can be transmitted between adjacent precast components by direct bearing or through intermediate medium such as in-situ mortar, fine concrete, bearing pads or other bearing elements.

Direct contact between elements should only be used when great accuracy in manufacturing and erection is achieved and when the bearing stresses are small, usually less than $0.2f_{cu}$. Particular attention should be given in the reinforcement detailing of the precast components when a large concrete cover is required to achieve a high fire rating or when high strength steel bars with large bending radii are used.

It is more common and advisable to use intermediate bedding material for direct transmission of compression forces between precast elements as there will always be surface irregularities at the jointing surfaces. Cementatious materials such as in-situ mortar, fine concrete or grouting are often used in the joints between load bearing elements in columns and walls as well as for beams and floor elements. The nominal thickness is about 10 to 30 mm for mortar and grout, and 30 to 50 mm for fine concrete. The bedding is usually without reinforcing bars. The mode of failure is precipated by crushing of the mortar or splitting of the precast components in contact with it.

Although the mortar, grout or fine concrete is in a highly confined state under predominantly plane stress conditions and should achieve compressive strength higher than f_{cu}, a low design strength is normally used because the edges of the bedding tend to spall off. This will lead to non-uniform stress distribution. The situation can be exacerbated by poor workmanship, unintentional eccentricity, spurious bending moments and shear forces. Another factor which leads to a reduction of the joint strength is when there is a great difference in the elastic response between the bedded material and the precast concrete which may result in localised contraction, lateral tensile stress and splitting forces as shown in Figure 3.1. This effect may become important when the joint thickness is greater than 50 mm.

Hard bearing elements which usually consist of cast-in steel sections or plates with confining reinforcement are used when large concentrated forces are to be transmitted.



Force transfer through compression joint having an elastic modulus
a) less than the precast
b) equal to precast
c) greater than precast
d) as (c) but with reduced breadth

Figure 3.1 Vertical Transfer Of Compressive Forces

There are no explicit design equations in the Code to determine the joint strength taking into consideration the mentioned factors. The Code, however, suggests in Part 1, clause 5.3.6 that in calculating the compressive strength of the mortar joint, the area of the concrete in the joint should be the greater of :

a. the area of the in-situ concrete ignoring the area of any intruding components, but not greater than 90% of the contact area, or
b. 75% of the actual contact area

Vambersky (reference 8) proposed that the bearing capacity of unreinforced mortar joint can be calculated from:

$$n_w = \eta_0 \beta f_{cu} \qquad \text{—— (3.1)}$$

where f_{cu} = weaker concrete compressive strength of either the joint mortar or the precast components adjacent to the joint

η_0 = reduction factor reflecting the trapped air content

For a precast component placed onto a mortar bed η_0=0.3. For the case of joint infill after the components are placed:

η_0 = 0.7 for dry packed mortar and

= 0.9 for colloidal pouring mortar

The effects of joint geometry and the different quality conditions between site mixed mortar and under laboratory tests are reflected in the expression:

$$\beta = K \frac{5(1-K)+\delta^2}{5(1-K)+K\delta^2} \qquad (3.2)$$

where $K = \eta_m f_{cw}/f_{cu}$

f_{cw}/f_{cu} = concrete compressive strength of mortar and precast component respectively

η_m = 0.75 if site cubes are tested
= 1.00 if cores are cut and tested

δ = ratio of joint width to joint thickness
= t/v or x/v where x is the effective compression length of the joint under eccentric loading

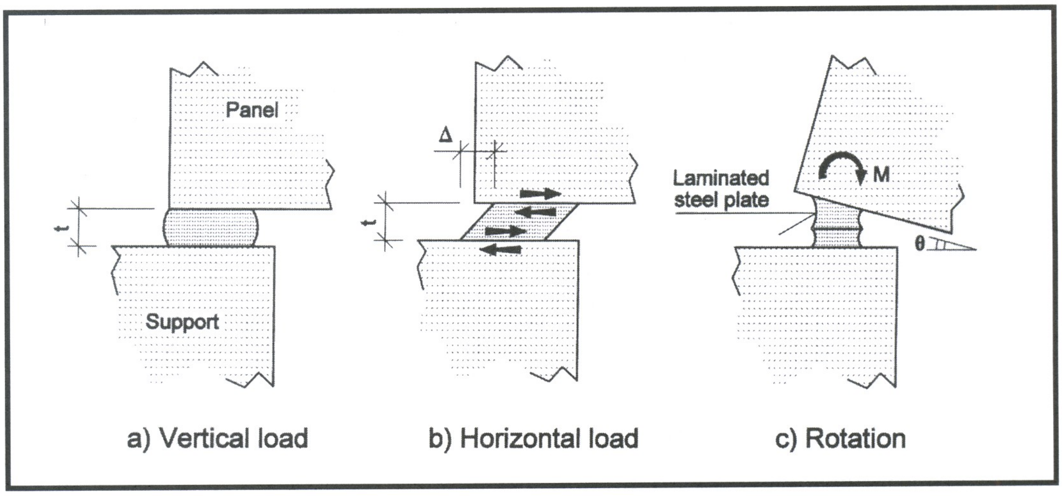
It may be noted from the above expressions that a design stress of $0.4f_{cu}$ (or $0.4f_{cw}$, whichever is lower) as in Part 1, clause 5.2.3.4 of the Code may be adopted as the bearing capacity of the joint provided that:

a. t/v ratios are between 8 and 10

b. the difference in strength between the mortar and the precast component does not differ by more than 25%.

In practice, both the above criteria are generally satisfied.

Soft bedding materials such as neoprene pads are also used to even out surface irregularities. The thickness of the bearing pads may vary from 2 mm to 20 mm or even more. The larger thickness is used to allow displacements and rotations in order to reduce force built-up at the connection as shown in Figure 3.2.

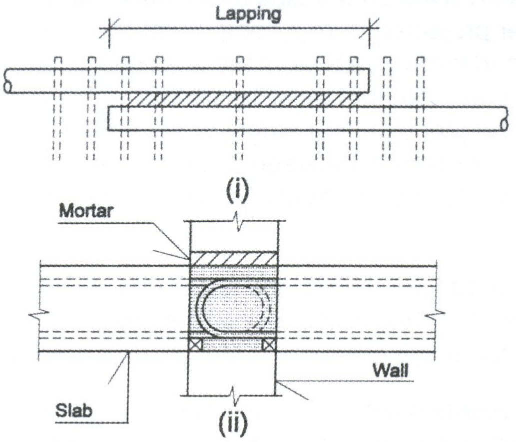


a) Vertical load b) Horizontal load c) Rotation

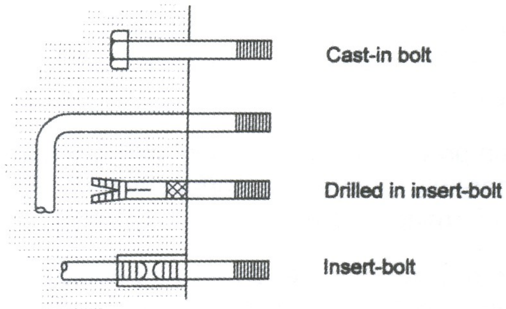
Figure 3.2 Loading Condition At Bearing Pads

3.6.2 Tensile joints

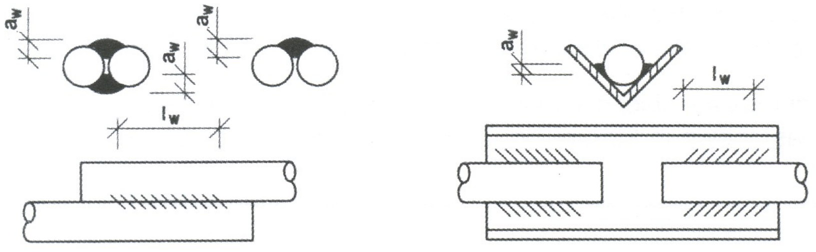
Tensile forces are transferred between concrete elements by means of various types of steel connectors which are anchored into each side of the elements at the joint with continuity achieved by overlapping of steel bars, dowel action, bolting or welding as shown in Figure 3.3.



a) Tensile joint by lapping reinforcement and dowel action

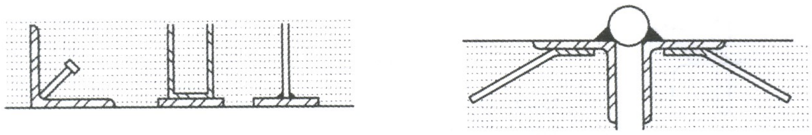


b) Tensile joint by bolting



(i) (ii)

Reinforcing bars welded together



(iii) (iv)

Anchor plate

c) Tensile joint by welding

Figure 3.3 Tensile Force Transfer (reference 3)

The tensile capacity of the connection can be determined by either the strength of the steel elements or by the anchorage capacity. The latter is normally achieved by bonding along reinforcing bars or by means of end anchorage devices.

For the transfer of tensile forces such as vertical tie forces or tension in the force couple of a section under moments, one of the popular methods is the grouted pipe sleeves with in-situ lapped reinforcement. The method involves an annulus metal duct with a diameter of at least 20 to 30 mm larger than the bar diameter projected from a component to be jointed. The bar is inserted into the duct and grout is then injected through a hole at the base. Alternatively, the grout may be placed by gravity pouring. In either cases, the duct must be vented to prevent formation of air pockets. The lapped reinforcement can be placed either singly or symmetrically by the sides of the duct. As in normal lapping, the transfer of forces between bars can be visualised to consist of a series of compressive strut-tension ties. To ensure effective force transfer, stirrups are placed along the lapping length.

Bolting is used extensively to transfer tensile and shear forces. Anchorages such as bolts, threaded sockets and captive nuts are attached to the rear of the plates which are anchored into the precast units. Tolerances are provided using oversized holes in the connecting members.

Welding is used to connect between projections from adjacent precast components. The connection is either made direct or via an intermediate piece.

3.6.3 Shear joints

Shear forces between adjacent precast concrete components can be transferred through bond, interface joint friction, interlocking by shear keys, dowel action of transverse steel bars or rods, welding or by other mechanical means.

Shear transfer by bond between precast and in-situ elements is possible when the shear stress is low. It is not necessary to deliberately roughen the surface texture of precast units beyond the as-cast finish which may be of slip-forming, extrusion or tamped finish.

Shear transfer by shear friction requires the presence of a permanent normal compressive force. The force may arise from permanent gravity loads, by prestressing or artificially induced by reinforcement bars placed across the joint.

Shear keys for the transfer of shear forces between elements are obtained by cast in-situ concrete or grout in joints between the elements with surface castellations. Under the action of shear load, the shear keys act as mechanical locks that prevent significant slip at the interface.

When steel bars or rods are placed across the joint, shear forces can be transferred between elements by dowel action as shown in Figure 3.4. The dowel is loaded by shear at the joint interface and

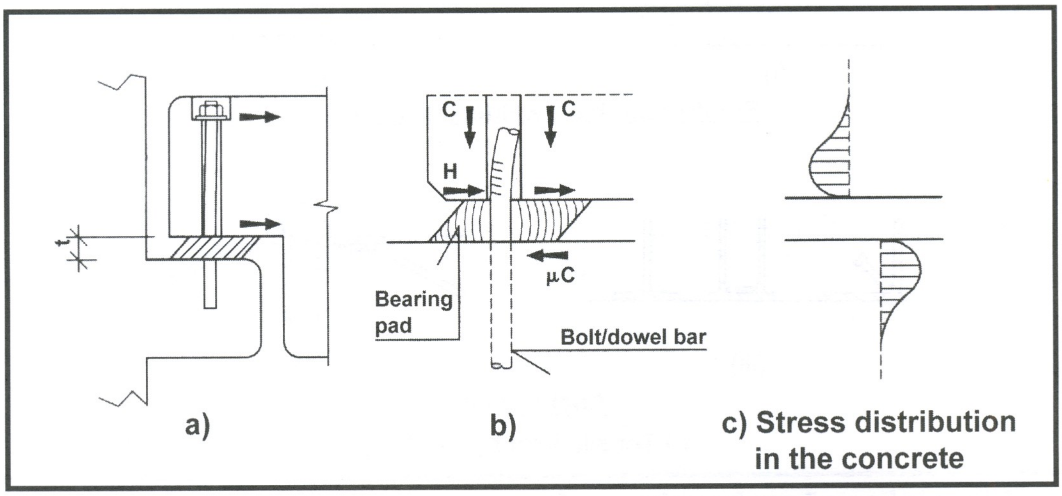


Figure 3.4 Shear Transfer By Dowel Action (reference 3)

supported by contact stresses in the concrete which result in significant bending deformation in the dowel. In the ultimate state, the concrete crushes locally at the contact area and plastic hinge forms at the dowel. Shear capacity depends on the bar diameter and the strength of the concrete. The capacity by dowel action decreases considerably when the dowel is loaded by eccentric shear away from the interface. It is necessary to provide splitting reinforcement around the dowels particularly if they are placed near to the edge or corner of a component. Combined action by shear friction and dowel action can be obtained if the dowels are anchored by bond or by end anchorages.

3.6.4 Flexural and torsional joints

Forces acting in a flexural or torsional joint can always be resolved into tension and compression force couples. The principle in the connection design is based on splicing of reinforcement between units by means of overlapping, bolting or welding as discussed earlier. In the case of torsion, the resulting torque in a precast component is resisted by force couples at the support. The resulting torque is transformed into bending moment in the supporting members.

3.7 Shear Friction Design Method

Shear friction is a very useful and simple method in connection design as well as in the application to the design of composite structures. A basic assumption in the concept is that a crack will form at a potential failure plane where direct shear stresses are high; or at actual planes of weakness created during construction. Propagation of this crack is inhibited by ductile steel reinforcement placed across the cracks. The tension developed by these shear friction reinforcement will provide a force normal to the crack plane as shown in Figure 3.5. If the shear friction coefficient at the crack surface is μ, the resistance to the shear force parallel to the crack, V, is given as

$$V = 0.87 f_y A_s \mu$$

$$A_s = V / (0.87 f_y \mu) \qquad \text{— (3.3)}$$

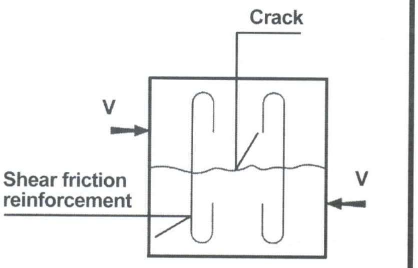
Values of shear friction coefficient μ for various surface conditions are shown in the table below (reference 1).

Type of Surface	μ
Smooth untreated concrete interface	0.7
Artificially roughened or castellated surface	1.4
Monolithic concrete	1.7

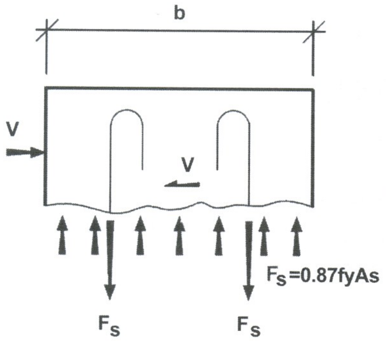
Table 3.1 Shear Friction Coefficients For Various Concrete Surfaces

If there is an applied force N, acting normal to the crack plane, equation 3.3 will be modified as follow:

If N is compression : $A_s = (V / \mu - N) / (0.87 f_y) \qquad \text{— (3.4)}$



(a) Applied shear



(b) Free body forces in concrete above crack

Figure 3.5 Basis Of Shear Friction Design Method

If N is tension : $A_s = (V / \mu + N) / (0.87f_y)$ ——(3.5)

When the interface shear stress reaches the limiting value of $v_c = 0.8\sqrt{f_{cu}}$ or 5 N/mm^2 whichever is smaller, no further increase of shear friction is allowed. This limiting shear stress will provide a limit to the steel proportion ($\rho_s = A_s/bd$) placed across a crack as

$$\rho_s = v_c / (0.87 f_y \mu) \qquad \text{——(3.6)}$$

3.8 Static Friction

The maximum force resulting from frictional restraint of axial movements can be determined by :

$$N = \mu_s V \qquad \text{——(3.7)}$$

where V = design vertical force
μ_s = static coefficient of friction
N = horizontal frictional resistance

The static coefficients of friction are shown in Table 3.2. The coefficients of friction are for a dry condition and the values should be reduced by 15% to 20% for moist conditions. If friction is to be depended upon for support of temporary loads at construction, the coefficients should include a safety factor of 5 (reference 9).

Materials	μ_s
Elastomeric pads to steel or concrete	0.7
Concrete to concrete	0.8
Concrete to steel	0.4
Steel to steel (not rusted)	0.25
Hardboard to concrete	0.5
Laminated cotton fabric to concrete	0.6
Multipolymer plastic (non-skid) to concrete	1.2
Multipolymer plastic (smooth) to concrete	0.4

Table 3.2 Static Coefficients Of Friction Of Dry Material (reference 9)

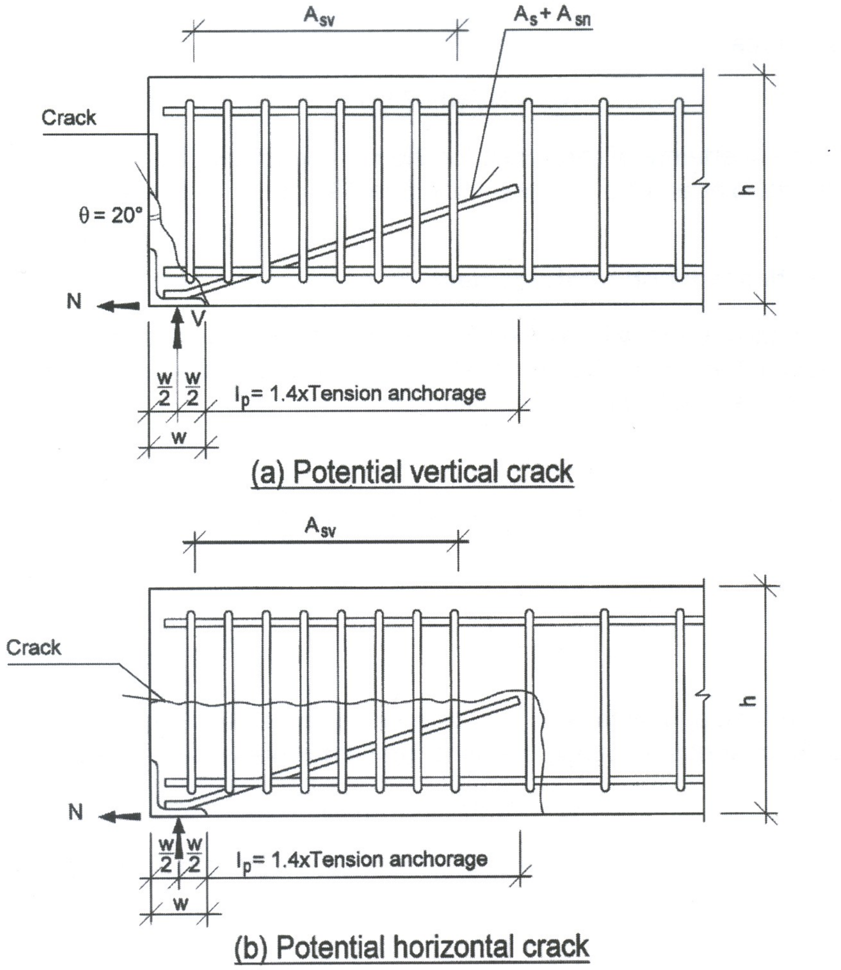
3.9 Bearing On Concrete

The effective bearing area for a structural member may be determined, based on the weaker of the bearing surfaces and using the ultimate concrete bearing stress as follows (reference 1):

1. direct bearing without bedding or bearing pads = $0.4\ f_{cu}$
2. with intermediate bedding = $0.6\ f_{cu}$
3. with cast-in steel bearing plates = $0.8\ f_{cu}$

Bearing using flexible padding may be designed using stresses intermediate between (1) and (2). When the bearing stresses exceed the above limits, reinforcement is to be provided in the bearing area. Reinforcement may be determined using the shear friction design method.

Figure 3.6 shows a bearing area where reinforcement is provided across the potential vertical and horizontal cracks induced by vertical load V and horizontal load N.



Note : w=width of bearing
b=breadth of member
lp=embedment length of horizontal shear friction reinforcement

Figure 3.6 Shear Friction Reinforcement At Potential Cracks At Bearing Area

3.9.1. Horizontal shear friction reinforcement

The horizontal shear friction reinforcement in Figure 3.6a is calculated as follows:

$$A_s = V_\theta / (0.87 f_{yn} \mu) \qquad \text{— (3.8)}$$

where V_θ = V/cos θ
θ = shearing angle (assumes to be 20°)
μ = shear friction coefficient
f_{yn} = characteristic strength of horizontal shear friction reinforcement

The average shear stress, ν_θ at the inclined crack plane is given as

$$\nu_\theta = V \tan\theta / bw \qquad \text{— (3.9)}$$

and $\nu_\theta < \nu_c$, where $\nu_c = 0.8\sqrt{f_{cu}}$ or 5 N/mm², whichever is smaller.

If N is a tensile force

$$A_{sn} = N / (0.87\, f_{yn}) \qquad \text{— (3.10)}$$

Because of the uncertainty of the exact location of the crack, the horizontal shear friction reinforcement should at least have a total embedment length of 1.4 times the tension anchorage value.

3.9.2 Vertical shear friction reinforcement

Potential horizontal crack may be formed as the entire anchorage assembly has the tendency to be pulled horizontally out of the member. The required vertical shear friction reinforcement across the horizontal crack can be calculated as:

$$A_{sv} = 0.87\, f_{yn} (A_s + A_{sn}) / (0.87 f_{yv}\, \mu) \qquad \text{— (3.11)}$$

where f_{yv} = yield strength of vertical reinforcement

If $f_{yn} = f_{yv}$

$$A_{sv} = (A_s + A_{sn}) / \mu \qquad \text{— (3.12)}$$

Shear links used for diagonal tension reinforcement can be considered to act as A_{SV}.
The average shear stress, ν_h, along the horizontal crack plane is given as

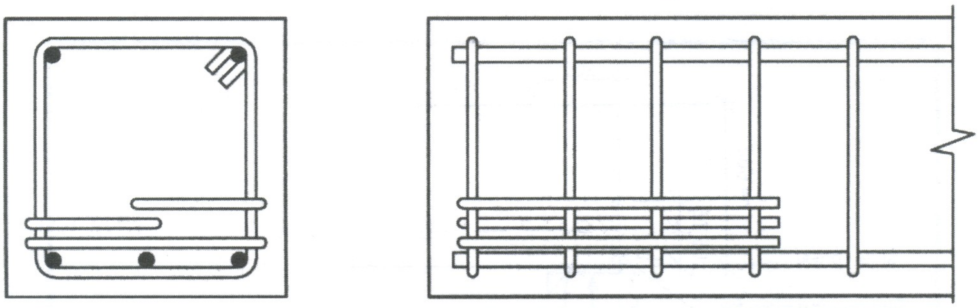
$$\nu_h = 0.87 f_{yv} A_{SV} / (b l_p) \qquad \text{— (3.13)}$$

where l_p = embedment length of the horizontal shear friction reinforcement

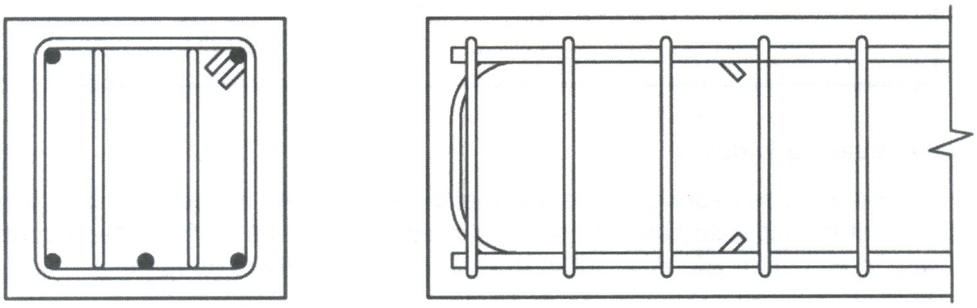
and $\nu_h < \nu_c$

The designer should note that the use of cast-in steel items in reinforced end bearing of precast components as those shown in Figure 3.6 may entail higher production cost and complicate manufacturing processes. Unless necessary, the preferred method of reinforced end bearing should adopt looped reinforcement as shown in Figure 3.7.

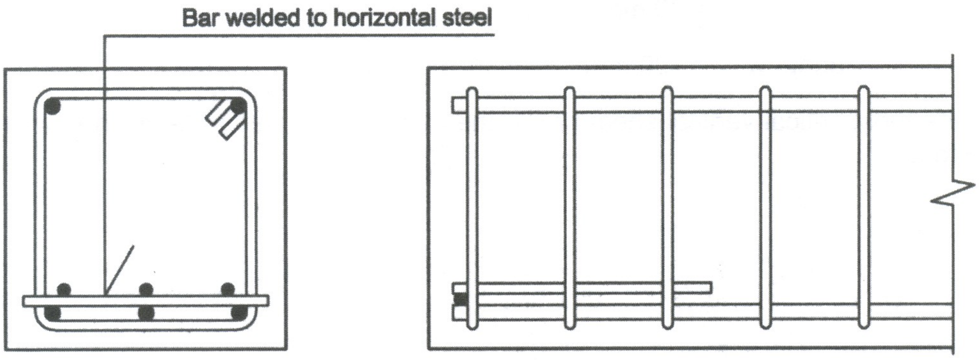
The design of the looped reinforcement is similar in approach as outlined above.



(a) Horizontal looped reinforcement



(b) Vertical looped reinforcement



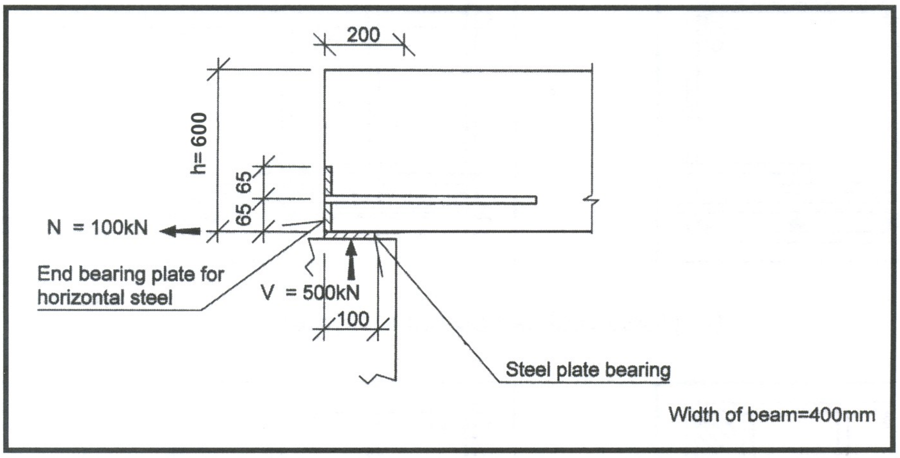
(c) Welded reinforcement

Figure 3.7 Concrete Bearing With Looped Reinforcement

Design Example 1 : Reinforced Concrete Bearing

Design the end bearing of a precast beam 400 x 600mm deep which is subjected to an ultimate vertical reaction of V = 500 kN and a horizontal tension force of N = 100 kN. The beam rests on steel plate at the support. The design concrete cube strength for both beams and supporting member is f_{cu} = 35 N/mm^2

Figure below shows general details of the end bearing with the horizontal shear friction steel butt-welded to a cast-in end plate in the beam:



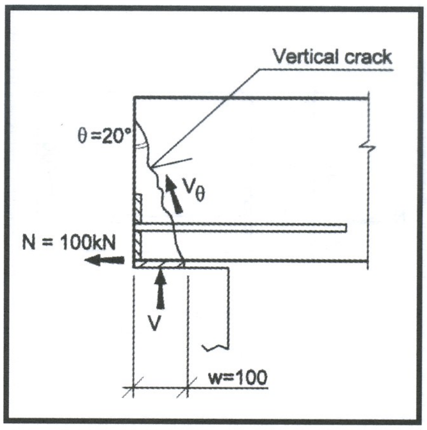
1. Effective bearing width

Steel plate is used as bearing for the beam at the support. As the bearing plate is not cast-in, the design of the ultimate concrete bearing stress is taken as $0.6f_{cu}$ in the support member. Assuming length of the plate l_b= 300mm, the effective width (w) of bearing

$$
\begin{aligned}
w &= V / (0.6\ f_{cu} \times l_b) \\
&= 500 \times 10^3 / (0.6 \times 35 \times 300) \\
&= 79\ \text{mm} \\
\text{adopt } w &= 100\ \text{mm}
\end{aligned}
$$

For the most critical vertical shear crack, the plate is assumed to be flushed with the beam end face.

2. Horizontal shear friction reinforcement



Vertical Shear Crack

a. Determination of A_s

From equation 3.8, the shear friction steel area perpendicular to the inclined vertical crack is

$A_{s\theta}$ = $V_\theta / (0.87 f_{yn}\ \mu)$

V_θ = $V / \cos\theta$

θ = $20°$

μ = 1.7 (monolithic concrete)

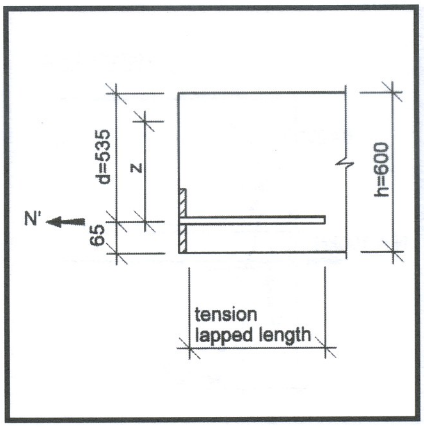
$A_{s\theta}$ = $500 \times 10^3 / (0.87 \times 460 \times 1.7 \times \cos 20°)$
= $782\ mm^2$

The crack is inclined at an angle of 70° to the horizontal shear friction reinforcement. Resolve V_θ vertically, i.e., crack plane perpendicular to the horizontal shear friction reinforcement.

A_s = $A_{s\theta} \cos\theta$
= $782 \times \cos 20°$
= $735\ mm^2$

Note: The above intends to show, the determination of the shear friction reinforcement across a potentical crack from first principle. The calculations can in fact be simplified by directly calculating $A_s = V/(0.87 f_y \mu)$.

b. Determination of A_{sn}



Horizontal Force At Bearing

Resolving the force N at the level to horizontal shear friction reinforcement which is 65 mm above the beam soffit.

$$N' = N(h/z - d/z + 1)$$

where z is the lever arm of tension steel to centroid of the compressive block. Assume conservatively z = 0.8d then

$$\begin{aligned} N' &= 1.25N\,(h/d - 0.20) \\ &= 1.25 \times 100\,(600/535 - 0.20) \\ &= 115.2 \text{ kN} \end{aligned}$$

$$\begin{aligned} A_{sn} &= 115 \times 10^3/(0.87 \times 460) \\ &= 287 \text{ mm}^2 \end{aligned}$$

c. Total horizontal shear friction reinforcement

$$\begin{aligned} A_s + A_{sn} &= 735 + 287 \\ &= 1022 \text{ mm}^2 \end{aligned}$$

Provide 4T20 ($A_s = 1257$ mm^2) welded to the beam end plate.

d. Check ultimate shear stress

From equation 3.9, the average shear stress V_θ along the crack plane is:

$$\begin{aligned} \nu_\theta &= V\tan\theta / bw \\ &= 500 \times 10^3 \tan 20° / (400 \times 100) \\ &= 4.55 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} \nu_C &= 0.8\sqrt{35} \\ &= 4.73 \text{ N/mm}^2 > \nu_\theta \end{aligned}$$

OK

3. Vertical shear friction reinforcement

The vertical reinforcement, A_{sv} , across potential horizontal shear cracks is calculated from equation 3.11.

$$\begin{aligned} A_{sv} &= (A_{sv} + A_{sv}) / \mu \\ &= 1022 / 1.7 \\ &= 601 \text{ mm}^2 \end{aligned}$$

The A_{sv} is to be checked against shear links provision. If the shear links provision is less than A_{sv}, then provide A_{sv}.

Check shear stress at horizontal crack face

Embedded length of T20 $= 1.4 \times 35\ \varphi + w$

$= 980$ mm+100

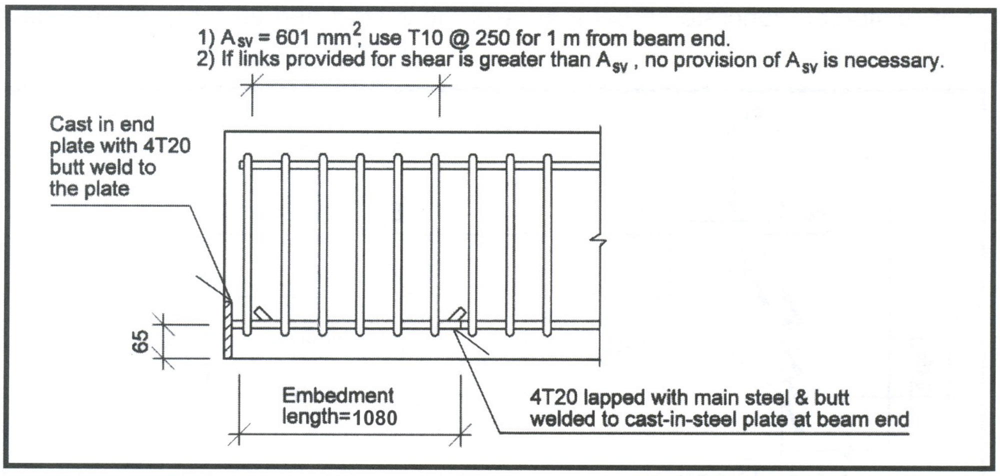
say $l_p = 1080$ mm

Average shear stress v_h at the horizontal crack face is calculated using equation 3.13

$$\begin{aligned} v_h &= 0.87\ f_{yv}\ A_{sv} / (b \times l_p) \\ &= 0.87 \times 460 \times 601 / (400 \times 1080) \\ &= 0.56 \text{ N/mm}^2 < 4.73 \text{N/mm}^2 \end{aligned}$$

OK

4. Detailing



1) $A_{sv} = 601$ mm^2, use T10 @ 250 for 1 m from beam end.
2) If links provided for shear is greater than A_{sv} , no provision of A_{sv} is necessary.

3.10 Reinforced Concrete Corbel

Figure 3.8 illustrates the definitions of reinforced concrete corbel in accordance with the Code in Part 1, clause 5.2.7.1.

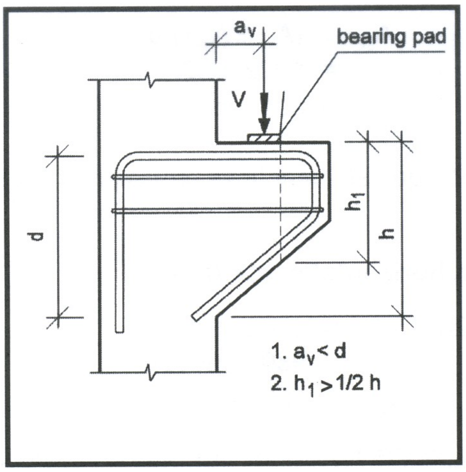


Figure 3.8 Definition Of Corbel

Reinforced concrete corbel may be designed either by:

1. Strut and tie system
2. Shear friction method

3.10.1 Corbel design by strut and tie force system

The design of reinforced concrete corbel may assume a strut and tie force system as shown in Figure 3.9.

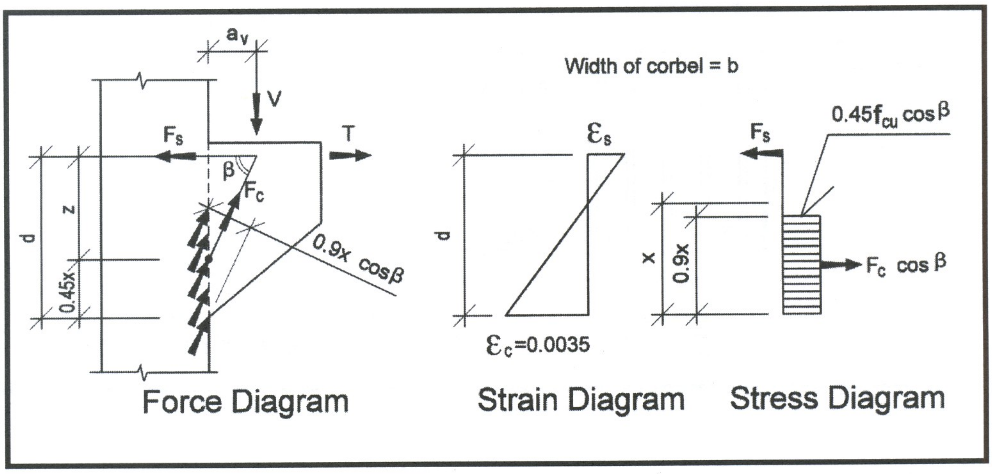


Figure 3.9 Strut And Tie Force System

The design may follow the following steps:

1. **Shear at support face**

The minimum depth of corbel is determined from

a. $$V / bd \leq 2dv_c / a_v$$

$$d \geq \sqrt{Va_v / (2b\, \nu_c)} \quad \text{— (3.14)}$$

b. $$V / bd \leq \nu_c$$

$$d \geq V / b\nu_c \quad \text{— (3.15)}$$

where ν_c in b is equal to $0.8\sqrt{f_{cu}}$ or $5N/mm^2$, whichever is smaller.

2. **Strut and tie forces**

From Figure 3.9, the strut and tie forces are derived as :

Tension tie forces :

$$F_s = T + F_c \cos\beta$$

$$F_s = T + V\, a_v/z \quad \text{— (3.16)}$$

where T is the horizontal force acting at the support.

3. **Compressive strut forces**

$$F_c = 0.45 f_{cu}\, b(0.9\, \chi \cos\beta)$$

$$F_c = 0.405 f_{cu}\, b\, \chi \cos\beta \quad \text{— (3.17)}$$

4. **Derivation of z/d**

$$V = F_c \sin\beta$$

From equation 3.17, $V = 0.405 f_{cu}\, b\, \chi \cos\beta \sin\beta$

$$\cos\beta = \frac{a_v}{(a_v^2+z^2)^{1/2}}$$

$$\sin\beta = \frac{z}{(a_v^2+z^2)^{1/2}}$$

$$V = 0.405 f_{cu}\, b\, \chi \frac{a_v\, z}{(a_v^2 + z^2)}$$

$$\nu = V/bd$$

$$\nu = 0.405 f_{cu} \left(\frac{z}{d}\right) \frac{\chi\, a_v}{(a_v^2 + z^2)} \quad \text{— (3.18)}$$

Substituting $\chi = (d - z)/0.45$ into equation 3.18 and rearranging the terms will result in the following simplified expressions :

$$\frac{\nu}{f_{cu}} = \frac{0.9(z/d)(a_v/d)(1 - z/d)}{(a_v/d)^2 + (z/d)^2} \qquad \text{— (3.19)}$$

From equation 3.19, a graph for the determining of z/d is shown in Figure 3.10.

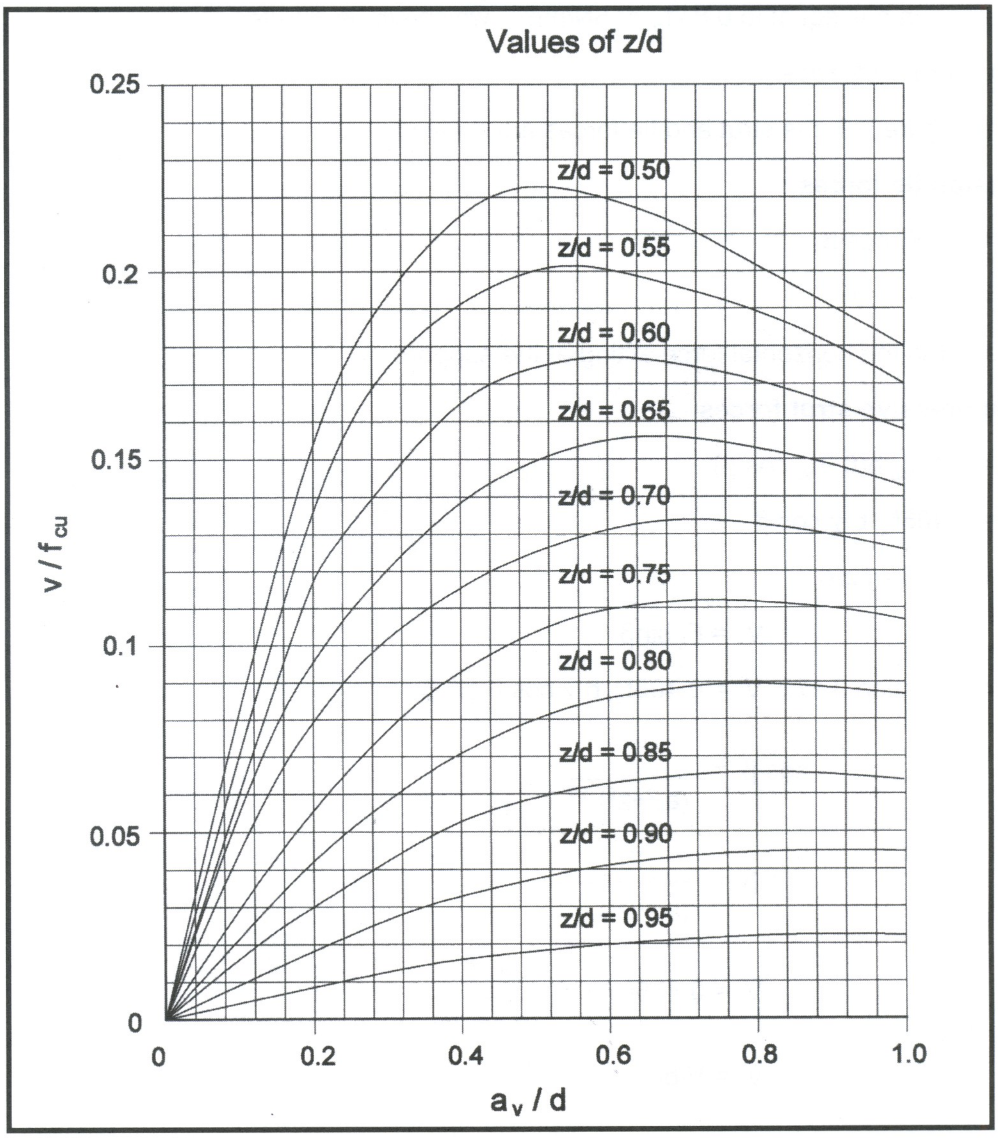


Figure 3.10 Chart For Determining z/d

Part 1, clause 5.2.7.2 requires that the resistance provided to horizontal tie force should not be less than one-half of the design vertical load and in the absence of external horizontal forces,

$$F_s = V\,(a_v / z)$$
$$= V\,(a_v / d) / (z / d)$$

If

If $F_s = 0.5V$, then

$$z/d \geq 2\,a_v/d \qquad \text{— (3.20)}$$

Substituting equation 3.20 into equation 3.19

$$v/f_{cu} = 0.36(1 - 2a_v/d) \qquad \text{— (3.21)}$$

Equation 3.21 provides the boundary below which F_s is taken as $F_s = V/2$

5. Steel stresses

From the strain diagram in Figure 3.9,

$$\varepsilon_s/(d - \chi) = \varepsilon_c/\chi$$

Substituting $\varepsilon_c = 0.0035$, $\chi = (d - z)/0.45$, $f_s = E_s\varepsilon_s$ and $E_s = 200$ kN/mm^2 into the above equation, the steel stress can be calculated :

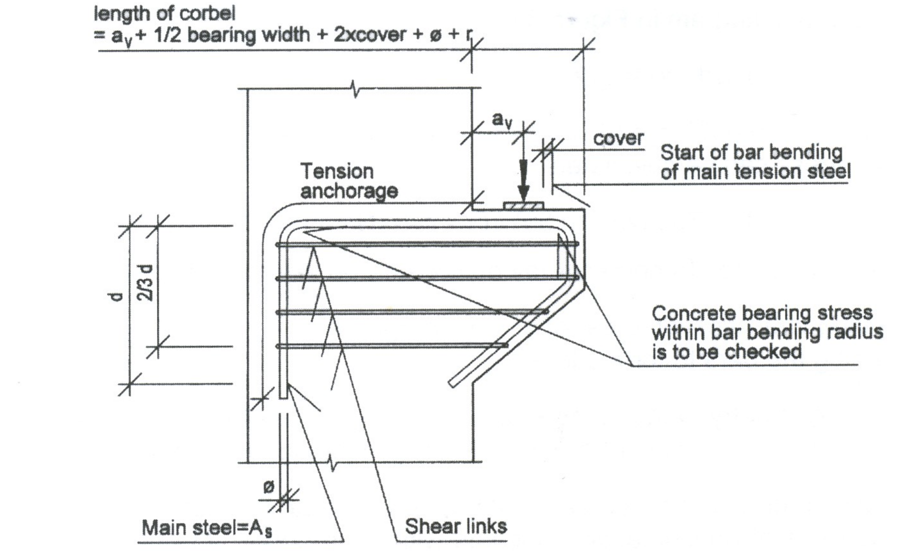
$$f_s = 700\ (z/d - 0.55)\ /\ (1 - z/d)\ (\text{N/mm}^2) \qquad \text{— (3.22)}$$

The main tension reinforcement may be considered anchored by:

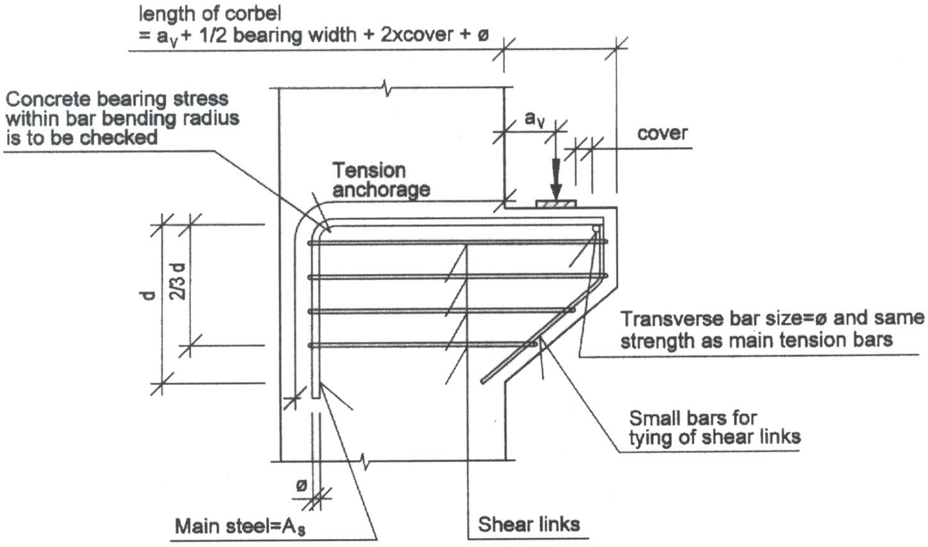
a. welding to a transverse bar of equal strength or
b. bending the bars to form a loop

The length geometry of the corbels with anchorage of reinforcement using method (a) or (b) and the relative positioning of bearing plates or bedding are illustrated in Figure 3.11.

Shear reinforcement, if required, must not be less than 50% of the main tension steel area and it consists of horizontal links located within the upper 2/3 of the effective depth of the corbel.



(a) Looping of main bars



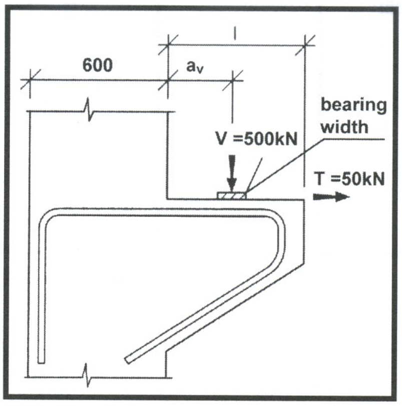
(b) Welding of transverse bars

Note : Min. shear links area if needed, $\geq 1/2\ A_s$

Figure 3.11 Anchorage And Relative Position Of Bearing Plates In R.C. Concrete Corbel

Design Example 2: Corbel Design By Strut And Tie Forces System

Design a reinforced concrete corbel to support a vertical ultimate load V = 500 kN and an ultimate horizontal force of T = 50 kN. The corbel is projected from a column 600 x 400 with width 400 mm and a_v = 100 mm from the face of the column. Bearing plate is used to transmit both vertical and horizontal load to the corbel. Assume cover to all steel = 35 mm and f_{cu}= 35 N/mm^2.



1. Corbel geometry

a. Assume tension bar = $\phi 16$
link size = $\phi 10$

b. Bearing width :
Length of bearing width l_b = 350 mm
Maximum bearing stress = $0.6f_{cu}$

Width of bearing, $w = V/0.6f_{cu}\ l_b$
$= 500 \times 10^3/(0.6 \times 35 \times 350)$
= 63.5 mm
say w = 65 mm

c. Length of corbel (assuming looped tension reinforcement)

Length of corbel $= a_v + 0.5w + 2 \times \text{cover} + \phi_{link} + \phi_{main}$ + bend radius of main steel (4ϕ)
$= 100 + 0.5 \times 65 + 2 \times 35 + 10 + 16 + 4 \times 16$
= 292.5 mm,
say = 300 mm

d. Depth of corbel

Assume overall depth of corbel h= 400mm and with the following dimensions :

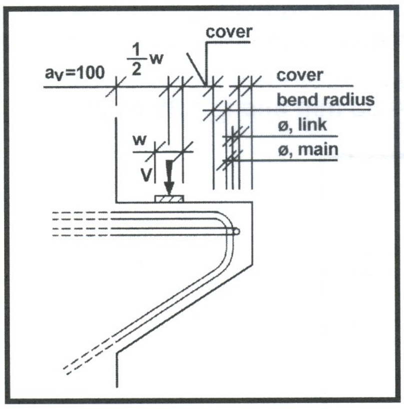
$h_1 = 200 + (167.5 \times 200/300)$
$= 311\text{mm} > h/2$ OK

$d = 400 - 35 - 8$
$= 357\text{mm} > a_v$ OK

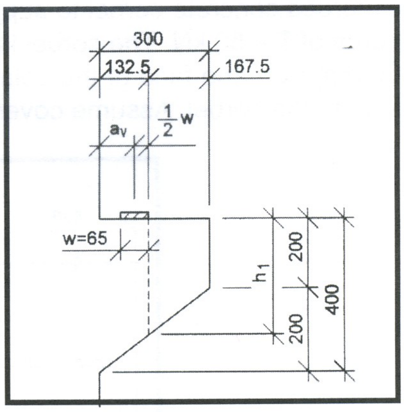
e. Check shear stress

$\nu = V / bd$
$= 500 \times 10^3/(400 \times 357)$
$= 3.50 \text{ N/mm}^2 < 0.8\sqrt{f_{cu}}$ OK
$= 4.73 \text{ N/mm}^2$

The final dimensions of the corbel are shown below:



Length Of Corbel



Depth Of Corbel

2. Corbel reinforcement

a. Main tie steel

Check if $F_s > V / 2$

From equation 3.21, $\nu/f_{cu} = 0.36(1 - 2a_v/d)$

$\nu/f_{cu} = 0.158$

But $\nu/f_{cu} = 3.5/35 = 0.100 < 0.158$

Hence, design for minimum tension

$$F_s = 0.5V = 0.5 \times 500 = 250 \text{ kN}$$

Total tension force in tie $= F_s + T$

$= 250 + 50$

$= 300$ KN

b. Check steel stress

From equation 3.19,

$$\frac{\nu}{f_{cu}} = \frac{0.9(z/d)(a_v/d)(1 - z/d)}{(a_v/d)^2 + (z/d)^2}$$

$\nu/f_{cu} = 0.10$

$a_v/d = 0.28$

Substituting $\nu/f_{cu} = 0.10$ and $a_v/d = 0.28$ into the equation and re-arranging the terms, the following quadratic equation for z/d is obtained :

$(z/d)^2 - 0.716(z/d) + 0.0227 = 0$

$z/d = 0.683$

Alternatively, z/d may be obtained directly from the graph in Figure 3.9.

Substituting z/d = 0.683 into equation 3.22

$$f_s = 700(0.683 - 0.55)/(1 - 0.683) = 293 \text{ N/mm}^2$$

Hence, tension steel area

$A_s = (F_s + T)/f_s$
$= 300 \times 10^3/293$
$= 1024\ \text{mm}^2 > 0.004bd$ (min A_s) OK

Provided 6T16 (A_s=1207mm^2)

c. Check bearing stress within bend

Tensile force per bar

$$F_s' = \frac{(F_s + T)}{\text{numbers of bar}} \times \frac{A_{s\,req}}{A_{s\,prov}}$$

$= (300/6) \times (1024/1207)$
$= 42.4$ kN

Minimum bend radius

$$r = \frac{F_s'}{\phi} \times \frac{1 + 2(\phi/a_b)}{2f_{cu}}$$

$a_b = (400 - 35 - 35 - 16)/5 = 62.8$ mm say $= 60$ mm

$r = 42.4/16 \times 10^3 \times (1 + 2 \times 16/60) / (2 \times 35)$
$= 58$mm

$r = 4\phi$ as assumed OK

d. Shear links

$\rho_s = 100A_s/bd$
$= 100 \times 1207/(400 \times 357)$
$= 0.845$

From Table 3.9 Part 1 of the Code

$\nu_c = 0.67\ \text{N/mm}^2$

Enhanced $\nu_c' = 2d\ \nu_c/a_v$
$= 2 \times 357 \times 0.67/100$
$= 4.77 > 0.8\sqrt{f_{cu}}$

Hence $\nu_c' = \nu_c = 4.73\ \text{N/mm}^2$

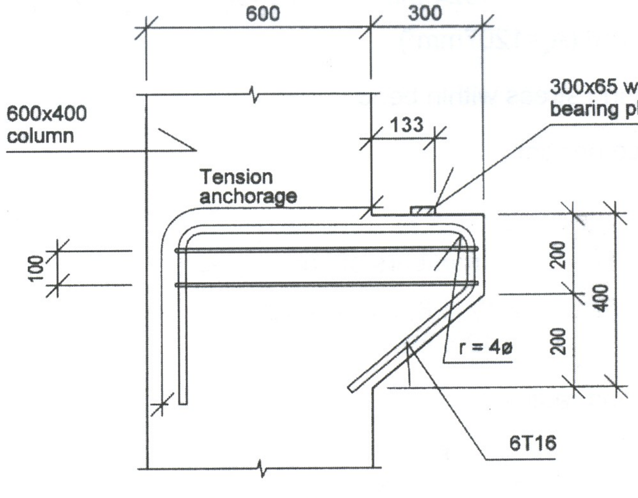
Shear stress $\nu = 500 \times 10^3/(400 \times 357)$
$= 3.50\ \text{N/mm}^2 < \nu_c'$ OK

No shear links needed. But provide 25% of main steel for confinement purpose as well as for binding of the main steel.

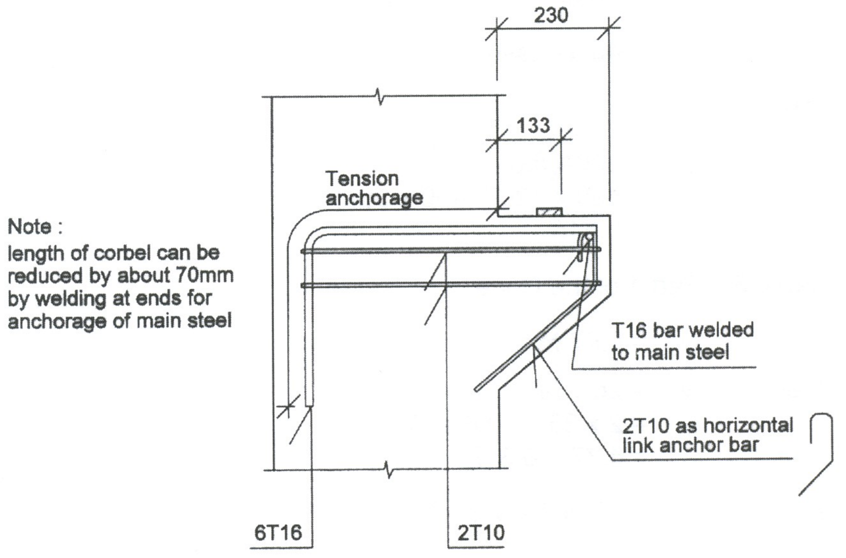
Link area $= 0.25 \times A_{s,\,req}$
$= 0.25 \times 1045$
$= 261\ \text{mm}^2$

Provide 2 numbers of T10 links (4 legs)

3. Detailing



(a) Loop reinforcement



Note :
length of corbel can be reduced by about 70mm by welding at ends for anchorage of main steel

(b) Alternative reinforcement detailing

Corbel Reinforcement Design Using Strut And Tie Forces System

3.10.2. Corbel design by shear friction method

The design of corbel by shear friction method involves the investigation of several potential crack planes as illustrated in Figure 3.12. The associated reinforcement for each of the crack plane being considered is as follows:

1. Flexural (cantilever bending) and axial tension at the corbel projection. Provide reinforcement A_s (flexural) and A_{sn} (axial tension).
2. Direct shear at corbel junction with main member from which the corbels are projected. Provide shear friction A_s and A_{sh}.
3. Diagonal tension in corbel. Provide shear friction reinforcement A_{sh}
4. Inclined shear crack at bearing. Provide A_s and A_{sh} and ensure anchorage of these steel is achieved.

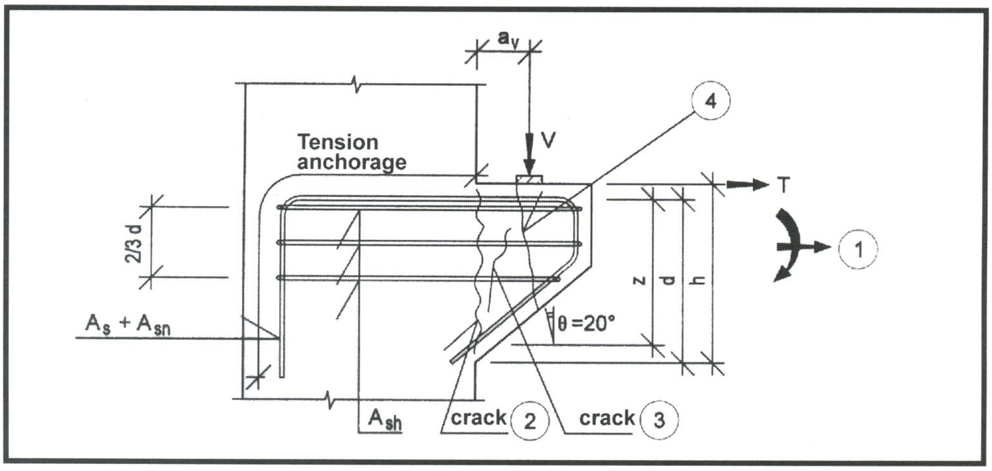


Figure 3.12 Potential Cracks In Corbel

The reinforcement calculated above is not cumulative but the greater of the different considerations. As stated in the shear friction design method, the average shear stress at any crack plane must be less than or equal to $0.8\sqrt{f_{cu}}$ or 5 N/mm², whichever is smaller.

The reinforcement by shear friction design method is calculated as below:

1. Flexural and axial tension steel

Flexural steel : $A_s = V(a_v / z) / (0.87f_y)$

Axial steel : $A_{sn} = T(h / z - d / z + 1) / (0.87f_y)$

Assuming z = 0.8d, then

$$A_s = 1.25V \times (a_v / d) / (0.87f_y) \quad \text{— (3.23)}$$

$$A_{sn} = 1.25T \times (h / d - 0.2) / (0.87f_y) \quad \text{— (3.24)}$$

2. **Direct shear at junction with main member**

$$A_s = \frac{2}{3} \times V / (0.87 f_y \mu) \qquad \text{—— (3.25)}$$

$$A_{sh} = \frac{1}{3} \times V / (0.87 f_y \mu) \qquad \text{—— (3.26)}$$

A_{sn} is as in equation 3.22 above. A_{sh} should be distributed uniformly within (2/3)d of the corbel depth and minimum $A_{sh} = 0.5(A_s + A_{sn})$

3. **Bearing on corbel**

The design approach is described in Section 3.9.

Design Example 3: Corbel Design By Shear Friction Method

Redesign the corbel in Design Example 2 using shear friction design method. Adopt the corbel geometry and all relevant design parameters.

$V = 500$ KN
$T = 50$ KN
$a_v = 100$ mm
$d = 357$ mm
$z = 0.8d$
$= 286$ mm
$\mu = 1.7$ (monolithic concrete)
$f_y = 460 N/mm^2$ for all steel bars

1. **Flexural and axial tension steel**

From equation 3.23,

$$\begin{aligned} A_s &= 1.25V\,a_v/(0.87f_y\,d) \\ &= 1.25 \times 500 \times 10^3 \times 100/(0.87 \times 460 \times 357) \\ &= 437\ mm^2 \end{aligned}$$

From equation 3.24,

$$\begin{aligned} A_{sn} &= 1.25T(h/d - 0.2)/0.87f_y \\ &= 1.25 \times 50 \times 10^3(400/357 - 0.2)/(0.87 \times 460) \\ &= 144\ mm^2 \end{aligned}$$

$$A_s + A_{sn} = 581\ mm^3$$

2. **Direct shear at junction**

From equation 3.25,

$$A_s = \frac{2}{3} \times V / (0.87f_y\,\mu)$$

$$\begin{aligned} A_s &= 2 \times 500 \times 10^3/(3 \times 0.87 \times 460 \times 1.7) \\ &= 490\ mm^2 \end{aligned}$$

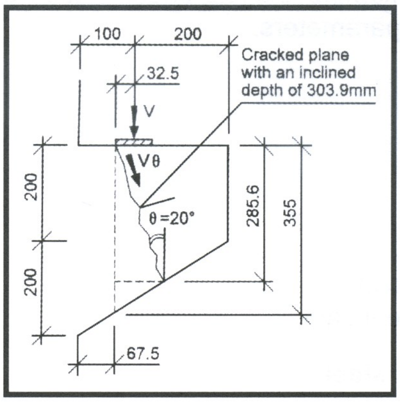
From equation 3.26,

$$\begin{aligned} A_{sh} &= \frac{1}{3} \times V / (0.87fy\,\mu) \\ &= 500 \times 10^3/(3 \times 0.87 \times 460 \times 1.7) \\ &= 245\ mm^2 \end{aligned}$$

$A_{sn} = 144\ mm^2$ as in 1 above.

Required total steel area:
$A_s + A_{sn} = 634\ mm^2$
$A_{sh} = 245\ mm^2$

3. Bearing on corbel



Inclined Crack At Bearing

From equation 3.8 and resolving inclined crack plane vertically

$$\begin{aligned} A_s &= V/0.87 f\mu \\ &= 500 \times 10^3/(0.87 \times 460 \times 1.7) \\ &= 735 \text{ mm}^2 \end{aligned}$$

From equation 3.8 and transferring the axial force to the A_{sn} level

Then $A_{sn} = 1.25T\,(h/d - 0.2)/(0.87f_y)$

as in (1) above, $A_{sn} = 144 \text{ mm}^2$

$$\begin{aligned} A_s + A_{sn} &= 735 + 144 \\ &= 879 \text{ mm}^2 \end{aligned}$$

4. Check average shear stress at inclined crack plane at bearing

$$\begin{aligned} v_\theta &= V/(\cos\theta \times bh) \\ &= 500 \times 10^3/(\cos 20^\circ \times 400 \times 303.9) \\ &= 4.38 \text{ N/mm}^2 < 0.8\sqrt{f_{cu}} = 4.73 \text{ N/mm}^2 \end{aligned}$$

5. Design steel area

Steel area calculated from 1 to 3

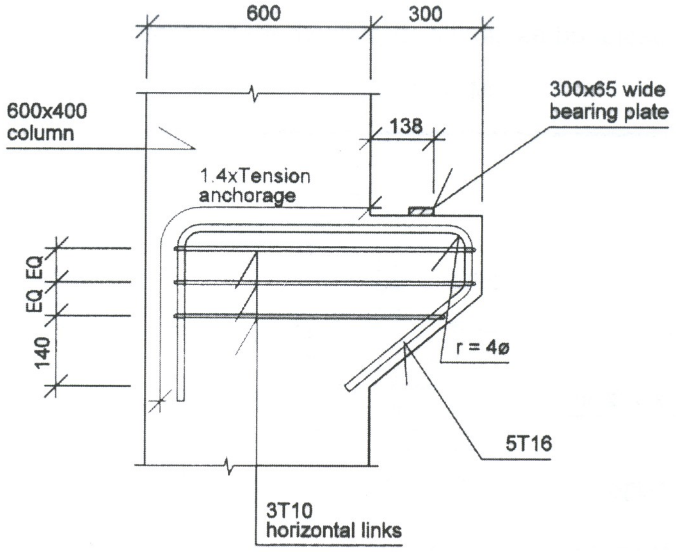
i. Max $A_s = 735 \text{ mm}^2$
Max $A_{sn} = 144 \text{ mm}^2$
$A_s + A_{sn} = 879 \text{ mm}^2$, use 5T16 ($A_s$=1006mm^2)

ii. Max $A_{sh} = 245 \text{ mm}^2$
Check Minimum $A_{sh} = 0.5\,(A_s + A_{sn})$
$= 0.5\,(735 + 144)$
$= 440 \text{ mm}^2$

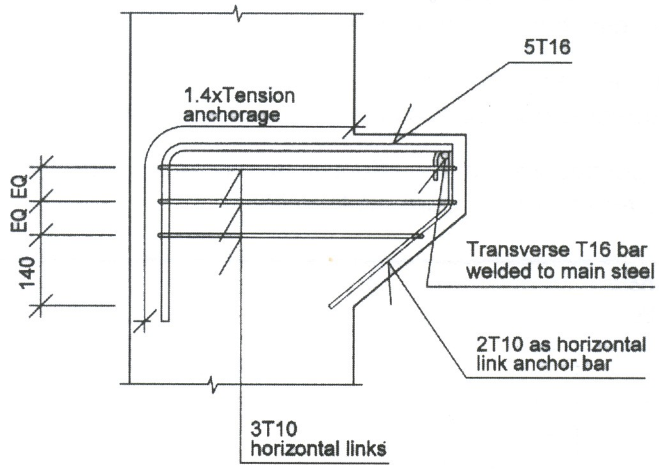
use 3T10 (A_{sh} = 471mm^2, 2 legs) and space them at (2/3)d in the corbel.

The checks for concrete bearing stress for T16 bars are similar to Design Example 2.

6. Detailing



(a) Loop reinforcement in corbel



(b) Alternative Reinforcement Detailing

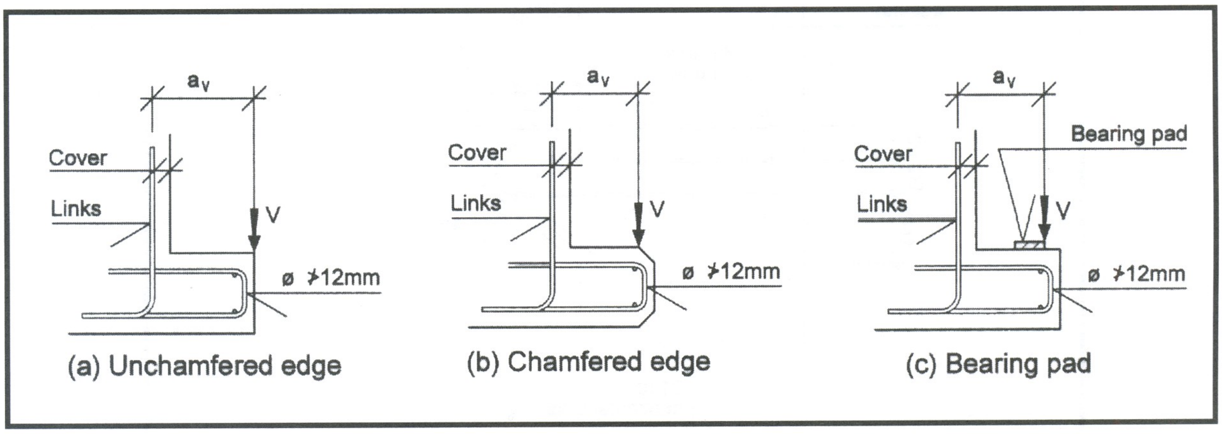
Corbel Reinforcement Designed Using Shear-Friction Method

3.11 Reinforced Concrete Nib

Reinforced concrete nibs are short cantilever projections from walls, columns or beams to provide support for floor elements. The nibs are usually less than 300 mm deep with a_v greater than the effective depth. Figure 3.13 illustrates the line of action of vertical load and the various a_v as defined in accordance with Part 1 clause 5.2.8.1 of the Code.

The concrete nibs are designed as cantilever slab where the moment in the nib is taken as :

$$M = V \times a_v \qquad \text{— (3.27)}$$



(a) Unchamfered edge (b) Chamfered edge (c) Bearing pad

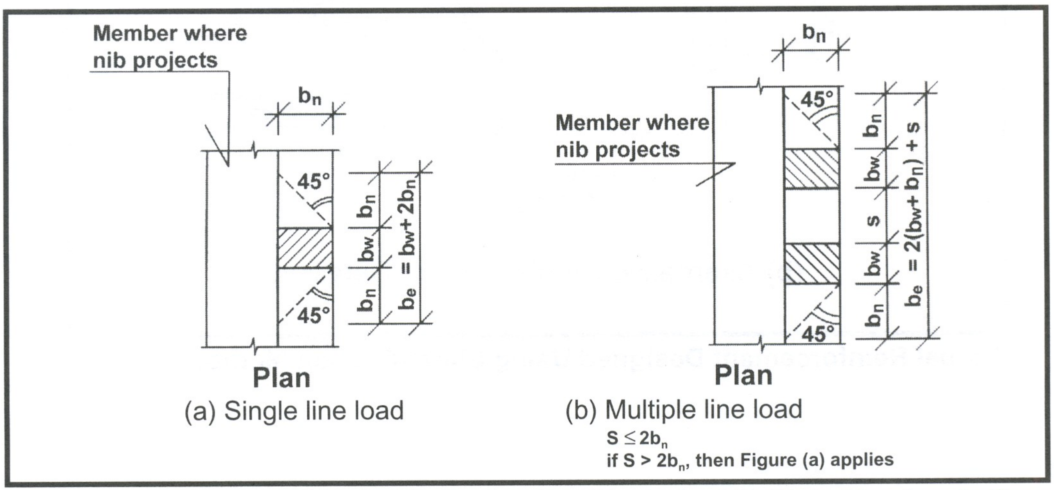
Figure 3.13 a_v In Concrete Nib

The design concrete shear stress v_c may be enhanced by a factor $2d/a_v$. As in the slab design, the depth should be sufficiently deep to avoid the provision of shear links within the nib.

The main tension reinforcement may be anchored in a similar manner as in corbel. The size of main tension steel should be less than 12 mm in diameter.

Vertical reinforcement consisting of links should be provided in the member from which the nib projects. The reinforcement should be designed to carry the vertical load on the nib.

For isolated loads in a continuous nib, the effective width of load dispersal may assume to be at a 45° angle line of failure as shown in Figure 3.14.



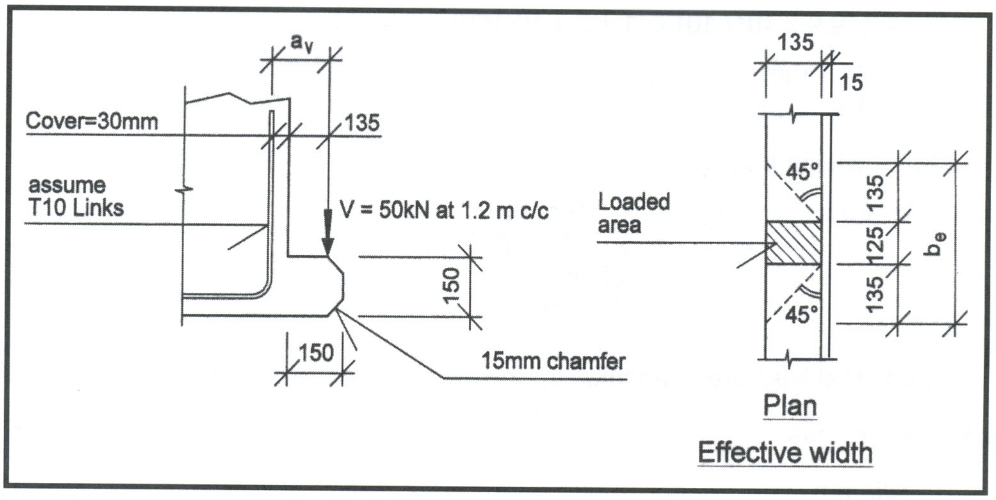
Plan
(a) Single line load

Plan
(b) Multiple line load
$S \leq 2b_n$
if $S > 2b_n$, then Figure (a) applies

Figure 3.14 Effective Width In Nib Design For Single And Multiple Line Loads On Nib

Design Example 4: Reinforced Concrete Nib

Design the reinforcement in the nib of a beam supporting multiple ultimate line loads of 50 kN at 1.2 m spacing. The loaded width is 125 mm; the nib is 150 mm wide and 150 mm deep with 15 mm chamfered at the outer edge. Concrete mortar is used as bedding material. Adopt design concrete strength f_{cu} = 35 N/mm^2 and concrete cover = 30 mm.



The nib is designed as isolated single line load using an effective width, b_e

$$b_e = 2 \times 135 + 125$$
$$= 395 \text{ mm} < 1200 \text{ mm}$$

1. Main tension steel

$$a_v = 150 - 15 + 30 + 5$$
$$= 170 \text{ mm}$$

$$d = 150 - 30 - 5$$
$$= 115 \text{ mm}$$

Moment, $M = V \times a_v$
$= 50 \times 0.17$
$= 8.5$ kNm

$$M / b_e d^2 = 8.5 \times 10^6 / (395 \times 115^2)$$
$$= 1.63$$

$$z = 0.946d$$

$$A_s = 8.5 \times 10^6 / (0.87 \times 460 \times 0.946 \times 115)$$
$$= 195 \text{ mm}^2$$
$$= 494 \text{ mm}^2\text{/m}$$

For practical reasons, the bars are to be uniformly placed in the nib with T10-100 c/c (A_s = 785 mm^2/m)

2. Check bearing stress within bend

Tensile force per bar

$$F_s = 0.87 f_y A_s \times (A_{s,req} / A_{s,pro})$$
$$= 0.87 \times 460 \times 78 \times (494/785) \times 10^{-3}$$
$$= 19.9 \text{ kN}$$

Minimum bending radius :

$$r = \frac{F_s}{\phi} \times (1 + \frac{2\,\phi}{a_b})/2f_{cu}$$

$a_{b,}$ = 100mm

$r = (19.9 \times 10^3 / 10) \times (1 + 2 \times 10/100) / (2 \times 35)$

= 34.1 mm

use $r = 4\,\phi$

3. Vertical links within beams

Vertical links area $A_{sv} = V / (0.87 f_y)$

$= 50 \times 10^3 / (0.87 \times 460)$

$= 125\ mm^2$

$= 104\ mm^2/m$

The required steel links are in addition to any other links resisting shear forces in the beam.

4. Check shear stress

b_e = 395 mm

$\nu = V/b_e d$

$= 50 \times 10^3/(395 \times 115)$

$= 1.10\ N/mm^2$

r_s = 0.68% (T10-100 c/c)

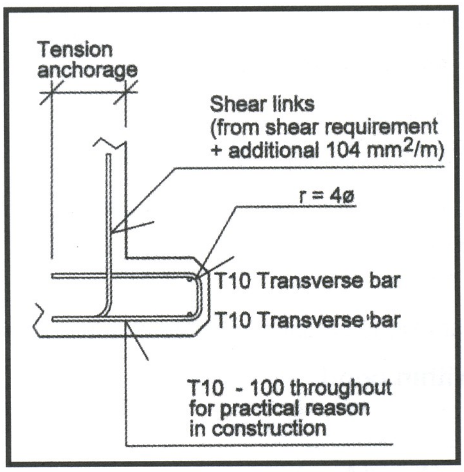
$\nu_c = 0.85 N/mm^2$

Enhanced $\nu_c' = \nu_c \times 2d/a_v$

$= 0.85 \times 2 \times 115 / 170$

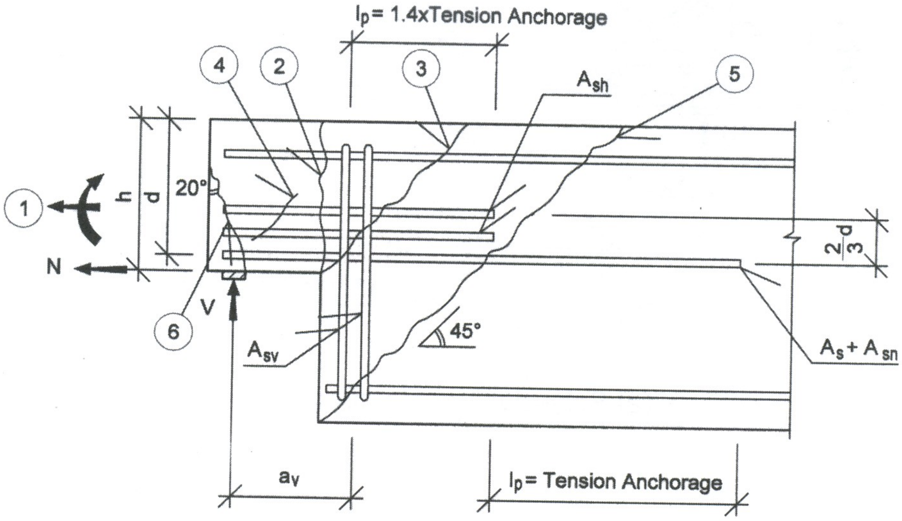
$= 1.15 N/mm^2 > 1.10 mm^2$ OK

5. Detailing

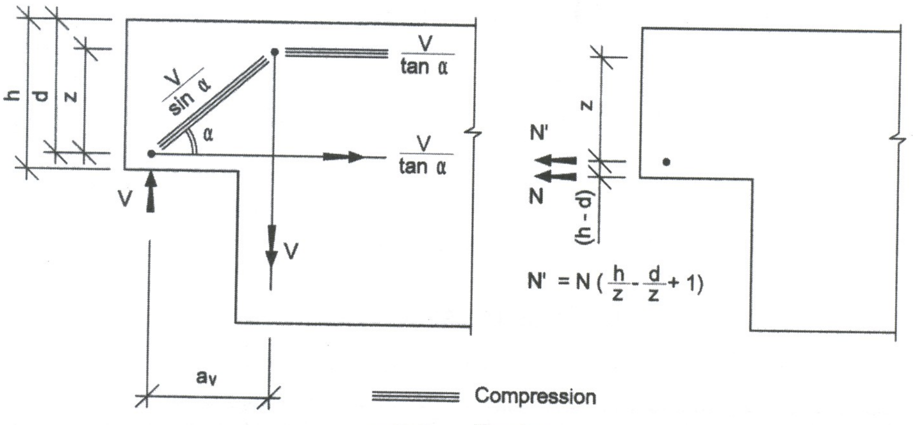


3.12 Beam Half Joint

The design of beam half joints involves the investigation of several potential crack planes which are illustrated in Figures 3.15 and 3.16 respectively.



(a) Schematic reinforcement

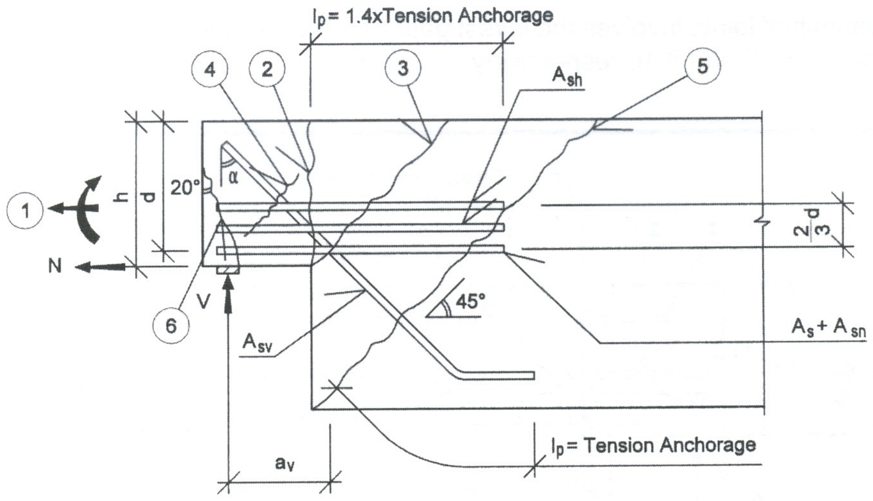


(i) Vertical force

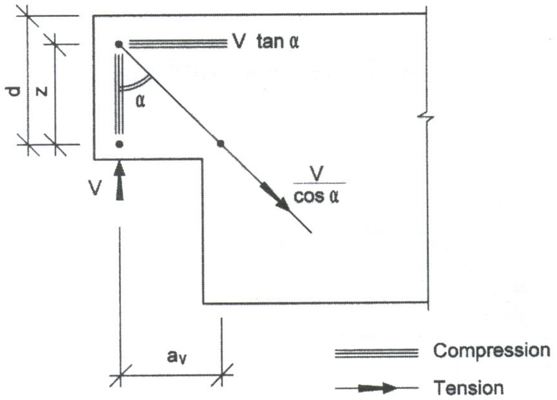
(ii) Horizontal force

(b) Force resisting system

Figure 3.15 Reinforcement And Force Resisting System In Beam Half Joint



(a) Schematic reinforcement



(i) Vertical force

(b) Force resisting system

Note : For horizontal force, refer to Figure 3.15 (b)

Figure 3.16 Alternative Reinforcement And Force Resisting System In Beam Half Joint

In Figures 3.15 and 3.16, the reinforcement for each of the crack plane considered is listed as below :

1. Flexural (cantilever bending) and axial tension at the extended end. Provide reinforcement A_s (flexure) and A_{sn} (axial tension).

2. Direct shear at joint junction with main member. Provide shear friction reinforcement A_s and A_{sh}.

3. Diagonal tension at re-entrant corner. Provide shear friction reinforcement A_{sv}.

4. Diagonal tension in half-joint. Provide reinforcement A_{sh}.

5. Diagonal tension in main member. Provide A_s and A_{sv} with full tension anchorage beyond the potential crack plane.

6. Inclined shear crack at beam half-joint bearing. Provide A_s and A_{sn}. Refer Section 3.9 when investigating potential horizontal crack at the bearing, although it is generally not critical.

The reinforcement as determined from (1) to (6) above is not cumulative and should be :

A_s, the greater of (1), (2), (5) or (6)
A_{sh}, the greater of (2) or (4)
A_{sv}, the greater of (3) or (5)

The determination of reinforcement in the beam half joint is based on the shear friction design method. The reinforcement across each of the potential cracks is calculated as follows :

3.12.1 Reinforcement as in Figure 3.15

1. Flexural and axial tension steel

Flexural steel : $A_s = V(a_v / z) / (0.87f_y)$ —— (3.28)

Axial tension steel : $A_{sn} = N(h / z - d / z + 1) / (0.87f_y)$ —— (3.29)

Assuming $z = 0.8d$, then

$$A_s = 1.25V (a_v / d) / (0.87f_y) \qquad \text{—— (3.30)}$$

$$A_{sn} = 1.25 N (h / d - 0.2) / (0.87f_y) \qquad \text{—— (3.31)}$$

2. Direct shear at joint junction

$$A_s = \frac{2}{3} \text{ x } V / (0.87f_y\mu) \qquad \text{—— (3.32)}$$

$$A_{sh} = \frac{1}{3} \text{ x } V / (0.87f_y\mu) \qquad \text{—— (3.33)}$$

A_{sn} is as in equation 3.29.

A_{sh} should be uniformly distributed within 2/3d of the half joint depth.

The maximum shear stress at the joint junction is determined as

$$\nu = V/bd \leq \nu_c$$

where $\nu_c = 0.8 \sqrt{f_{cu}}$ or 5.0 N/mm^2, whichever is smaller.

3. Diagonal tension at re-entrant corner

$$A_{sv} = V/0.87f_{yv} \qquad \text{—— (3.34)}$$

Based on shear friction design method, A_{sv} should be

$$A_{sv} = V/0.87f_{yv}\mu$$

Due to high stress concentration at the re-entrant corner, $\mu = 1.0$ should be conservatively adopted at re-entrant corner crack plane.

4. Bearing at half joint

Refer to Section 3.9 for the design of shear friction reinforcement at the half joint bearing.

3.12.2. Reinforcement as in Figure 3.16

The alternative reinforcement in the force resisting system in Figure 3.16 is calculated as follow:

1. **Axial tension steel**

   Assuming $z = 0.8d$

   $A_{sn} = 1.25N (h / d - 0.2) / (0.87f_y)$ as in equation 3.31

2. **Diagonal tension at re-entrant corner**

   Adopting $\mu = 1.0$

   $A_{sv} = V / (0.87f_{yv} \cos \alpha)$

   where α is the angle between the diagonal tension and V.

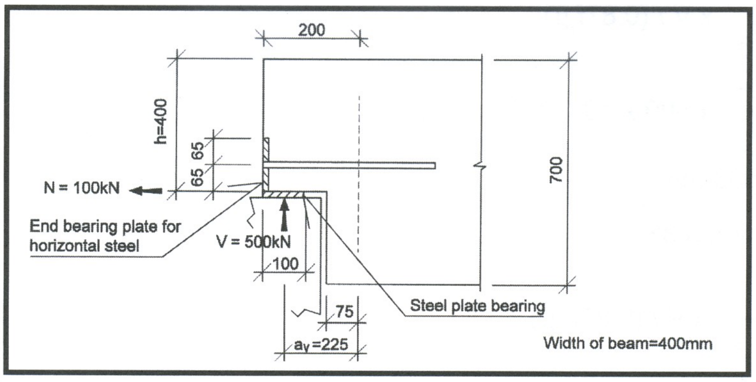
$$\cos \alpha = \frac{z}{\sqrt{a_v^2 + z^2}}$$

Assuming $z = 0.8d$

$$A_{sv} = \frac{1.25V}{0.87f_{yv}d} \sqrt{a_v^2 + (0.8d)^2} \qquad \text{—— (3.35)}$$

Design Example 5: Beam Half Joint

Design the beam half joint shown in the figure below for an ultimate vertical reaction V = 500 kN and an ultimate horizontal tension of N = 100 kN. Design concrete cube strength is f_{cu} = 35 N/mm^2 and f_y = 460 N/mm^2 for all steel.



General design data for half-joint

h = 400 mm
b = 400 mm
d = 400 - 65 = 335 mm
z = 0.8d = 268mm
a_v = 200 + 75 - 50 = 225 mm
V = 500 kN
N = 100 kN
f_{cu} = 35 N/mm^2
f_y = 460 N/mm^2
μ = 1.0

1. Reinforcement as in Figure 3.15

a. Flexural and tension steel

From equation 3.30

$$A_s = 1.25V(a_v / d) / (0.87f_y)$$
$$= 1.25 \times 500 \times 10^3 \times (225 / 335) / (0.87 \times 460)$$
$$= 1049 \text{ mm}^2$$

From equation 3.31

$$A_{sn} = 1.25N\,(h / d - 0.2) / (0.87f_y)$$
$$= 1.25 \times 100 \times 10^3 \times (400 / 335 - 0.2) / (0.87 \times 460)$$
$$= 310 \text{ mm}^2$$

$A_s + A_{sn}$ = 1359 mm^2

b. Direct shear at joint junction

From equation 3.32

$$A_s = \frac{2}{3} \times V / (0.87 f_y \mu)$$

$$= \frac{2}{3} \times 500 \times 10^3 / (0.87 \times 460)$$

$$= 833 \text{ mm}^2$$

From equation 3.33

$$A_{sh} = \frac{1}{3} \times V / (0.87 f_{yv} \mu)$$

$$= \frac{1}{3} \times 500 \times 10^3 / (0.87 \times 460)$$

$$= 417 \text{ mm}^2$$

c. Check shear stress at joint junction

$$\nu = 500 \times 10^3 / (400 \times 335)$$

$$= 3.73 < 0.8\sqrt{f_{cu}} = 4.73 \text{ N/mm}^2$$ O K

d. Diagonal tension at re-entrant corner

From equation 3.34

$$A_{sv} = V / 0.87 f_{yv}$$

$$= 500 \times 10^3 / (0.87 \times 460)$$

$$= 1249 \text{ mm}^2$$

e. Bearing at half-joint

Refer to Design Example 1 where $A_s + A_{sn} = 1022 \text{ mm}^2$

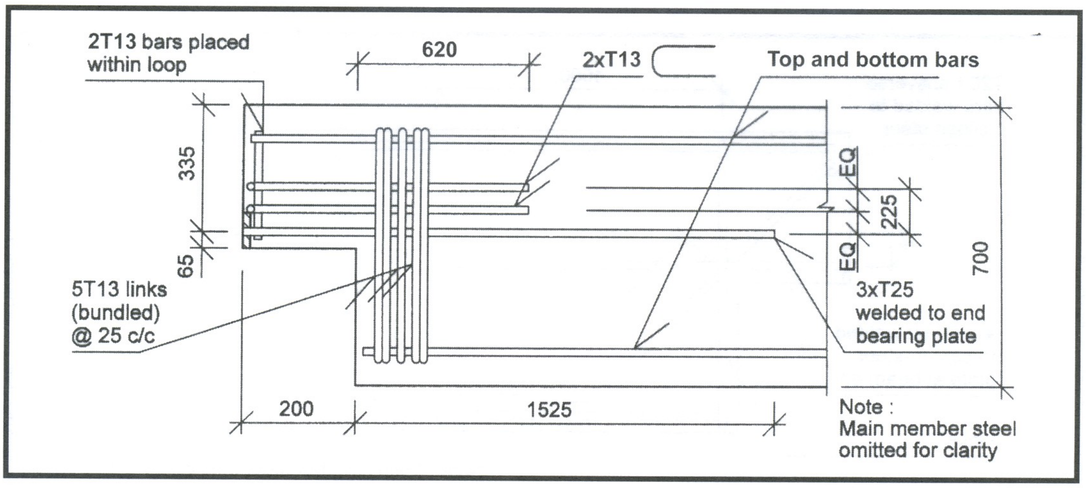
f. From (a) to (e) the greater value of A_s, A_{sn}, A_{sv}, and A_{sh} is adopted

i. $A_s + A_{sn} = 1359 \text{ mm}^2$
Provide 3T25 welded to bearing plate at beams end ($A_s = 1473 \text{ mm}^2$)

ii. $A_{sh} = 417 \text{ mm}^2$
Provide 2 numbers of T13 (4 legs = 531 mm^2)

iii. $A_{sv} = 1249 \text{ mm}^2$
Provide 5T13 (10 legs = 1327 mm^2)

g. Detailing



2. Alternative reinforcement as in Figure 3.16

a. Axial tension steel

From equation 3.31

$$\begin{aligned} A_{sn} &= 1.25N\,(h/d - 0.2)/(0.87f_y) \\ &= 1.25 \times 100 \times 10^3 \times (400/335 - 0.2)/(0.87 \times 460) \\ &= 310\ \text{mm}^2 \end{aligned}$$

b. Diagonal tension at re-entrant corner

From equation 3.35

$$A_{sv} = \frac{1.25V}{0.87f_{yv}d}\sqrt{a_v^2 + (0.8d)^2}$$

$$= \frac{1.25 \times 500 \times 10^3}{0.87 \times 460 \times 335}\sqrt{225^2 + (0.8 \times 335)^2}$$

$$= 1631\ \text{mm}^2$$

c. Bearing at half joint

As in Design Example 1, $A_s + A_{sn} = 1022\ \text{mm}^2$

d. Adopted steel area

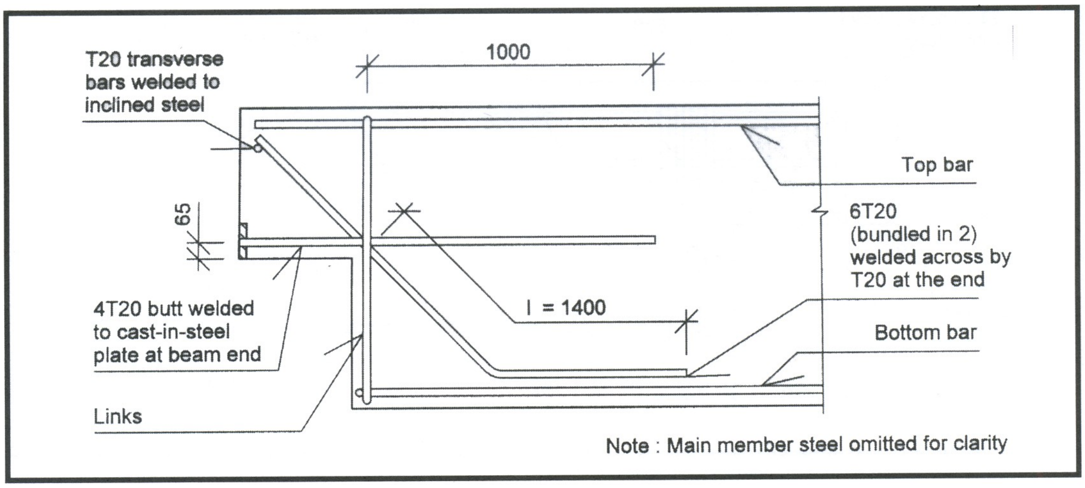
i. $A_s + A_{sn} = 1022\ \text{mm}^2$

Use 4T20 butt welded to cast-in steel plate at beam end. (Area = 1257mm^2)

ii. $A_{sv} = 1631\ \text{mm}^2$

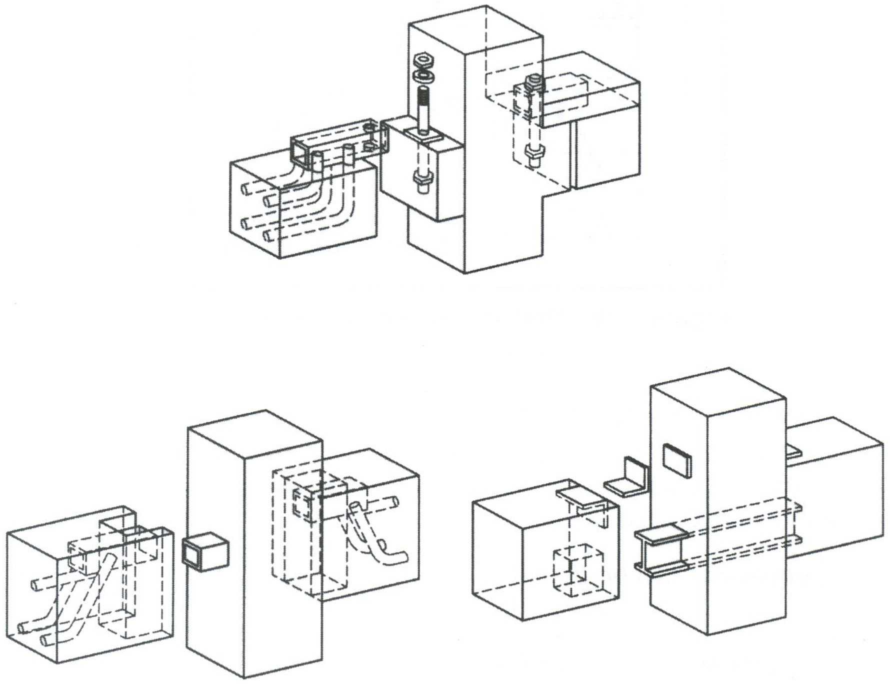
Use 6T20 welded with T20 transverse bar at the beam top to ensure effective anchorage.

e. Detailing

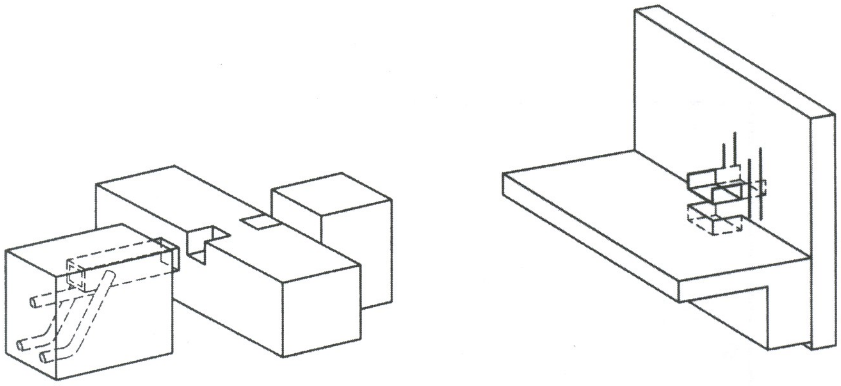

T20 transverse bars welded to inclined steel
1000
Top bar
65
6T20 (bundled in 2) welded across by T20 at the end
l = 1400
4T20 butt welded to cast-in-steel plate at beam end
Bottom bar
Links
Note : Main member steel omitted for clarity

3.13 Steel Sections Inserts

Connections between precast units may be made using embedded structural steel sections to form a simple bearing or bolted connection as illustrated in Figure 3.17. Minimum width of the steel inserts should preferably be 75 mm so that in the event of failure, it will be concrete compression rather than splitting failure. I-sections, channels, angles or hollow sections are commonly used in such connections.



(a) Beam to column connection



(b) Beam to beam connection

Figure 3.17 Examples Of Structural Steel Insert Connection

3.13.1. Steel inserts cast in column

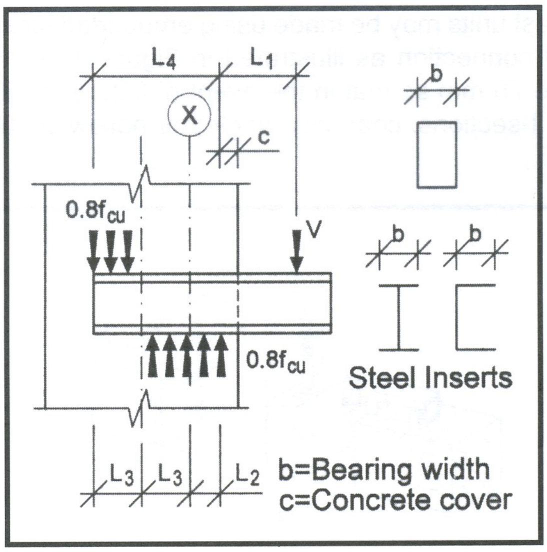


Figure 3.18 Distribution Of Forces In Column Inserts

Figure 3.18 shows a steel insert in column loaded on one side only with an embedded length L_4, and the distance between the applied load V and the effective bearing edge being L_1. The distribution length of concrete bearing stresses is defined as over L_2 and L_3. The ultimate concrete bearing stress is $0.8f_{cu}$ (clause 5.2.3.4, Part 1).

The following equations can be derived :

By geometry : $2L_3 + L_2 = L_4$ —— (3.36)

Resolve forces vertically : $V = 0.8f_{cu}L_2 b$ —— (3.37)

Taking moment about point x : $0.8f_{cu}b(L_3^2+0.5L_2^2)=(L_1+L_2)V$ —— (3.38)

Combining the above equations and after rearranging,

$$V = 0.8f_{cu}b\alpha L_4 \qquad \text{—— (3.39)}$$

where $\alpha = [(1+2L_1/L_4)^2 + 1]^{1/2} - 2L_1/L_4 - 1$ —— (3.40)

The variations of α with L_1/L_4 are shown in Figure 3.19 below.

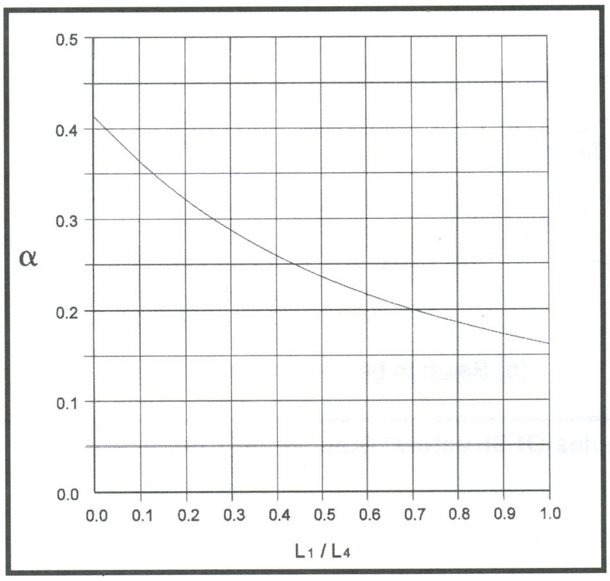
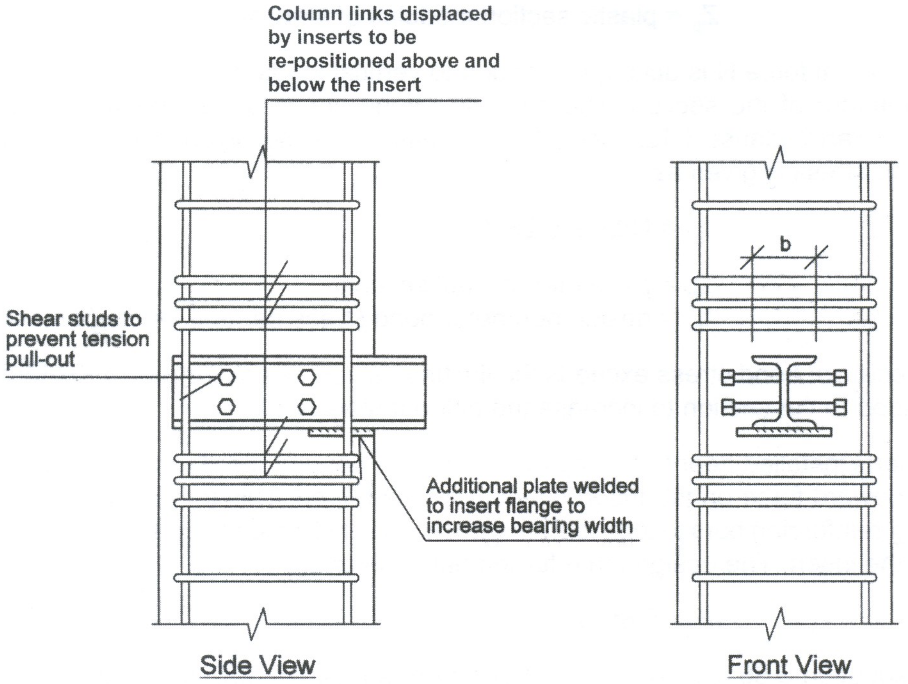


Figure 3.19 Variations Of α With L_1/L_4

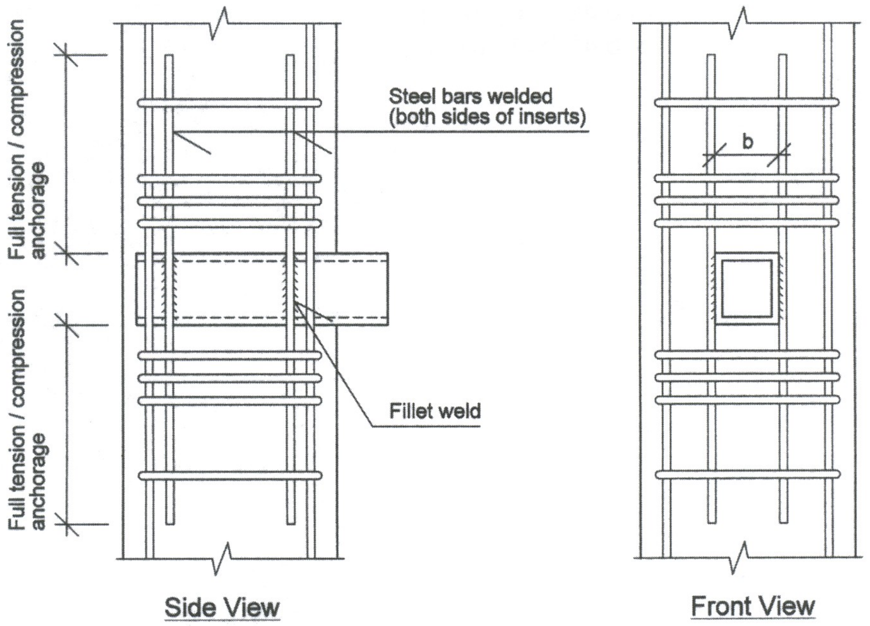
The capacity of the inserts may be increased as shown in Figure 3.20 by:

a. welding additional flanges to the inserts to increase the effective bearing width, and

b. welding vertical reinforcing bars to the steel section



(a) Welding of flange to inserts



(b) Welding of vertical bars to inserts

Figure 3.20 Increasing The Capacity Of Steel Inserts

The steel section is determined by

Flexural strength, $Z_p = M/p_y$ —— (3.41)
Shear strength, $V = 0.6dt\ p_y$ —— (3.42)
$M = V \times L_1$

where
p_y = yield strength of steel section
d, t = depth and thickness of steel web respectively
Z_p = plastic section modulus of steel section

If a horizontal force N is acting, in addition to vertical loads, the force is resisted by bond on the perimeter of the section. The perimeter bond stress should be within the permissible values in Part 1, clause 3.12.8 of the Code. Treating steel section as mild steel (under tension), the bond stress is given as:

$$f_b = N/\Sigma p \leq 0.28\sqrt{f_{cu}} \qquad \text{—— (3.43)}$$

where
Σp = perimeter of steel section
f_b = design perimeter bond stress per unit length of steel section

If the resultant bond stress exceeds the limiting values, headed studs or reinforcing bars can be welded to the section to increase the pull-out resistance.

For inserts installed near to the top of column, the bearing resistance of the column concrete to the upward force on the insert cannot be relied upon. This force is to be fully resisted by welding reinforcing bars to the steel inserts with full anchorage length into the column concrete below the insert. The design force for the bar is given by:

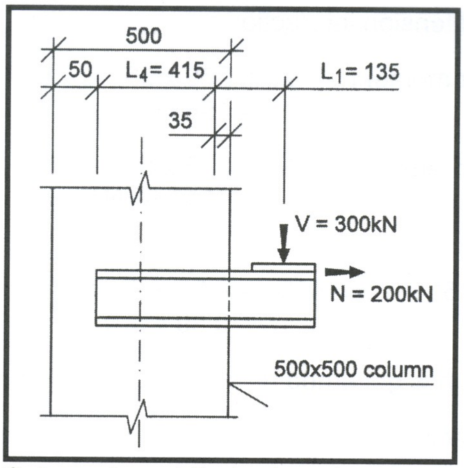
$$F_s = 0.8f_{cu}bL_3$$

From equation 3.36, $L_3 = 0.5\ (L_4 - L_2)$ and from equations 3.37 and 3.39, $L_2 = \alpha\ L_4$. Making these substitutions to the above expression:

$$\begin{aligned} F_s &= 0.8f_{cu}b \times 0.5\ (L_4 - L_2) \\ &= 0.4f_{cu}b\ (L_4 - \alpha\ L_4) \\ &= 0.4f_{cu}b\ (1 - \alpha)\ L_4 \qquad \text{—— (3.44)} \end{aligned}$$

Design Example 6: Steel Insert Cast In Columns

Design a column steel insert to support a vertical ultimate load of 300 kN. The column inserts are to be within a 500 x 500mm square column with f_{cu} = 35 N/mm^2. Also check the adequacy of the insert design if an ultimate horizontal force of 200 kN acts at the support. Concrete cover = 35 mm and yield strength of steel section p_y = 275N/mm^2.



Column Insert Embedment

1. Effective bearing width of insert

L_1 = 135 mm
L_4 = 415 mm
L_1/L_4 = 0.325

From equation 3.40 or from Figure 3.19, α = 0.279

Hence minimum effective bearing width of insert required from equation 3.39 is

$$\begin{aligned} b &= V/0.8f_{cu}\alpha\, L_4 \\ &= 300 \times 10^3/(0.8 \times 35 \times 0.279 \times 415) \\ &= 92.5 \text{ mm} \end{aligned}$$

2. Steel section for insert

Moment M = 300 x 0.135
= 40.5 kNm
Shear V = 300 kN
Horizontal Force, N = 200 kN

a. Plastic section modulus required

$$\begin{aligned} Z_p &= M/p_y \\ &= 40.5 \times 10^6/(275 \times 10^3) \\ &= 147 \text{ cm}^3 \end{aligned}$$

O K

b. Minimum d x t required

$$\begin{aligned} d \times t &= V/0.6p_y \\ &= 300 \times 10^3/(0.6 \times 275) \\ &= 1818 \text{ mm}^2 \end{aligned}$$

c. Minimum steel area required under tension

$$\begin{aligned} \text{Area} &= N/p_y \\ &= 200 \times 10^3/(275 \times 10^2) \\ &= 7.3 \text{ cm}^2 \end{aligned}$$

Try 254 x 146 x 43 kg/m I-section

$Z_p = 567.4\ cm^3 > 147\ cm^3$ OK
$A = 55\ cm^2 > 7.3\ cm^2$ OK
$d = 260$ mm, $t = 7.3$ mm
$d \times t = 1898\ mm^2 > 1818\ mm^2$ OK

d. Check moment and tension interaction :

Section capacity for moment M $= 275 \times 567.4 \times 10^{-3}$
$= 156$ kNm

Section capacity for tension N $= 275 \times 55 \times 10^{-1}$
$= 1512.5$ kN

Hence interaction
$40.5/156 + 200/1513 = 0.26 + 0.13$
$= 0.39 < 1.0$ OK

Column insert design is governed by shear. Section is adequate.

e. Check bearing capacity :

$b = 147\ mm > 95.2\ mm$ OK

Max. bearing capacity of insert

$V = 0.8\ f_{cu}\ b\ \alpha\ L_4$
$= 0.8 \times 35 \times 147 \times 0.279 \times 415 \times 10^{-3}$
$= 477\ kN > 300\ kN$ OK

f. Check bond stresses

Total perimeter of steel section is about 1095 mm

Total contact area between steel section and concrete $= 1095 \times 415$
$= 454425\ mm^2$

Average bond stress $= 200 \times 10^3 / 454425$
$= 0.44\ N/mm^2$

Permissible ultimate bond stress

$f_b = 0.28\sqrt{f_{cu}}$
$= 0.28\sqrt{35}$
$= 1.66\ N/mm^2 > 0.44\ N/mm^2$ OK

3.13.2 Steel inserts cast in beam

Steel section inserts used in beam connection may generally consist of:

a. wide flange section
b. steel plate with welded bearing flanges, and
c. exposed section on beam top;
as illustrated in Figure 3.21 below.

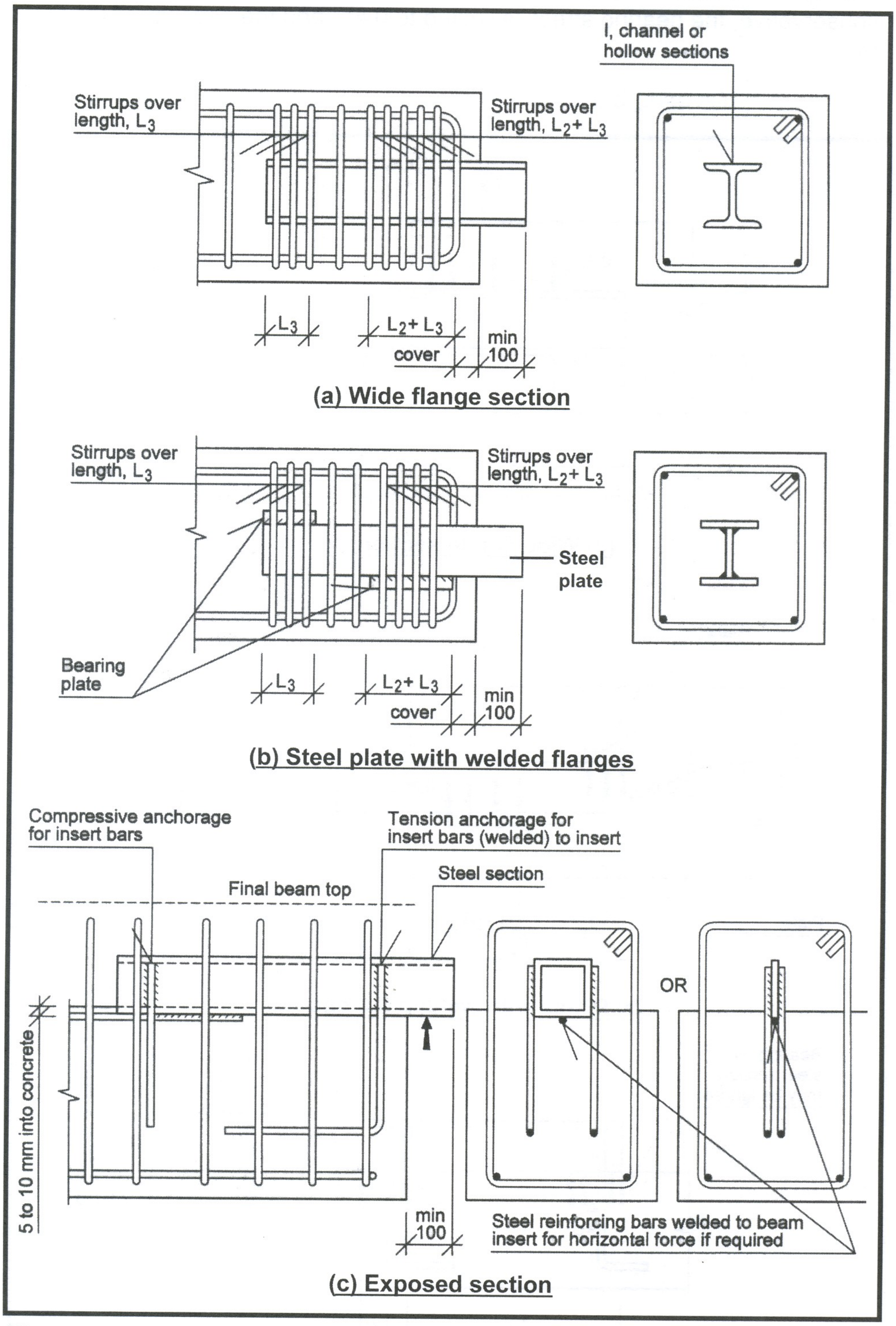


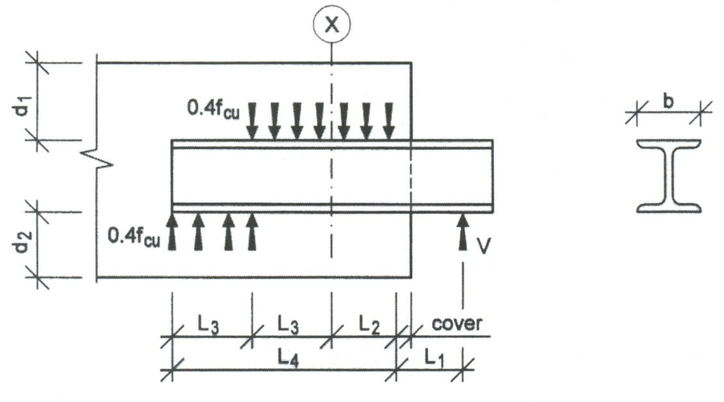
Figure 3.21 Steel Inserts Cast In Beams

Distribution of forces in the inserts is illustrated in Figure 3.22 (a) to (c) and the method of analysis may be as follows :

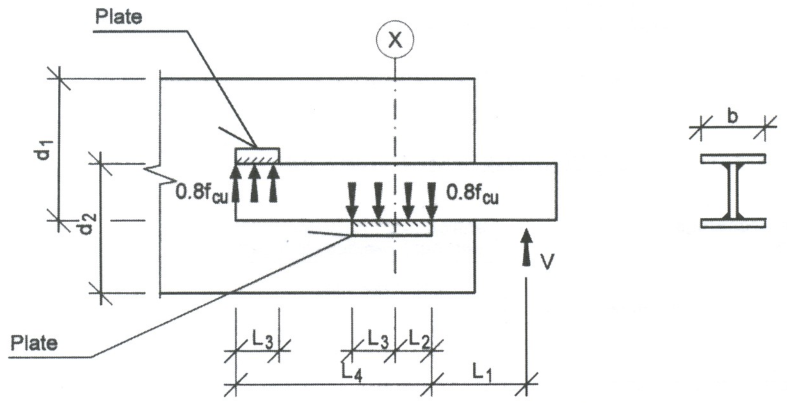
a. **Wide flange section (Figure 3.22a)**

The design and force distribution of wide flange section are similar to column inserts as shown in Figure 3.18. However, unlike column inserts, there is less depth (d_1 and d_2) of concrete above and below the insert to resist the bearing pressure. To be conservative, the bearing stress is limited to $0.4f_{cu}$ and the capacity of the insert is given by:

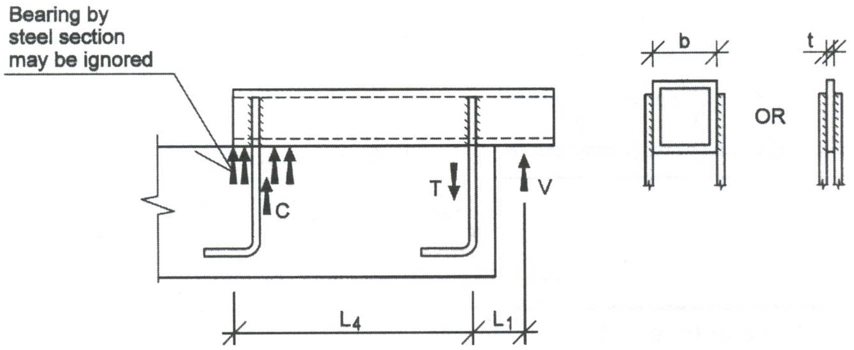
$$V = 0.4f_{cu}\, b\, \alpha\, L_4 \qquad (3.45)$$



(a) Wide flange section



(b) Plates with welded flanges



(c) Exposed section

Figure 3.22 Distribution Of Forces In Beam Inserts

The designer should further ensure that:

i. the breadth and depth of the steel inserts should not exceed 1/3 of the respective breadth and depth of the concrete beam.

ii. reinforcing bars are provided at both the inner and outer bearing, in order to resist fully the bearing forces generated. The provision of steel area may be detemined as in plate with welded flanges insert below.

If horizontal forces are present, the bond stress around the section perimeter should not exceed $0.28\sqrt{f_{cu}}$ as in the column insert design.

b. Plate with welded flanges (Figure 3.22b)

The method of analysis of plate with welded flanges insert may be as below:

i. Resolve the forces vertically

$$V = 0.8f_{cu}\, b\, L_2 \qquad \text{—— (3.46)}$$

ii. Taking moment about X

$$V(L_1+L_2) = 0.8f_{cu}b[(L_4 - L_2 - L_3)L_3 + 0.5L_2^2] \qquad \text{—— (3.47)}$$

From the above equations, L_4 can be calculated from :

$$(L_4 - L_2 - L_3)L_3 = L_2(L_1 + 0.5L_2) \qquad \text{—— (3.48)}$$

A hanger system consisting of steel stirrups is to be provided at the inner and outer bearing plates. The area of the steel required may be :

Inner bearing plate: $A_{sv} = 0.8bf_{cu}L_3/0.87f_y$ —— (3.49)

Outer bearing plate: $A_{sv} = 0.8bf_{cu}(L_2 + L_3)/0.87f_y$ —— (3.50)

It is important to ensure that the main tension steel in the beam is fully anchored at the beam ends to prevent shear tension or shear bond failures at the beam support.

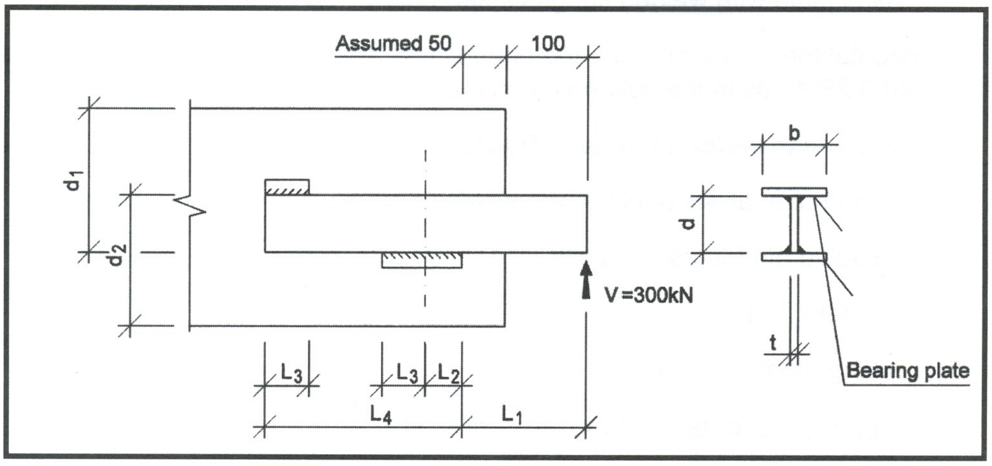
The dimensions of d_1 and d_2 should be proportional to the respective bearing forces i.e.

$$d_1/d_2 = (L_2 + L_3)/L_3 \qquad \text{—— (3.51)}$$

If horizontal tension forces are present, the bond stress must be checked as described earlier.

Design Example 7: Steel Plate Insert Cast In Beams

Design the steel plate with bearing flanges as an insert to support a vertical ultimate reaction from the beam of 300 kN acting at 100 mm from the beam end. The beam size is given as 300 x 600 mm deep and concrete strength f_{cu} = 30 N/mm^2. The ultimate yield strength of the steel plate is p_y = 275 N/mm^2. Also determine the appropriate position of the insert within the beam depth.



1. Insert geometry

Assuming bearing plate width b $\approx$ 1/3 beam width
= 100 mm

For equation 3.46

$$V = 0.8 f_{cu} b \, L_2$$

$$\begin{aligned} L_2 &= 300 \times 10^3/(0.8 \times 30 \times 100) \\ &= 125\text{mm} \end{aligned}$$

Taking L_1 = 150 mm, L_2 = 125 mm and L_3 = 125 mm as trial. Then from equation 3.48 :

$$\begin{aligned} (L_4 - L_2 - L_3)L_3 &= L_2(L_1 + 0.5L_2) \\ (L_4 - 125 - 125)125 &= 125(150 + 0.5 \times 125) \\ L_4 &= 462.5\text{mm} \end{aligned}$$

Check $2 \times L_3 + L_2 \leq L_4$

$$\begin{aligned} & 2 \times 125 + 125 \\ &= 375\text{mm} < 462.5\text{mm} \end{aligned}$$

O.K

Dimensions assumed in the insert are acceptable.

2. Design of steel plate

a. Shear

Required section for shear strength

$$\begin{aligned} d \times t &= V/0.6p_y \\ &= 300 \times 10^3 / (0.6 \times 275) \\ &= 1818\text{mm}^2 \end{aligned}$$

$$d = 1818 / t$$

b. Bending

Ultimate bending moment, M $= V \times L_1$
$= 300 \times 0.15$
$= 45$ kNm

Plastic section modulus required Z_p $= M/p_y$
$= 45 \times 10^6/(275 \times 10^{-3})$
$= 163.6\ \text{cm}^3$

Using 2 numbers of t = 15 mm thick plates

Required minimum d $= \sqrt{[163.6 \times 10^3 \times 4/(2 \times 15)]}$
$= 147.6$, say d = 150 mm

3. Bearing flanges design

Steel plate inserts are spaced at 70 mm.
Contact bearing pressure = $0.8f_{cu}$
Required bearing flange thickness

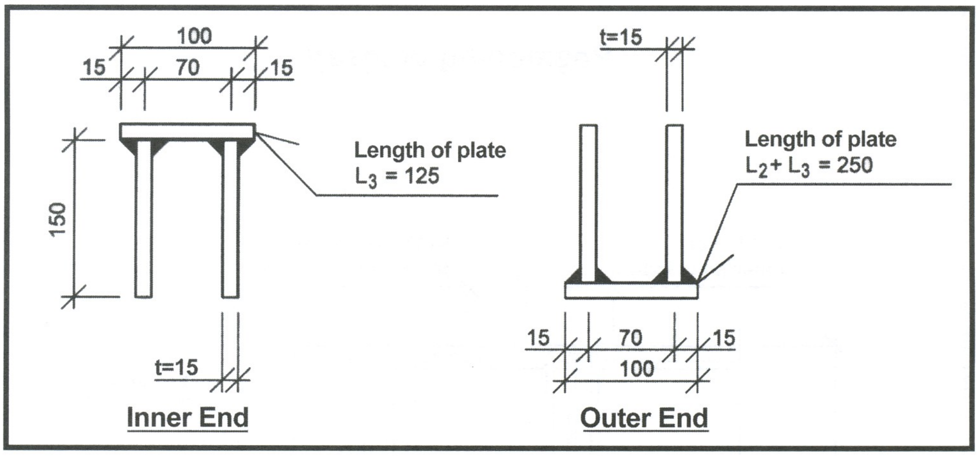
$(t^2/4) \times 275 = 0.8 \times 30 \times 70^2/8$
$t = 14.6$, say t = 15 mm thick

Check welding

Tension pull out force per mm run $= 0.8 \times 30 \times 50 \times 10^{-3}$
$= 1.20$ kN/mm

Use = 6 mm fillet weld
Weld strength $= 2 \times 6 \times 215 \times 10^{-3}/\sqrt{2}$
$= 1.82\ \text{kN/mm} > 1.20\ \text{kN/mm}$ OK

Steel plate insert = 2 numbers of 150 x 15 mm thick plates. The plates are as shown in the figure below.



4. Reinforcing bars to resist bearing forces

At inner plate : $A_s = 0.8f_{cu}\, b\, L_3/0.87f_y$
$= 0.8 \times 30 \times 100 \times 125/(0.87 \times 460)$
$= 750\ \text{mm}^2$
use 3T13 stirrups (6 legs) ($A_s = 796\ \text{mm}^2$)

At outer plate : $A_s = 0.8f_{cu} b(L_2 + L_3)/0.87f_y$
$= 0.8 \times 30 \times 100 \times (125 + 125)/(0.87 \times 460)$
$= 1500\text{mm}^2$
use 6T13 stirrups (12 legs) ($A_s = 1592\ \text{mm}^2$)

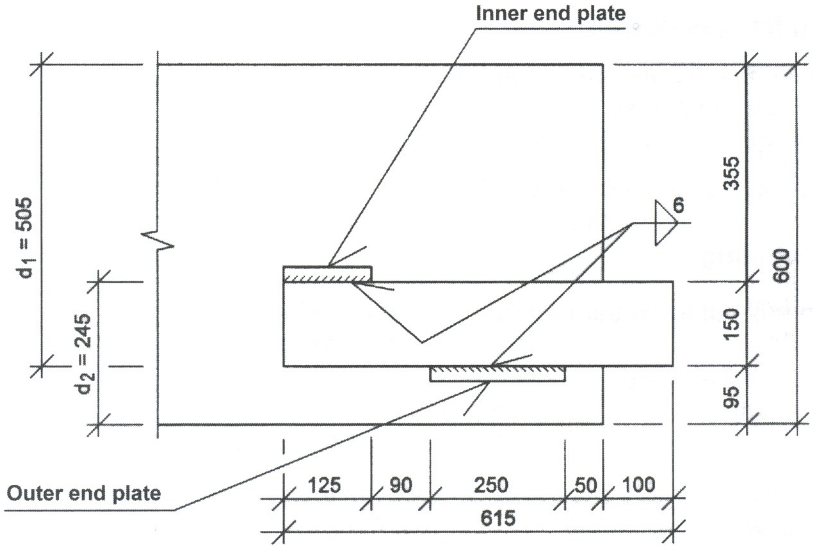
5. Position of insert

$$
\begin{aligned}
d_1/d_2 &\approx (L_2 + L_3)/L_3 \\
&= (125 + 125)/125 \\
&\approx 2
\end{aligned}
$$

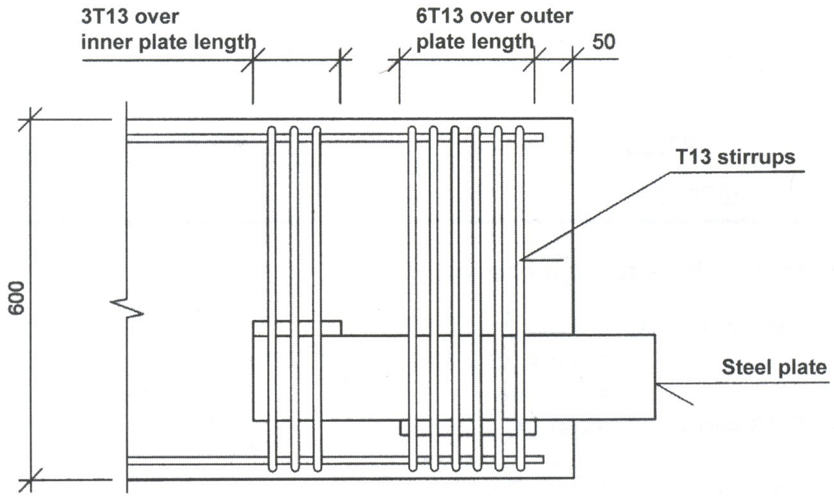
The final insert position is as shown in the detailing

$$
\begin{aligned}
d_1/d_2 &= 505/245 \\
&= 2.06
\end{aligned}
$$

6. Detailing



Positioning of insert



Links over bearing plates

3.13.3 Exposed sections

The exposed section inserts are commonly used in beam-to-beam connection when it is necessary to keep the structural beam shallow. As shown in Figure 3.21c, the exposed sections may consist of either wide flange sections or plates with reinforcing bars welded to the sides to provide the tension and compression reactions in a simple cantilever beam behaviour. The insert assembly will be embedded eventually in concrete.

The bearing pressure created at the far end of the insert is conservatively ignored, as it may be lost due to shrinkage, plastic cracking or surface grazing. There may also be partial or total loss of contact at the interface between the beam surface and insert as a result of fresh concrete settlement.

Referring to Figure 3.22c, the tension and compression reactions provided by the reinforcing bars may be obtained simply as :

Inner compression $C = V \times L_1/L_4$ —— (3.52)

Outer tension $T = V + C$ —— (3.53)

The reinforcing bars are calculated as :

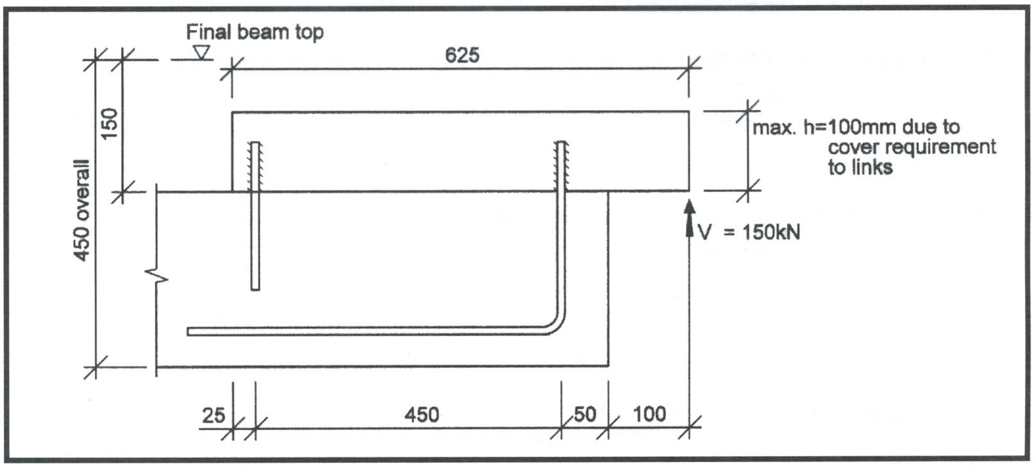
Tension and compression steel $A_s = T \text{ (or C)} / (0.87f_y)$

It is important to ensure that the welding is capable of developing the full strength of the reinforcing bars which are of full anchorage embedment length. The bearing stress within the bending radius of the bars has to be checked against local concrete bearing failure.

The choice of L_1 and L_4 is left entirely to the designer, bearing in mind that the larger the ratio of L_1/L_4, the heavier is the reinforcement needed in the steel insert.

Design Example 8: Exposed Steel Inserts Cast In Beam

Design an exposed steel insert at the ends of a precast beam subjected to an ultimate reaction of 150 kN. The semi-precast beam dimension is 300 x 300 and concrete strength is f_{cu} = 40 N/mm^2. The overall beam depth is restricted to 450 mm. The ultimate strength of the steel section p_y=275N/mm^2 and the characteristic strength of reinforcing bars f_y = 460N/mm^2



1. **Geometry of inserts**

Assuming a ratio of cantilever to internal span of 1:3 will result in a tension of 4/3 x V and a compression of 1/3 x V in the reinforcing bars. The geometry of the insert is shown in the above figure.

2. **Tension and compression forces**

$T = 4/3 \times V$
$= 200$ kN

$C = T - V$
$= 200 - 150$
$= 50$ kN

3. **Reinforcement design**

a. Bar in tension

Tension $T = 200$ kN

$A_s = T / 0.87f_y$ (f_y = 460 N/mm^2)
$= 500$ mm^2

Say 4 numbers of T13 with 2 bars welded to each side of the insert. (A_s=530mm^2)

Check tension anchorage

Tensile force per bar $F_s = 0.87f_yA_s$
$= 0.87 \times 460 \times 132 \times 10^{-3}$
$= 52.8$ kN

Tensile anchorage length required $l_p = F_s / (\pi\phi f_b)$

where $f_b = \beta\sqrt{f_{cu}}$
$= 0.5\sqrt{40}$
$= 3.16$ N/mm^2

$l_p = 52.8 \times 10^3 / (\pi \times 13 \times 3.16)$
= 409mm, say 450mm

The anchorage length of straight bars will exceed the overall beam depth of 300mm. Therefore, a bend will be provided in the tension bar.

Check bearing stress within the bend

Bearing stress $= \dfrac{F_s}{r\phi} \leq \dfrac{2f_{cu}}{1 + 2(\phi/a_b)}$

Minimum $r = \dfrac{F_s}{\phi} \times \dfrac{1 + 2(\phi/a_b)}{2f_{cu}}$

Assuming plate thickness of inserts = 35 mm

$a_b = 35 + \phi$
$= 48\text{mm}$

Hence minimum bending radius

$r = [52.8 \times 10^3/13] \times [1 + 2(13/48)] / (2 \times 40)$
$= 78$ mm

Use $r = 6\phi$ $(\phi=13)$
$= 78$ mm

b. Bar in Compression

Compression force in bar, C = 50 kN

$A_s = C/0.87f_y$
$= 50 \times 10^3/(0.87 \times 460)$
$= 125\ \text{mm}^2$

Use 2 numbers of T13, one on each side of insert. (A_s=265mm^2)

Check compression anchorage

Force per bar $F_s = 50/2$
$= 25$ kN

Compression anchorage required $f_b = 0.63\sqrt{f_{cu}}$
$= 3.98\ \text{N/mm}^2$

$l_b = 25 \times 10^3/(\pi \times 13 \times 3.98)$
$= 154$ mm

Provide l_b = 200 mm

4. Steel insert design

Ultimate bending moment $M = 0.15 \times V$
$= 0.15 \times 150$
$= 22.5$ kNm

Plastic section modulus required $Z_p = 22.5 \times 10^6/(275 \times 10^3)$
$= 81.8\ \text{cm}^3$

Restrict height of plate to 100 mm due to cover requirements to shear links.

Hence $t = 81.8 \times 10^3 \times 4/100^2$
$= 33$ mm, say t = 35 mm

Note: When t > 16mm thick, p_y should be 265N/mm^2. However, the adopted thickness of t=35mm remains valid in this case.

Check shear

Plate thickness under shear for d = 100

$d \times t = V/0.6p_y$

$t = 150 \times 10^3/(0.6 \times 275 \times 100)$
$= 9 \text{ mm} \leq 35\text{mm}$ OK

5. Welding design of steel bars

Maximum tension force = 52.8 kN per bar

Weld length available = 100mm

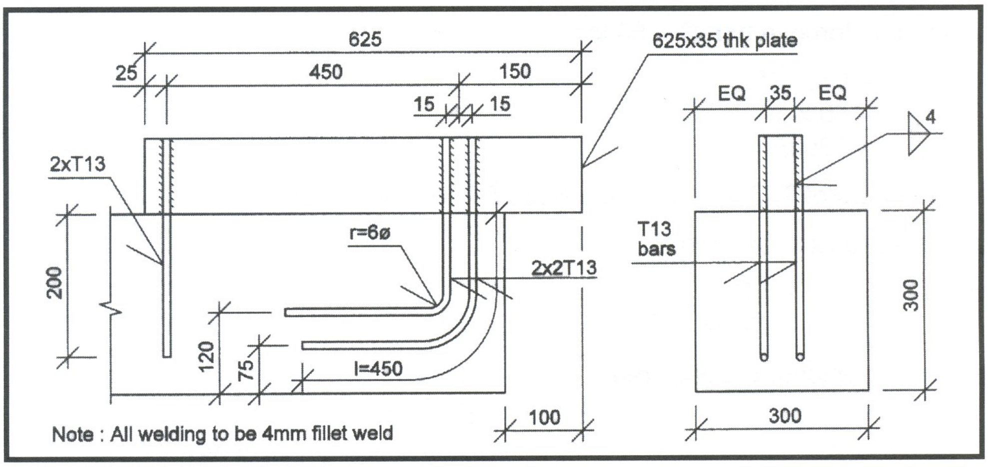
∴ Weld strength required = 52.8/100
= 0.528 kN/mm

Try 4 mm fillet weld on both sides of the bar.

Welding strength provided $= 0.707 \times 4 \times 215 \times 10^{-3} \times 2$
$= 1.22 \text{ kN/mm} > 0.528 \text{ kN/mm}$ OK

Provide continuous welding of 4 mm fillet weld on both sides of the bar.

6. Detailing



Note : All welding to be 4mm fillet weld

3.14 Force Transfer By Welding

Welding for the purpose of force transfer between reinforcing bars or between bars and other steel sections is permitted under Part 1, clause 7.6, of the Code. Provided the steels are weldable and suitable safeguards and techniques are employed, welding is a practical method to achieve the force transfer between connections in precast construction.

There are two types of welds, namely butt and fillet weld. Butt welds may be considered as strong as the parent steel as long as full penetration for the weld is achieved. The size of butt weld is specified by its throat thickness and is the smaller value of the two materials being joined. Figure 3.23 shows some typical butt welds between bars and other steel sections. Fillet welds are more commonly used as they do not require special surface preparation of the reinforcing bars or plates and are therefore cheaper than butt welds. Figure 3.24 shows some typical fillet welds between bars and other steel sections. The strength of fillet weld is given as:

$$\begin{aligned} P_w &= \text{(throat thickness)} \times \text{(unit length)} \times \text{(design shear stress)} \\ &= a_w \times p_w \\ &= 0.7 \times \text{leg length} \times p_w \\ &= 0.7 \times s \times p_w \qquad \text{—— (3.54)} \end{aligned}$$

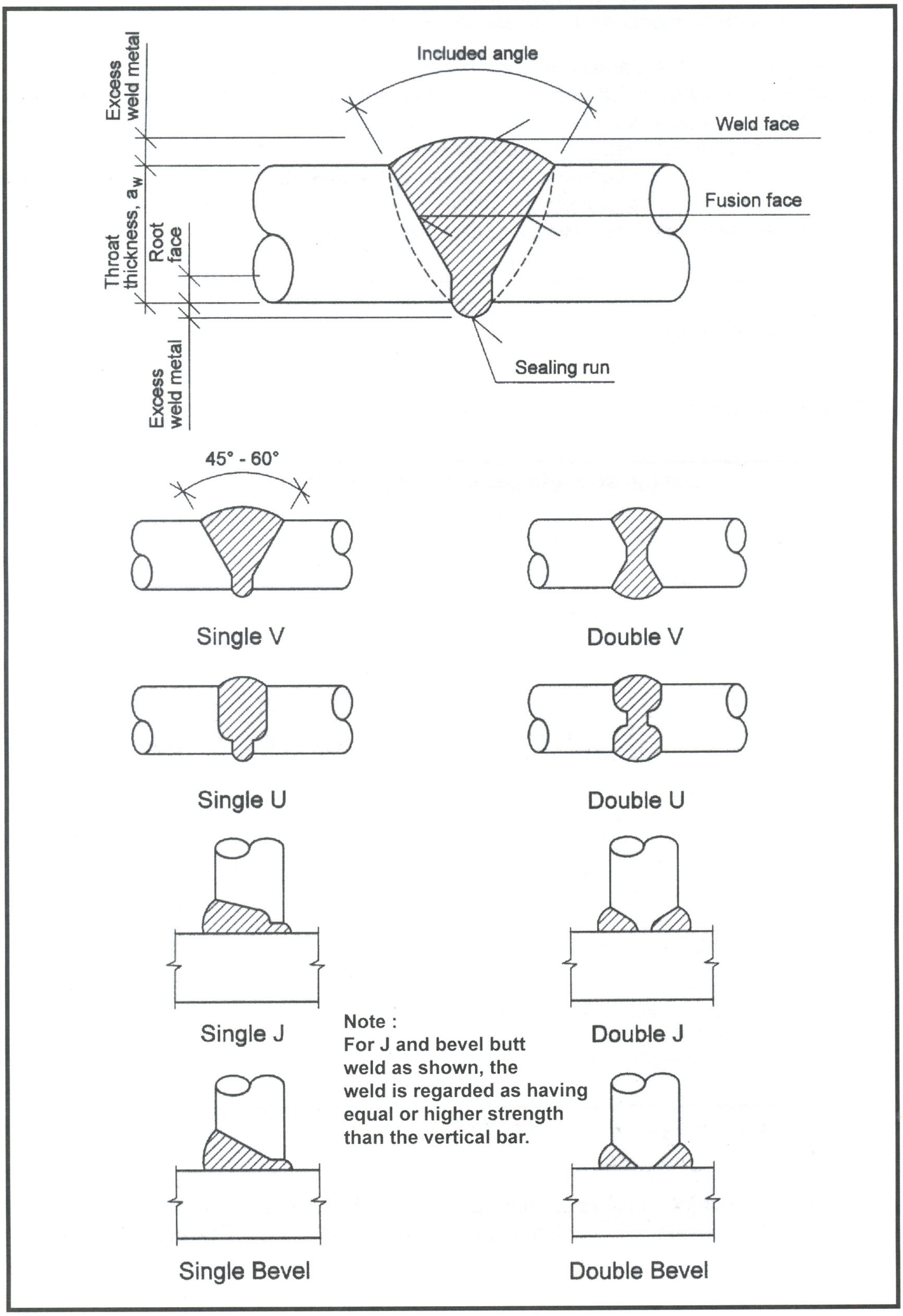
Table 3.3 shows the design strength for fillet welds.

	Design strength per unit length in kN/mm				
	Grade 43	Grade 50		Grade 55	
Leg length s (mm)	E43, E51	E43	E51	E51	E51*
	p_w = 215 N /mm^2		p_w = 255 N/mm^2		p_w = 275 N/mm^2
4	0.602		0.714		0.770
5	0.753		0.893		0.963
6	0.903		1.071		1.155
8	1.204		1.428		1.540
10	1.505		1.785		1.925
12	1.806		2.142		2.310
16	2.257		2.667		2.887
18	2.709		3.213		3.465
20	3.010		3.750		3.850
22	3.311		3.927		4.235
25	3.763		4.463		4.813

Note (1) E denotes electrode complying with BS639.
(2) Grade 43, 50, 55 complying with BS 4360.
(3) * only applies to electrodes having a minimum tensile strength of 550 N/mm^2 and a minimum yield strength of 450 N/mm^2.

Table: 3.3 Design Strength For Fillet Weld

For welding between reinforcing bars or reinforcing bars with steel sections, the reader may refer to guidelines on welding design from the latest edition of ANSI/AWS D1.4 - Structural Welding Code, Reinforcing Steel.



Single V

Double V

Single U

Double U

Single J

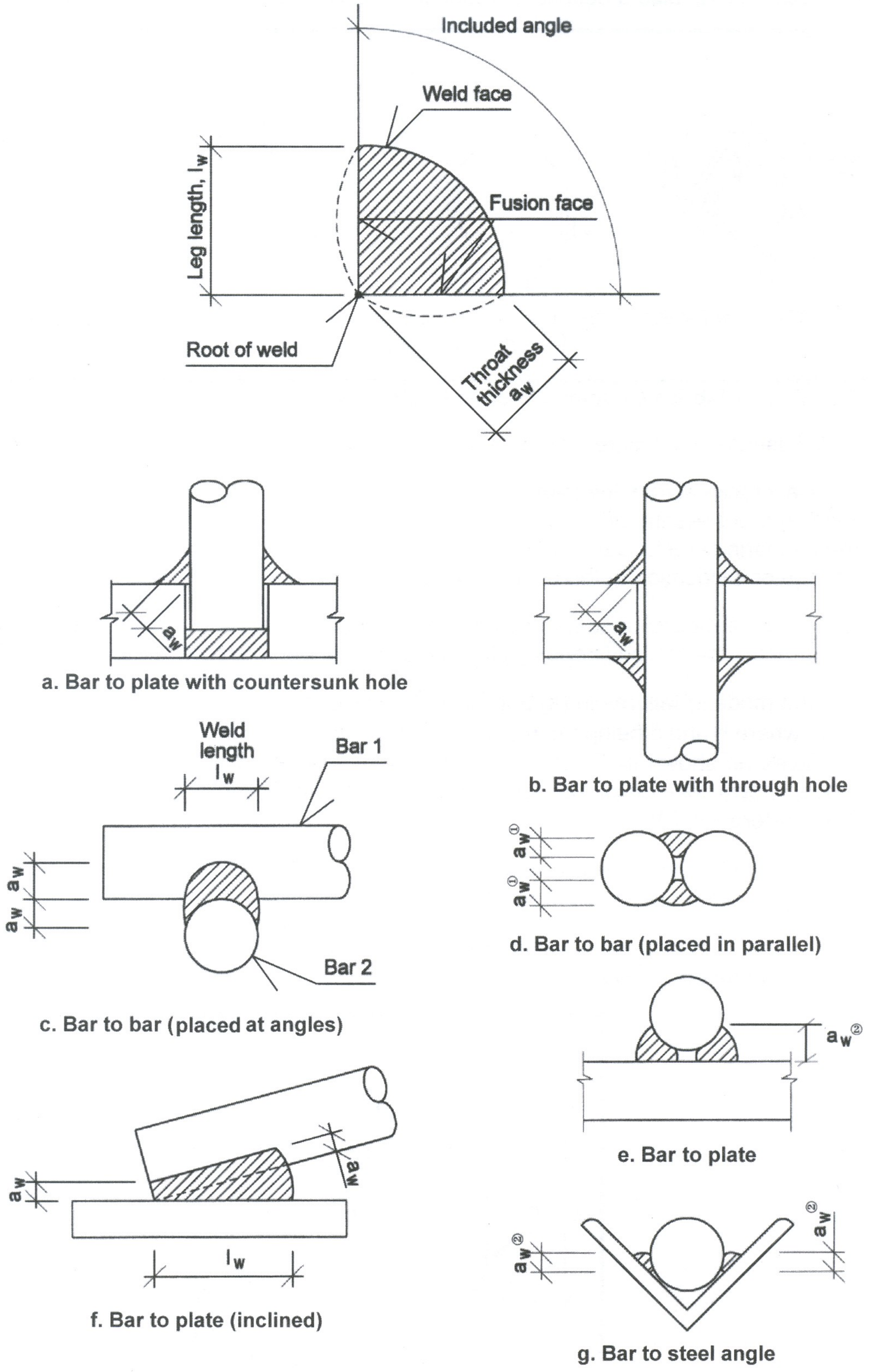
Double J

Single Bevel

Double Bevel

Note :
For J and bevel butt weld as shown, the weld is regarded as having equal or higher strength than the vertical bar.

Figure 3.23 Typical Butt Welding



a. Bar to plate with countersunk hole

b. Bar to plate with through hole

c. Bar to bar (placed at angles)

d. Bar to bar (placed in parallel)

e. Bar to plate

f. Bar to plate (inclined)

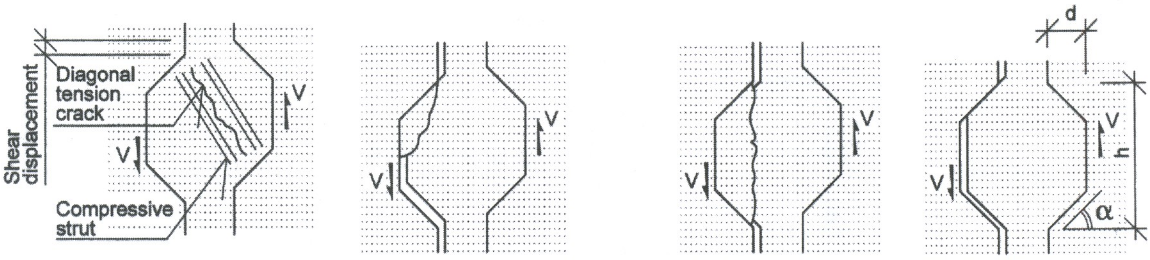
g. Bar to steel angle

Note: $a_w^{①} = 0.3ø$, $a_w^{\neq} = 0.2ø$ from ANSI/AWS D1.4 (1992)

Figure 3.24 Typical Fillet Welding

3.15 Shear Key Connection

Shear key connections may be un-reinforced . When the elements are prevented from moving apart under shear loading, usually by means of reinforcing ties at top and bottom of the joint as shown in Figure 3.35, tests have shown that there is similar deformation behaviour as those of reinforced keys. However, there is also a definite increase in joint strength if the shear keys are reinforced.



(a) Diagonal Tension (b) Crushing and Shearing off corner of shear key (c) Direct Shearing (d) Dislocation

Figure 3.25 Failure Modes Of Joint Concrete In Shear Key

Figure 3.25 illustrates the failure modes of shear key joints which are :

a. diagonal cracks across the joint,
b. crushing and shearing off corner of the shear key,
c. direct shearing cracks parallel to the joint, and
d. slippage or dislocation at the contact face.

The different failure modes of shear keys are dependent on the compressive strength of the joint concrete, surface adhesion and bonding strength at the contact surfaces and the profile of the keys.

In general, the mode of failures in (a) and (c) takes place in normal shear keys having $6 \leq h/d \leq 8$ and $\alpha = 30^\circ$, where h and d being the respective height and depth of the shear key and α the slope of contact faces as shown in Figure 3.26. The minimum dimension of d should be 10mm. Test results have shown that maximum joint capacity is obtained for shear key having the above dimensions. (reference 10)

Failure mode (b) generally occurs in long shear key where $h/d > 8$ and the key capacity is not fully mobilised. In (d), failure occurs when $\alpha > 30^\circ$.

Depending on the number of keys, the forces acting on a single key joint due to a distributed vertical shear force V can be obtained as shown in Figure 3.26 :

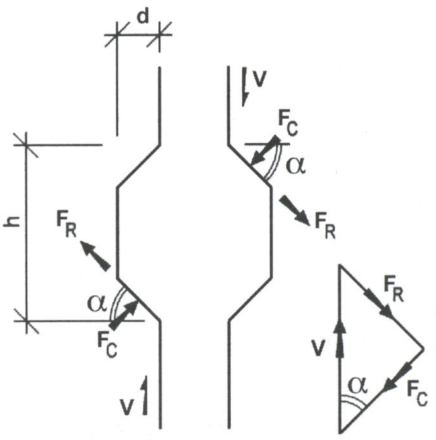


Figure 3.26 Forces In Shear Key Joint

Compression force perpendicular to the slope face of the key:

$$F_c = V \cos \alpha \qquad \text{— (3.55)}$$

Sliding force parallel to the key slope:

$$F_R = V \sin \alpha \qquad \text{— (3.56)}$$

The resistance to the sliding force F_R is given as :

$$\begin{aligned} F_R &= 0.6\mu F_c \\ &= 0.6 \mu \times V \cos \alpha \end{aligned} \qquad \text{— (3.57)}$$

where μ is the shear friction coefficient.

When F_R exceeds the frictional resistance by shear friction, shear friction reinforcement is to be provided for the total F_R. Reinforcement is calculated as in accordance with the Code:

$$A_s = V / (0.6 \mu \times 0.87 f_y)$$

Value of μ may be obtained from Table 3.1. In general it may be taken as 1.4 for shear key joint and A_s is then simplified as:

$$A_s = 1.2V / (0.87 f_y) \qquad \text{— (3.58)}$$

3.16 Column Base Connection

3.16.1 General

Column to foundation connections can be achieved through one of the following means :

1. socket connection
2. base plate connection
3. grouted pipe sleeves

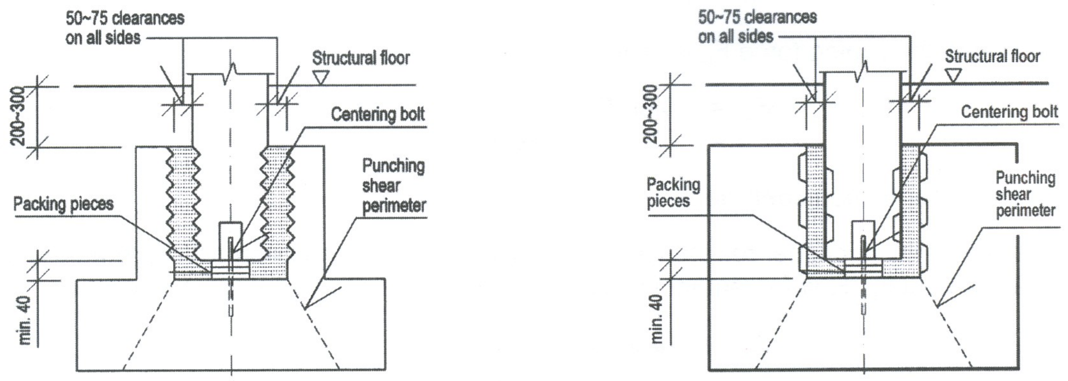
Column to column connections are, in principle, similar to column to foundation connections and methods (2) and (3) above are most commonly adopted in practice. The connection by pipe sleeves is explained in Section 3.6.

3.16.2 Socket connection

In socket connection, the precast columns are fixed rigidly to the foundation and loads are transmitted by skin friction in the socket and by end bearing. It is common to roughen the surfaces or form shear keys at the sides of the socket or columns in order to enhance the transfer of axial load by shear wedging. In the case of large overturning moment where the column reinforcement is in tension, the column bars extending into the socket must be fully anchored by bond or other means. The bars may be hooked at their ends for the purpose of reducing the depth of the socket. Additional links are also required in the precast columns to resist the bursting pressure generated by end bearing forces.

Figure 3.27 shows two variations of the socket connection in practice. The socket may also be precast when soil condition allows pad footing design.

In-situ socket is cast using a box shutter. The clearance gap between the socket wall and column should be at least 50 to 75 mm all round for ease of grouting or concreting as well as to allow for construction tolerances. The socket walls may be used to support precast or in-situ ground beams or slabs. In general, the structural floor may be 200 to 300 mm or greater above the socket. A minimum of 40 mm root depth should be allowed at the bottom of the socket.



(a) Socket above footing/pile cap

(b) Socket within footing/pile cap

Figure 3.27 Socket Connection

During erection, the precast column rests on packing pieces and is wedged into position using timber pieces as shown in Figure 3.28. Alternatively, column props at orthogonal directions may be used.

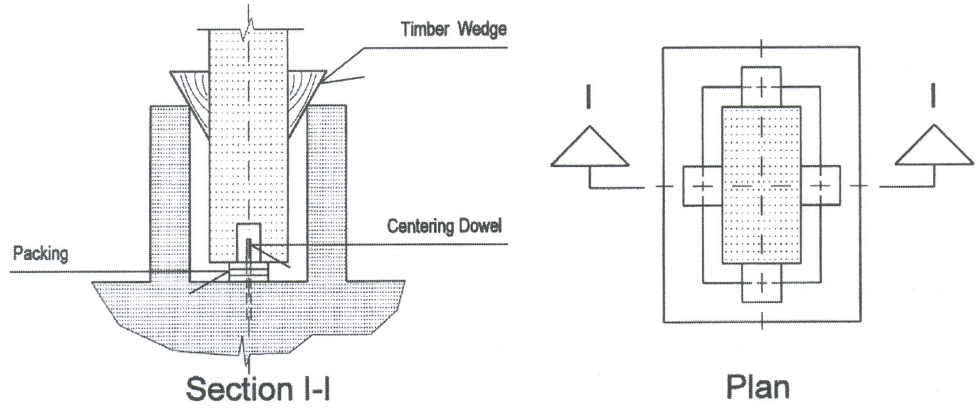


Figure 3.28 Timber Wedging At Column Installation

To assist in centering the column, a dowel may be provided at the base of the socket.

The design of the socket connection may adopt the following steps: (reference 11)

1. Support reactions

Referring to Figure 3.29, a rotation of the column in the socket under moment shifts the support reaction at the column base from the centre line towards the edge. The resultant reaction R at the base may be assumed to act at a distance a/6 from the column centre line.

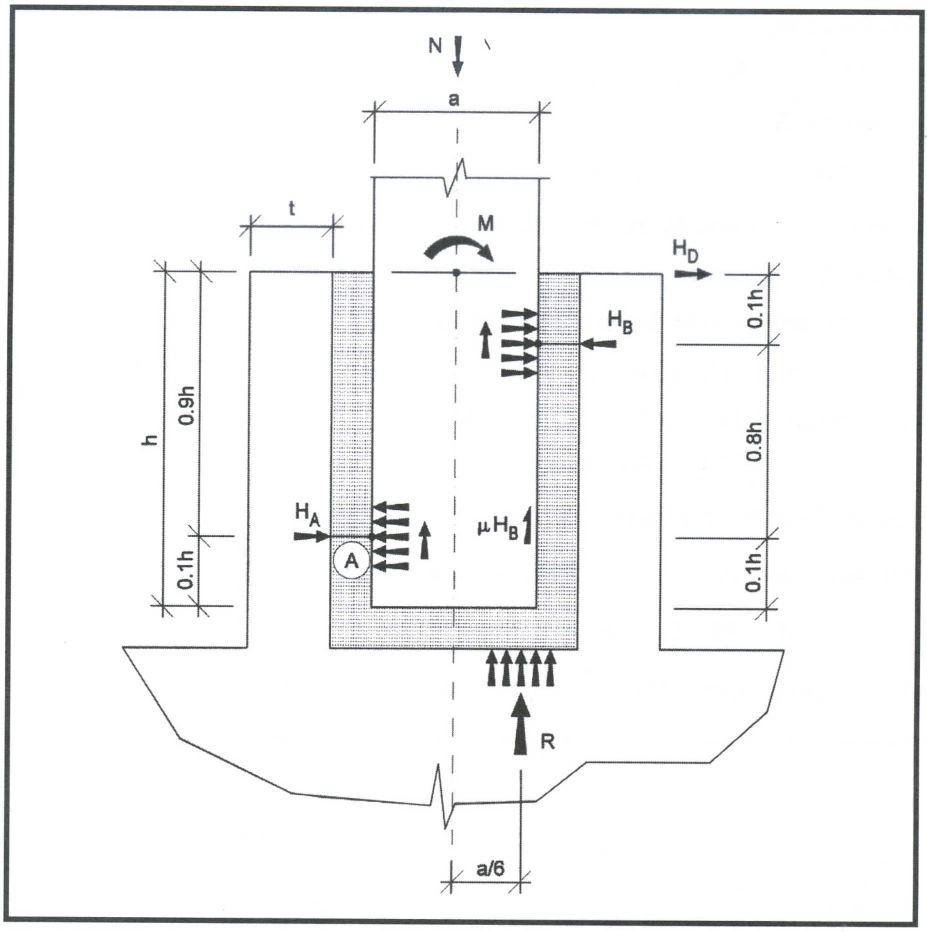


Figure 3.29 Distribution Of Forces In Column Socket

From Figure 3.29 the following forces may be derived :

a. **Horizontal force H_B**

Moment about point A

$M + N(a/2) + H_D \times 0.9h = H_B \times 0.8h + \mu H_B \times a + R(a/2 + a/6)$

Support reaction at column base is

$R = N - \mu H_B$

Substituting R into the above equation and after rearranging

$$H_B = (M - 0.17aN + 0.9hH_D) / (0.8h + 0.33a\mu) \qquad \text{—— (3.59)}$$

If the height of the socket is taken as h = 1.5a and ignoring the top 0.1h (assumed cover zone) of the socket height, the effective column embedment length is given as

$h = 0.9 \times 1.5a$

i.e. $a = 0.74h$

For a smooth column face, the coefficient of friction $\mu = 0.3$ is used and substituting $a = 0.74h$ and $\mu = 0.3$ into equation 3.59, the following simplified expression for horizontal force H_B is obtained:

$$H_B = 1.14\, M / h - 0.15N + 1.03H_D \qquad \text{—— (3.60)}$$

b. **Horizontal force H_A**

By equilibrium : $H_A = H_B - H_D$ — (3.61)

c. **Vertical reaction R**

Resolve vertically : $R = N - \mu H_B$ — (3.62)

Hence knowing H_D, N and M, H_A, H_B and R can be calculated.

d. **Reinforcement in socket**

The ring reinforcement at H_B level is calculated from

$$A_{SB} = H_B / (0.87f_y) \tag{3.63}$$

Ring reinforcement at H_A level is calculated from

$$A_{SA} = (H_A - \mu R) / (0.87f_y) \tag{3.64}$$

If μR is greater than H_A, no ring reinforcement is needed.

Vertical reinforcement A_{sv} in the socket is calculated at the lower level using the moment

$$M_v = M + H_D \times h \tag{3.65}$$

Schematic reinforcement in the socket is shown in Figure 3.30.

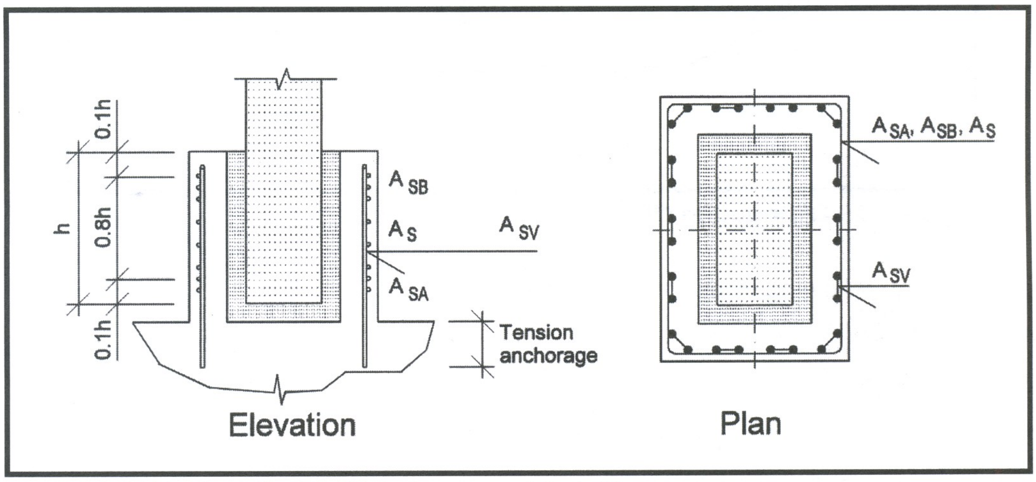


Figure 3.30 Typical Socket Reinforcement Detailing

2. **Column reinforcement in socket**

To reduce the depth of the socket, the main column reinforcement in the socket may be lapped with looped bars. Minimum reinforcement at the lower end of the column should be

$$A_s = H_A / (0.87f_y)$$

Typical column reinforcement in the socket is shown in Figure 3.31.

Within the socket depth, the column section should be designed for shear using H_A. Effect of increases in concrete shear capacity due to axial compression may be ignored. There should be additional stirrups in the embedded column as it will improve the main steel bar anchorage. The stirrups will also act as reinforcement for bursting forces at the base due to axial vertical load. The amount of stirrups to be provided (assumed to be uniformly distributed) for bursting effect may be calculated from

$\zeta = H_{bst}/N$
$= 0.11$ (Table 4.7, Part 1, CP 65)

$H_{bst} = 0.11N$ —— (3.66)

where H_{bst} is the bursting force and N is the axial vertical force.

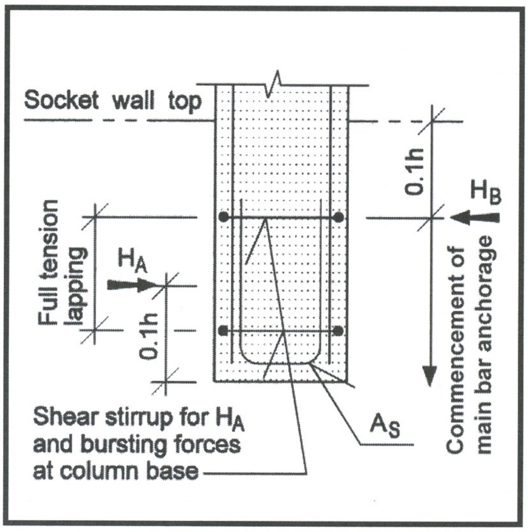


Figure 3.31 Typical Reinforcement Of Column In Socket

3. Socket dimensions

The following considerations have been commonly used in the dimensioning and design of column sockets :

a. strength of concrete in socket ≥ C35

b. strength of concrete/grout for the socket gap infill ≥ C35

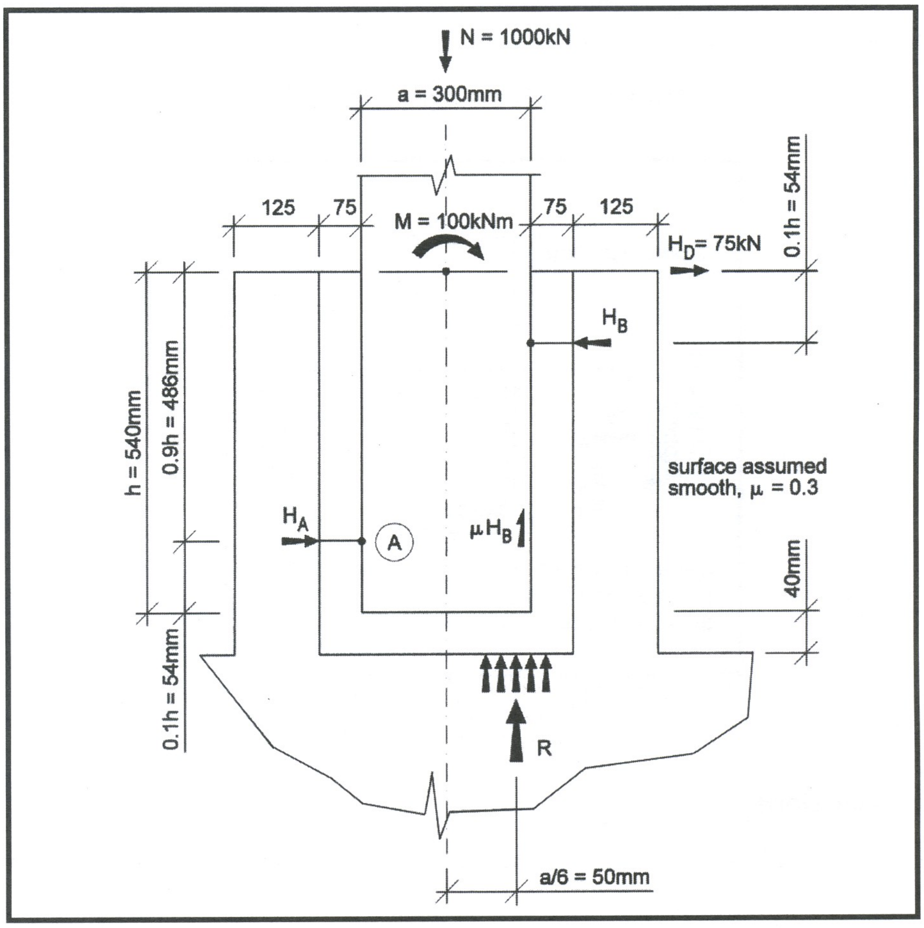
c. height of socket should be $a < h \leq 1.5a$ where a is the greater dimension of the column

d. the minimum wall thickness of the socket is $t = 0.18b + 70$ (mm) where b is the smaller column dimension

e. minimum static friction coefficient
$\mu = 0.3$ (smooth surface)
$\mu = 0.7$ (roughened surface)

Design Example 9: Socket Connection For Column

Design a column socket connection at the foundation which is required to support a 300 x 300 mm, C50 precast column subjected to an ultimate axial load of N = 1000 kN, a moment of M = 100 kNm and a horizontal force of H_D = 75 kN. Determine the minimum strength of the in-situ fill. Cover to column bars = 35 mm and to foundation bars = 50 mm. The column is reinforced with 4T16 and with links R10@150 c/c.



Forces In The Column Socket

Column size = 300 x 300mm
N = 1000 kN
M = 100 kNm
H_D = 75 kN

1. Socket dimensions

a. Socket depth

h = 1.5a
= 1.5 x 300
= 450 mm

Allow 40 mm at root depth, hence overall socket depth
= 450 + 50
= 490 mm, say 500 mm

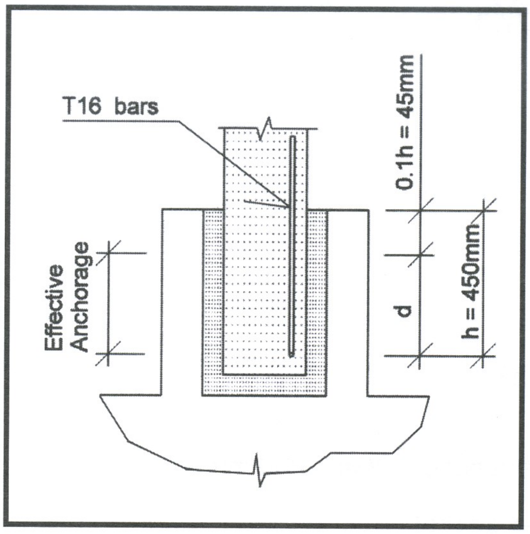
Ultimate bond stress in tension anchorage

$f_b = \beta\sqrt{f_{cu}}$
$= 0.5\sqrt{50}$
$= 3.54$ N/mm^2

For T16, tension anchorage $l_p = (0.87 f_y A_s) / (f_b \times \pi\phi)$

$= (0.87 \times 460 \times 201) / (3.54 \times \pi \times 16)$

$= 452$ mm



Anchorage Of Column Bars

Depth of socket

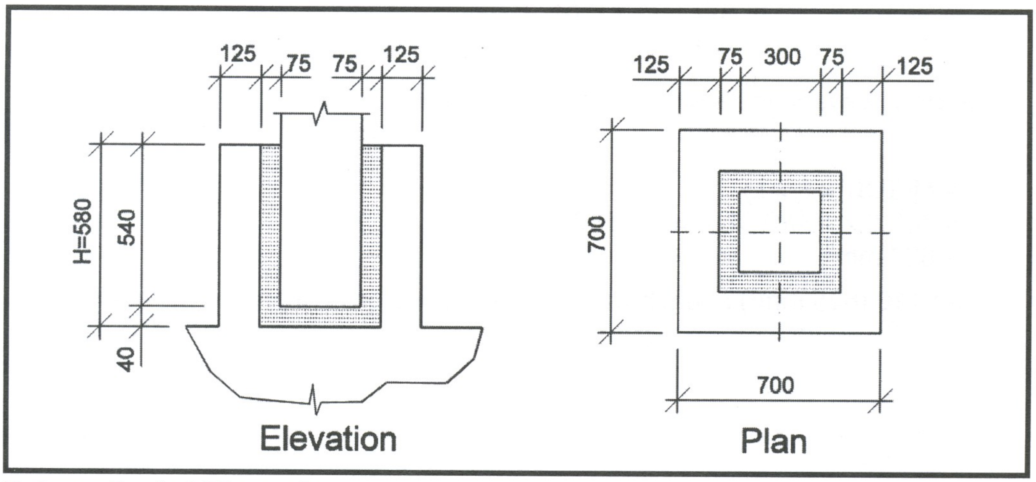
h = 452 + 0.1h + cover
= 452 + 0.1 x 452 + 35
= 532mm, say 540mm

Allowing 40mm root depth at the base, the total depth of the socket is 540+40=580mm

b. Socket wall thickness

t = 0.18b + 70
= 0.18 x 300 + 70
= 124mm, say 125 mm

The dimensions of the socket are shown below.



Column Socket Dimensions

2. Forces in socket

a. Horizontal force H_B

$H_B = 1.14\ M / h - 0.15N + 1.03H_D$
$= 1.14 \times (100/0.54) - 0.15 \times 1000 + 1.03 \times 75$
$= 211.1 - 150 + 77.3$
$= 138.3$ kN

b. Horizontal force H_A

$H_A = H_B - H_D$
$= 138.3 - 75$
$= 63.3$ kN

c. Vertical reaction R

$R = N - \mu H_B$
$= 1000 - 0.3 \times 138.3$
$= 958.5$ kN

Width of R acting at the base = 2/3a = 200 mm

3. Reinforcement design in socket

a. Ring reinforcement at H_B level

$A_{SB} = H_B/0.87f_y$
$= 138.3 \times 10^3/(0.87 \times 460)$
$= 346\ mm^2$, use 3T10 (6 legs) ($A_s = 471\ mm^2$)

b. Ring reinforcement at H_A level

$A_{SA} = (H_A - \mu R)/0.87f_y$
$= (63.3 - 0.3 \times 958.8)/(0.87 \times 460)$
= -ve value

No ring reinforcement is theoretically required but nevertheless provide 3T10 (6 legs).

c. Vertical reinforcement in socket wall

Design moment $M_v = M + H_D \times h$
$= 100 + 75 \times 0.54$
$= 140.5$ kNm

Lever arm between the steels in walls $z = 700 - 70 - 70$
$= 560$ mm

$A_{sv} = M/(0.87 \times f_y \times z)$
$= 140.5 \times 10^6/(0.87 \times 460 \times 560)$
$= 627\ mm^2$

Use 8T10 (628 mm^2) at each wall. Use U-shaped bars to ensure full anchorage.

d. Column reinforcement in socket

i. Bursting Reinforcement

$$H_{bst} = 0.11N$$
$$= 0.11 \times 1000$$
$$= 110 \text{ kN}$$

$$A_{bst} = 110 \times 10^3/(0.87 \times 250)$$
$$= 505 \text{ mm}^2 \text{ in column}$$

Links R10-150 in column ($A_s = 523$ mm^2/m) are adequate.

ii. End reinforcement

$$A_s = H_A/0.87f_Y$$
$$= 63.3 \times 10^3/(0.87 \times 460)$$
$$= 158 \text{ mm}^2 < 4T16, \text{ use } 4T16$$

iii. Check shear at column base

$$\text{Max. shear force} = H_A$$
$$= 63.3 \text{ kN}$$

$$\nu = 0.94 \text{ N/mm}^2$$
$$r_s = 4T16 = 1.14\%$$
$$\nu_c = 0.95 \text{ N/mm}^2 > 0.94 \text{ N/mm}^2$$

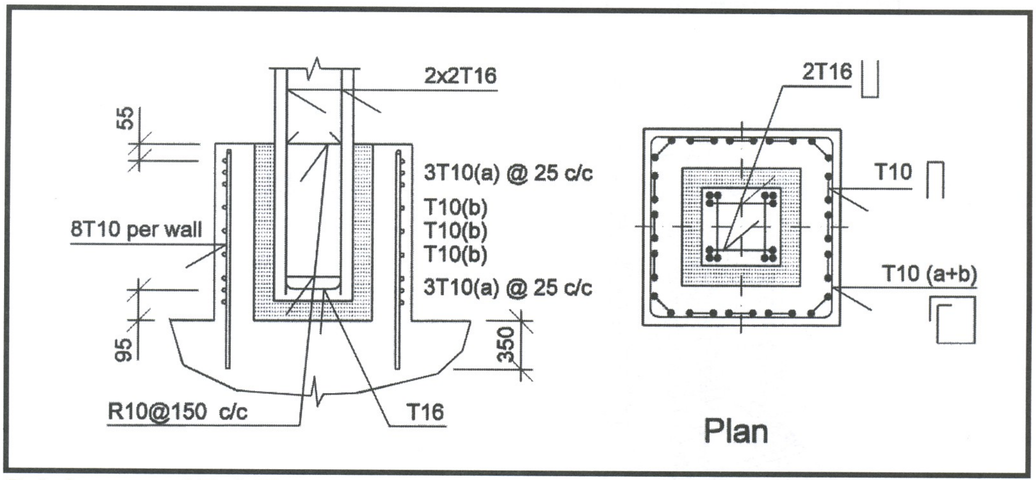
OK

4. Strength of infills

$$R = 958.5 \text{ kN}$$
$$\text{Contact area} = 300 \times 200\text{mm}^2$$
$$f_c = 958.5 \times 10^3 / (300 \times 200)$$
$$= 16.0 \text{ N/mm}^2$$

From clause 5.2.3.4, the concrete grade of infill $f_{cu} = 16.0/0.6$
$= 26.7$ N/mm^2, say min. C35

5. Detailing

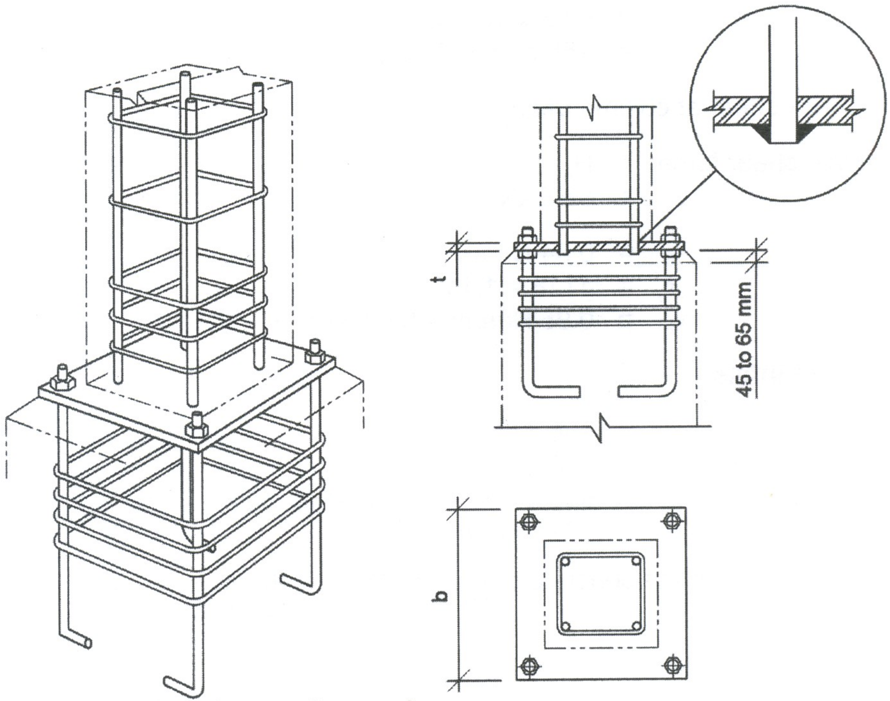


Reinforcement Detailing

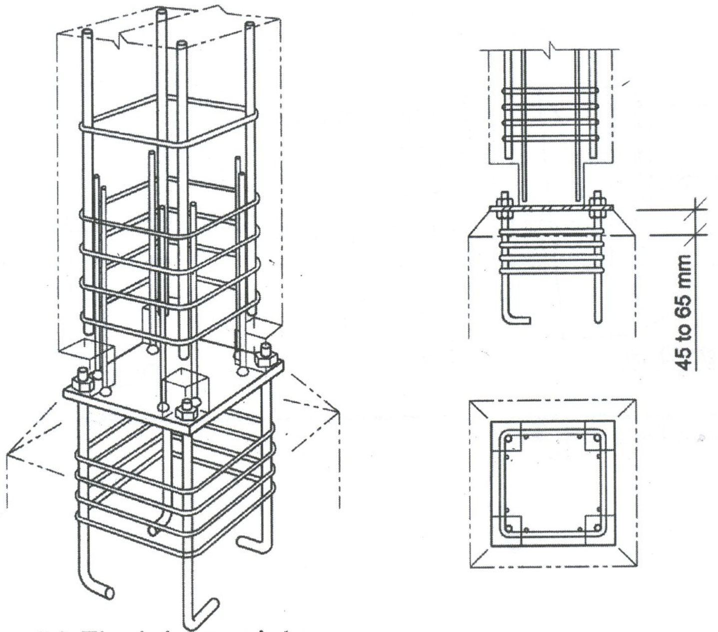
3.16.3 Base plate connection

Column base plates can be used when a moment connection is required. The base plates must be designed for both the erection loads and loads which occur in service. Unlike socket or grouted pipe sleeves connections which require time to develop the necessary strength, columns using base plate connections can achieve immediate stability which will greatly facilitate erection of other precast components. The choice in using base plates rather than sockets or grouted pipes is based more on work productivity than on structural requirements.

Two commonly used base plate details are shown in Figure 3.32. The column base plates may be larger than, flushed or smaller than, the column dimensions. In cases where base plates are larger than the columns, the plates must be protected against corrosion by concrete haunching, electro or hot-dip galvanising.



(a) Base plate larger than column



(b) Flush base plate

Figure 3.32 Column Base Plate Connections (references 5 and 9)

The base plates are fixed to the column by steel reinforcement welded to the plates. Additional links are usually provided to resist bursting pressure.

Holding down bolts are often specified to be either grade 4.6 or 8.8. The length of bolts should be designed in accordance with the acting forces. The end of the bolts may be hooked, L-shaped or fitted with a steel plate typically 100 x 100 x 9 mm thick to increase the pull-out capacity. Confined reinforcement in the form of links around the bolts is recommended and should not be less than 4 numbers of R8 links at 50 mm near to the top of the bolts.

The thickness of the base plate depends on the overhang projection from the column face. It is subjected to biaxial bending from the resultant compressive forces acting on the surface. For this reason, the overhang projection is normally limited to 100 to 125 mm; a minimum practical limit for detailing and site erection purposes. Holes in the plate are normally oversized to offset construction setting out and production tolerances. In general, there should be an all round gap of between 10 to 15 mm between the edges of the bolt and plate hole.

Figure 3.33 shows the distribution of forces in a column base plate connection.

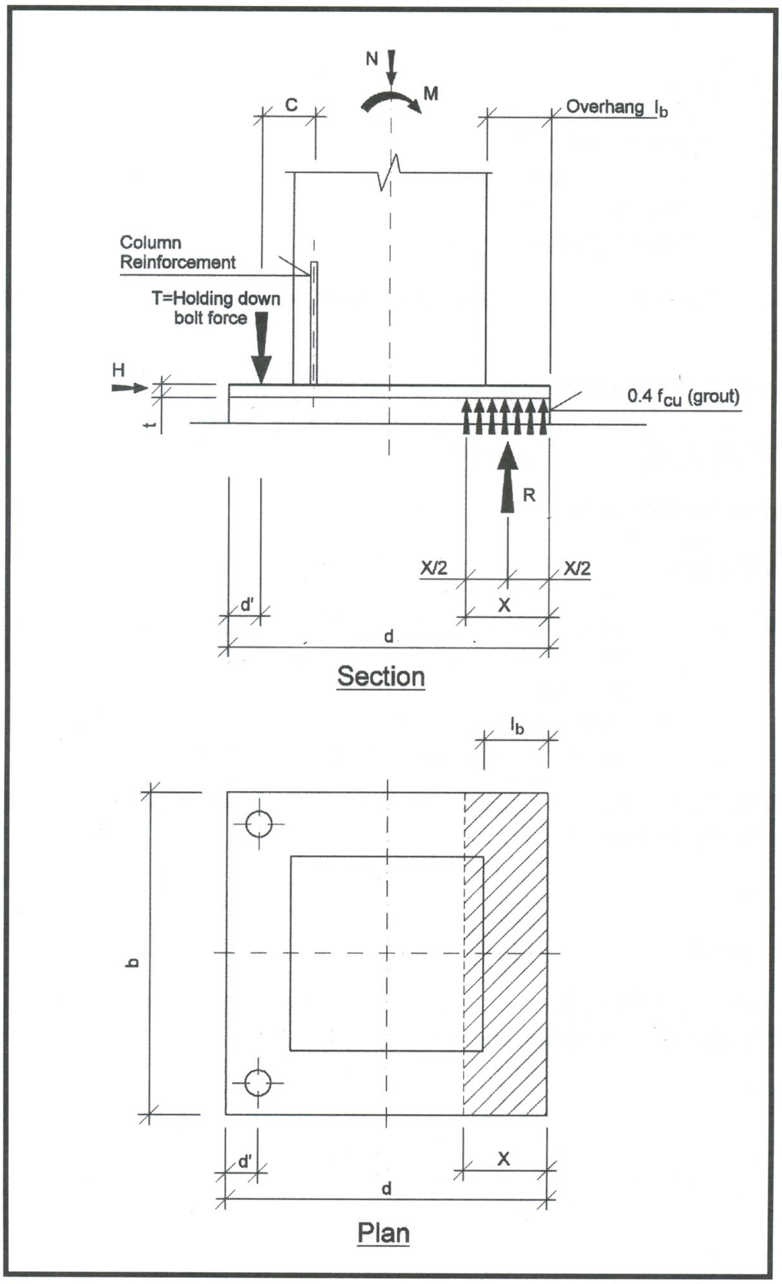


Figure 3.33 Force Distribution In Column Base Plate

The following steps may be used to calculate the base plate thickness:

a. Resolve forces vertically

$$N = 0.4f_{cu}b\chi - T \qquad \text{—— (3.67)}$$

where χ = compressive stress block depth

b. Moment about centreline of compressive stress block

$$T = [M - N(d/2 - \chi/2)] / (d - d' - \chi/2) \qquad \text{—— (3.68)}$$

Substituting equation 3.67 into 3.68 and putting M=Nxe, the following simplified expression is obtained.

$$(\chi / d)^2 - 2(1 - d' / d)(\chi / d) + 5\, N(e+0.5d-d') / (f_{cu}bd^2) = 0 \qquad \text{—— (3.69)}$$

from which χ/d and hence T can be calculated.

If $\chi/d > N/0.4f_{cu}bd$, then T is positive and the area of holding down bolts can be calculated from :

$$A_b = T / (\Sigma n \times f_{yb}) \qquad \text{—— (3.70)}$$

where Σn = number of bolts
f_{yb} = ultimate tensile strength of bolts
= 195 N/mm^2 for grade 4.6 bolts
= 450N/mm^2 for grade 8.8 bolts

The thickness t of the column base plate may be obtained using plastic analysis and is the greater of :

i. Based on compression side

$$t = \sqrt{0.8f_{cu}l_b^2/p_y} \quad \text{in mm} \qquad \text{—— (3.71)}$$

ii. Based on tension side

$$t = \sqrt{4Tc/(bp_y)} \quad \text{in mm} \qquad \text{—— (3.72)}$$

where p_y = yield strength of steel plate (refer to table 6, Part 1, BS 5950)
= not greater than 275 N/mm^2 for grade 43 and 355 N/mm^2 for grade 50 steel.
l_b = plate overhang beyond column face
c = distance between centres of bolt and column bar

If $\chi/d < N/0.4f_cbd$, then T is negative and the above equations are not valid. The analysis simplifies to the following :

$$N = f_c\, b\chi \qquad \text{—— (3.73)}$$

$$\chi/d = 1 - 2e/d \qquad \text{—— (3.74)}$$

where f_c is the uniformly distributed concrete compressive stresses under the combined action of N and M. The required plate thickness is given by

$$t = \sqrt{2f_cl_b^2/p_y} \qquad \text{—— (3.75)}$$

Design Example 10: Column Base Plate Connection

Design a column base plate connection for a 300 x 300mm precast column subjected to an ultimate axial load N = 1000 kN and a moment M = 100 kNm. Use grade 43 base plate and grout strength f_{cu} = 40 N/mm^2.

Try 500 x 500 plate with 100 mm overhang all round from the column face.

From equation 3.67

$$(\chi/d)^2 - 2(1-d'/d)(\chi/d) + 5\,N(e + 0.5d - d') / (f_{cu}bd^2) = 0$$

$e = M/N = 100$ mm
$b = d = 500$ mm
$d' = 50$ mm
$f_{cu} = 40$ N/mm^2

$$(\chi/d)^2 - 2(1 - 50/500)(\chi/d) + 5 \times 1000\,(100 + 0.5 \times 500 - 50) \times 10^3 / (40 \times 500 \times 500^2) = 0$$

$$(\chi/d)^2 - 1.8(\chi/d) + 0.3 = 0$$

$$(\chi/d) = 0.186$$

$$N / (0.4\,f_{cu}bd) = 1000 \times 10^3 / (0.4 \times 40 \times 500 \times 500)$$

$$= 0.25 > 0.186$$

Hence there is no tension in the holding down bolts.
Revise the design using equation 3.74.

$$\chi/d = 1 - 2e/d$$
$$= 1 - 2 \times 100/500$$
$$= 0.6$$

χ = 300 mm

From equation 3.73, $N = f_c b \chi$

$$f_c = 1000 \times 10^3/(500 \times 300)$$
$$= 6.67 \text{ N/mm}^2$$

From equation 3.75, the plate thickness

$$t = \sqrt{(2f_c l_b^2/p_y)}$$

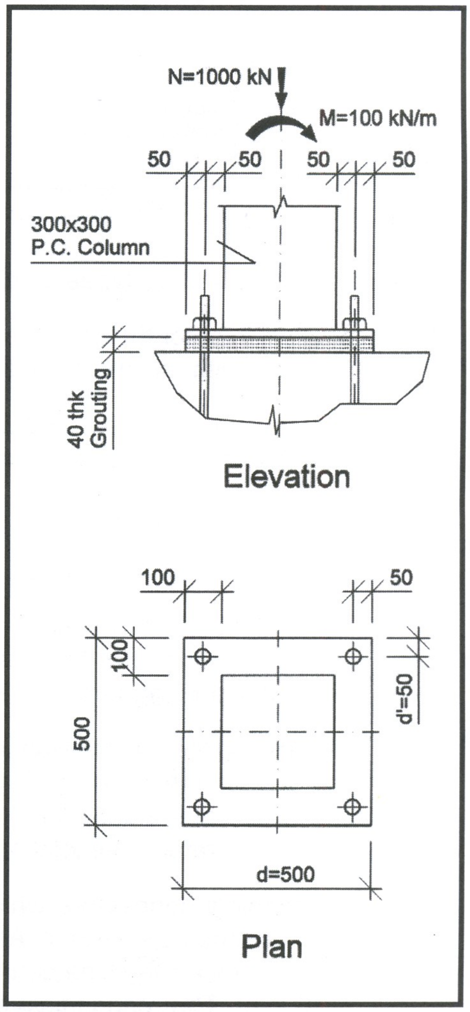
Using grade 43 steel plate, p_y = 275 N/mm^2 and l_b = 100 mm

$$t = \sqrt{(2f_c l_b^2/p_y)}$$
$$= \sqrt{(2 \times 6.67 \times 100^2/275)}$$
$$= 22 \text{ mm}$$

Use t = 25 mm

Plate size = 500 x 500 x 25 mm thick

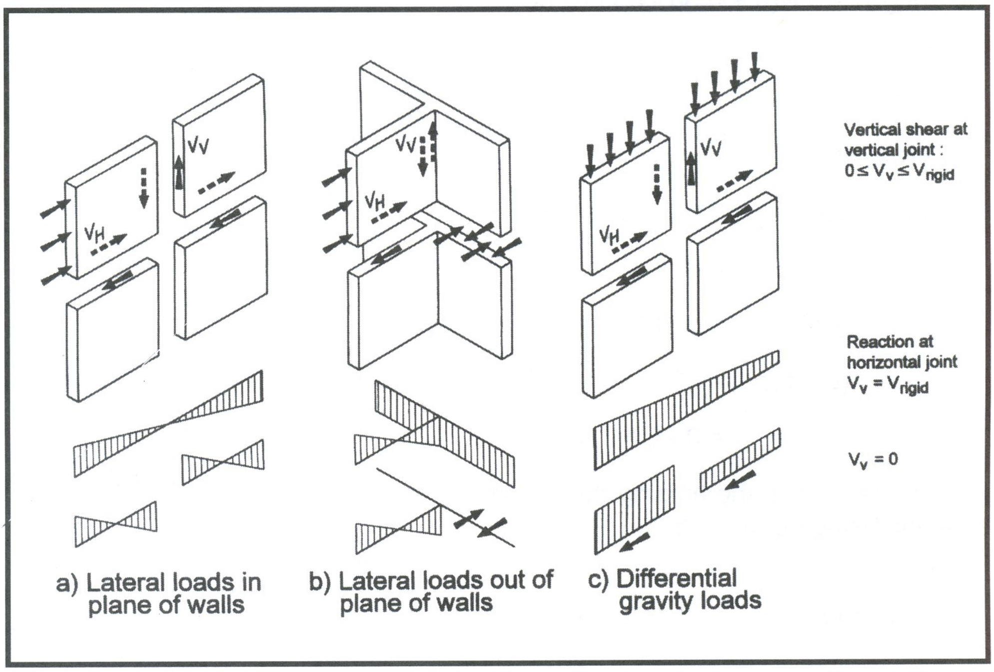
Note: When steel plate thickness t>16mm, the required plate thickness should be recalculated using p_y = 265 N/mm^2. The adopted plate t = 25mm is, however, still valid in the above calculations.



Column Base Plate

3.17 Connection Of Precast Walls

Precast wall panels are usually single-storey high panels which are connected to each other and to the floor slabs. The connections are an integral part of the structural support system for vertical gravity dead and live load as well as for the transfer of horizontal in-plane forces from the floor diaphragm action. Figure 3.34 illustrates the different resulting joint force systems from internal and external forces.



a) Lateral loads in plane of walls

b) Lateral loads out of plane of walls

c) Differential gravity loads

Figure 3.34 Exterior Forces And Joint Force System (references 5 and 9)

There are two principal types of joint in precast wall panels :

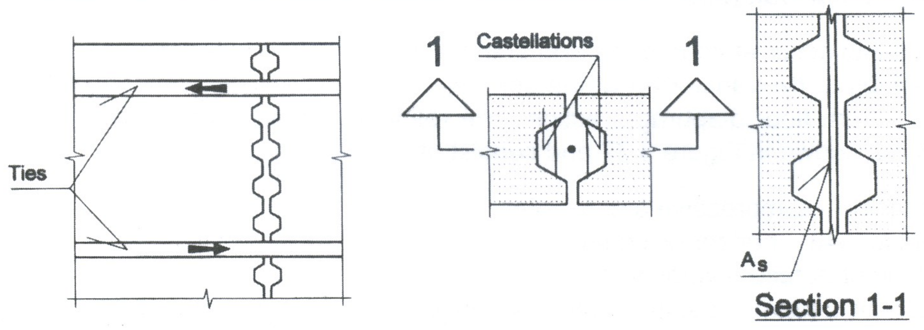
1. vertical joint for the purpose of transferring vertical shear forces from one wall component to the next with minimum relative movement
2. horizontal wall to floor and wall to foundation joint for the transferring of compressive, tensile and shear from one component to the other.

3.17.1. Vertical wall joint

Vertical wall joints may be formed by :

1. concentrating on the reinforcement crossing the joint at the top and bottom of the wall panels within the structural floor depth. The reinforcement serves as structural ties and provides artificial clamping forces to prevent the wall panels from separating. See Figure 3.35a.
2. embossing the edges of the wall panels with castellations or shear keys which act as mechanical locks when the panels deform under shear loading. Interlacing reinforcement projecting from the edges of the panels and running along the joint area as shown in Figure 3.35b may also be incorporated. The joint space is finally filled with concrete or grout.
3. mechanical connectors which consist of cast-in anchorage devices in the walls and steel section crossing the joint. As shown in Figure 3.36, the final connection is normally made by bolting or welding. The connections are usually located at the upper and lower region of the wall joint. The mechanical assemblies are eventually encased with concrete for protection against exposure to weather and fire.

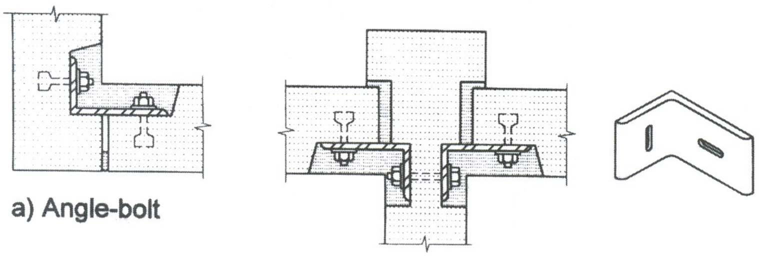
Figures 3.35 and 3.36 show the differences between each of the above joints.



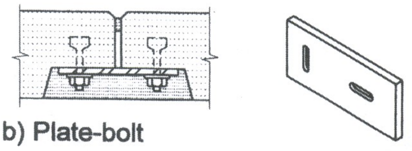
a) The reinforcing ties are placed in the horizontal joint

b) Tie bar in the vertical joint

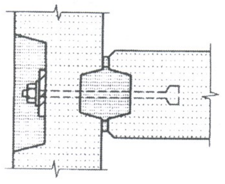
Figure 3.35 Connection At Vertical Shear Joints



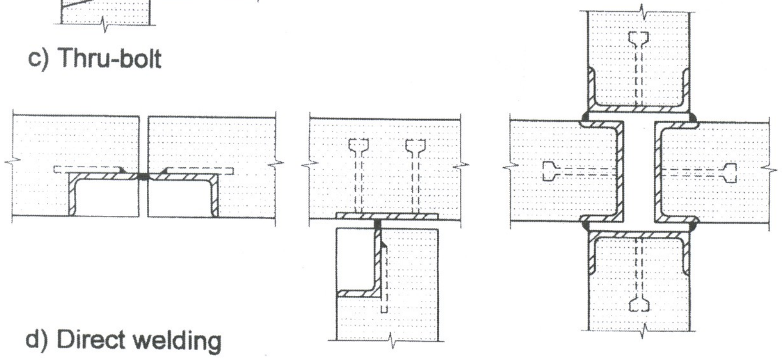
a) Angle-bolt



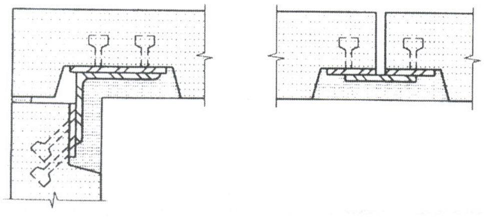
b) Plate-bolt



c) Thru-bolt



d) Direct welding



e) Welding with make-up pieces

Figure 3.36 Mechanical Connection In Vertical Wall Joint (references 5 and 9)

3.17.2 Horizontal wall joint

Horizontal wall joints occur at floor levels and at the transition to the foundation or transfer beams. The principal forces to be transferred at the connection are essentially vertical gravity loads and horizontal forces from floor diaphragm action. The resulting forces acting at the joint for design considerations are shown in Figure 3.37 and consist of:

1. normal to joint – compressive and tensile
2. horizontal to joint – horizontal shear
3. vertical to joint at face – vertical shear
4. perpendicular to joint – compressive and tensile from floor diaphragm action and bending stresses from framing action of the wall with floor slabs

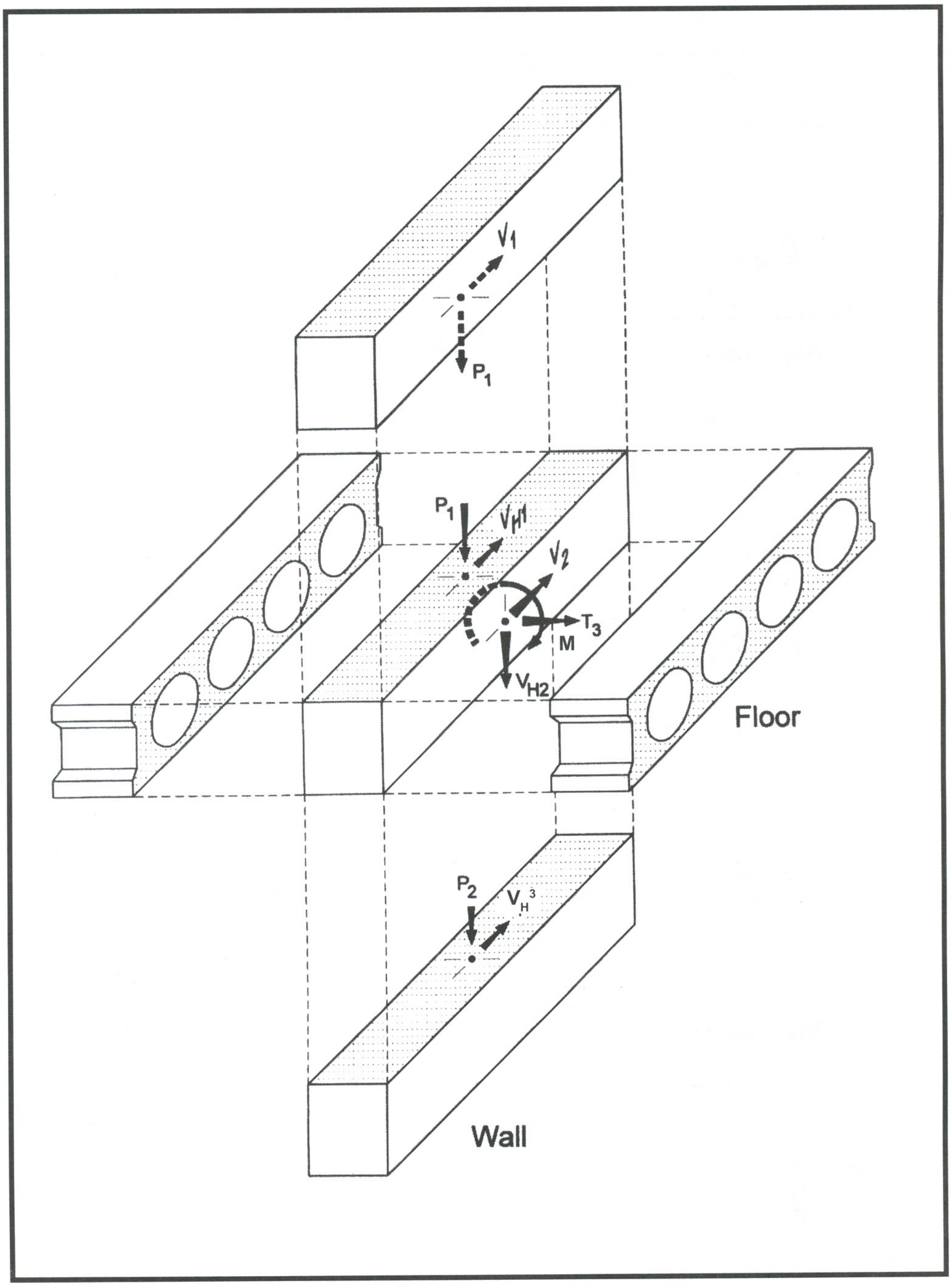


Figure 3.37 **Exploded View Of An Interior Floor To Wall Connection And The Various Connection Forces (references 5 and 9)**

The capacity of the joint to transfer vertical loads depends on a number of factors:

1. compressive strength of joint concrete and wall panels
2. ratio of loaded width to wall thickness
3. ratio of loaded width to joint thickness
4. splitting strength of wall ends and joint concrete
5. confinement of joint concrete
6. tensile strength of mechanical connectors

The horizontal load transfer capacity will depend on :

1. shear friction resistance
2. frictional resistance at interface
3. horizontal shear strength of mechanical connectors

Depending on the floor elements and their supporting details, the horizontal joint may be constructed continuous or with connections at isolated locations.

In a continuous joint, there are, in general, three basic types of details as shown in Figure 3.38.

1. thin mortar joint
2. wedged or open joint
3. platform or close joint

The isolated connections can be grouted pipe sleeves, dowels or mechanical connections with welded plates and bolts.

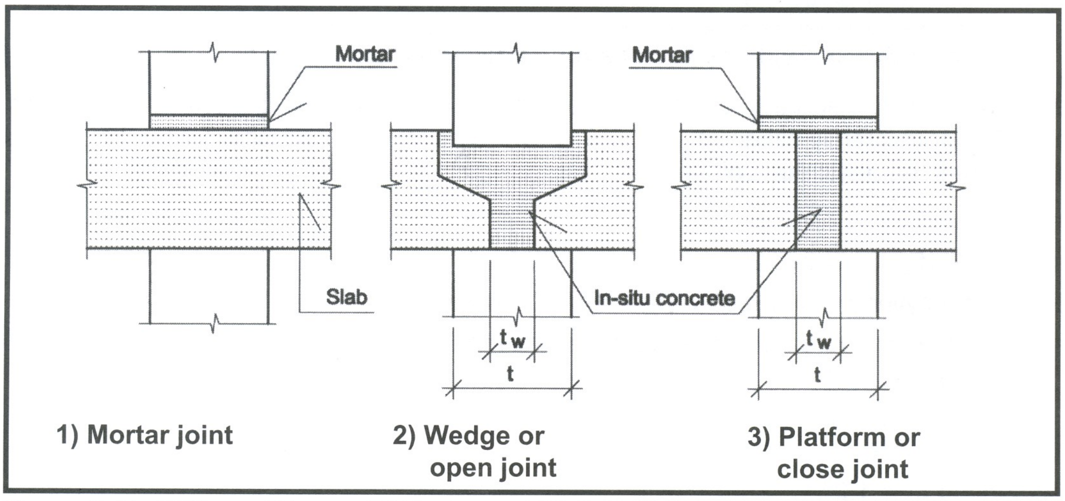


Figure 3.38 Different Forms Of Horizontal Wall Joints

3.17.3 Vertical load capacity of joint concrete or mortar

The mortar joint is cheap, simple and the most widely used continuous horizontal joint connection in precast wall construction. The mortar may be in fluid colloidal or dry-packed form and is either poured or packed in the joint space before or after the walls are erected. The joints are usually unreinforced.

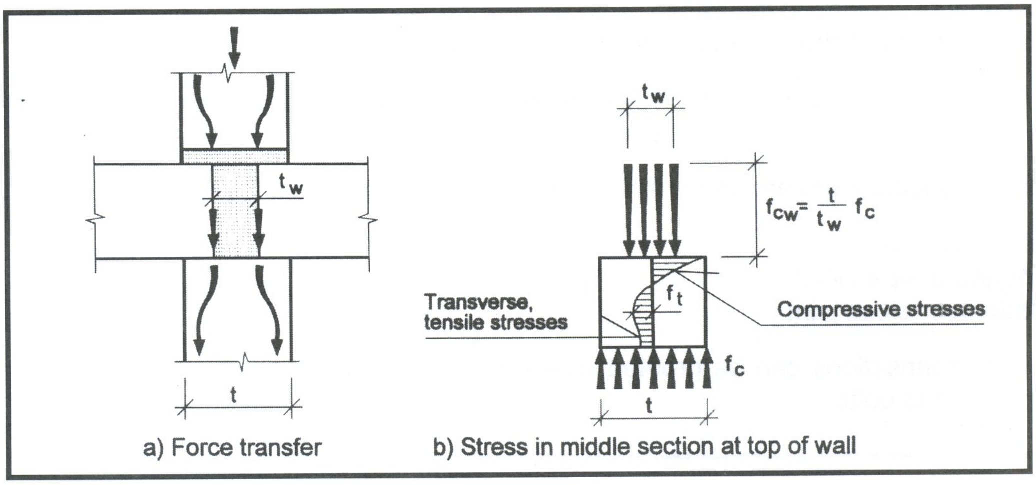
The strength of the mortar joint depends primarily on the relative strength (and hence the elastic response) of the mortar and the walls as well as the dimensions of the joint. Under vertical loads, the joint may fail in one of the manners as shown in Figure 3.1.

The vertical load capacity of the mortar joint may be determined either by stipulations in Part 1, clause 5.3.6 of the Code or by other approaches as given in Section 3.6.1.

For wedge or platform horizontal wall joints shown in Figure 3.38, a simple design method to determine the vertical load capacity of the precast wall panels is outlined in the SBI Direction 115 - Danish Building Research Institute 1981 (reference 6).

The design method assumes the occurrence of split failure in the wall panel concrete as shown in Figure 3.39. The vertical stresses are assumed to be uniformly distributed over the panel width, t, and the width of the joint concrete, t_w. In the mid-vertical section of the wall, transverse stresses occur in the form of compressive stresses at the top and tensile stresses a little further down. The maximum transverse stresses in the wall panel concrete, denoted as f_t, is related to the vertical compressive stresses, f_c, by the simple relationship

$$f_t / f_c = (1 - t_w / t) / 2 \qquad \text{—— (3.76)}$$



a) Force transfer

b) Stress in middle section at top of wall

Figure 3.39 Stress Systems In Horizontal Wall Joint

Failure will occur if f_t is equal to the tensile strength of the concrete. If the tensile strength of the concrete is conservatively taken to be 10% of the compressive strength, then the vertical load capacity per unit length of the wall panel is calculated as below:

From equation 3.76 $\quad f_c = 2f_t / (1 - t_w / t)$

Assuming $f_t = 0.1f_{cyd}$ $\quad f_c = 0.2f_{cyd} / (1 - t_w / t)$

Where f_{cyd} is the cylinder compressive strength of the wall panel.

The vertical load carrying capacity per unit length of the wall is given as:

$$\begin{aligned} n_w &= f_c \text{ x } t \\ &= 0.2f_{cyd} / (1 - t_w / t) \text{ x } t \\ &= 0.2\, t\, f_{cyd} / (1 - t_w / t) \end{aligned}$$

Assuming $f_{cu} = 1.25\, f_{cyd}$

$$n_w = 0.16 + f_{cu} / (1 - t_w / t) \qquad \text{—— (3.77)}$$

According to reference 6, equation 3.77 is valid up to a compressive failure of the wall panel when the compressive stresses in the wall panels reach $0.75f_{cyd}$. (equivalent to $0.6f_{cu}$).

Hence $n_w \leq 0.6f_{cu} \text{ x } t$

3.17.4 Isolated connection

Typical examples of isolated connections in precast walls are shown in Figure 3.40. Since all forces are concentrated at a few points, special attention must be taken to ensure that the concentrated forces at the connection can be safely dispersed to the upper and lower wall panels. Small diameter reinforcement in the form of stirrups or loops should be detailed in the immediate vicinity of the connection in the wall panels to prevent splitting, bursting and spalling of concrete.

In cantilever shear walls subjected to overturning in-plane moment, the moment can be resolved into tension and compression force couple. The tension force can be resisted by grouted pipe sleeves, dowels, couplers, bolts or welded plates connection. Similar details may also be applied to the compression force as in most cases, the moment is reversible.

3.17.5 Structural ties in wall joint

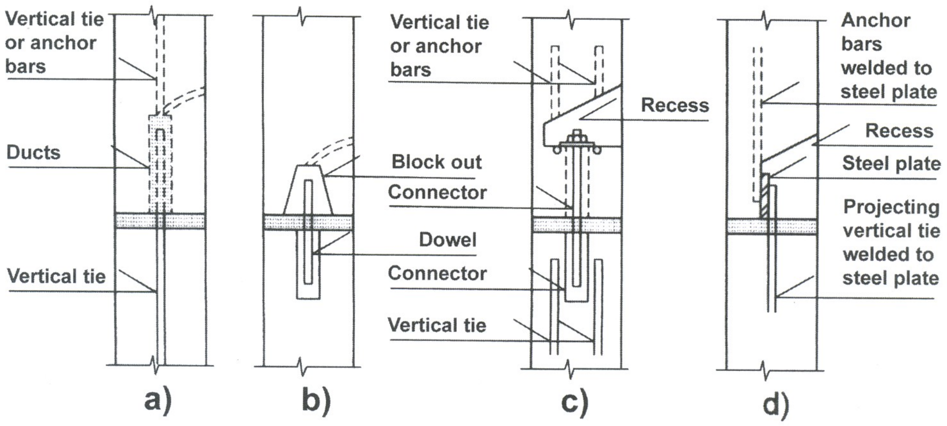
Structural vertical and horizontal integrity ties can be incorporated into the joint details and some typical examples are shown in Figure 3.35.

3.17.6 In-plane shear capacity

The in-plane shear capacity in vertical and horizontal wall joints with smooth or cast surfaces can be determined based on the permissible ultimate shear stress values given in Part 1, clause 5.3.7, under the following conditions :

1. 0.23 N/mm^2 in the absence of compressive forces across the joint
2. 0.45 N/mm^2 when the joint is under compression under all design conditions. The compressive force may be generated from gravity dead and live load or artificially created by reinforcement placed across the joint.

The in-plane shear capacity is only valid provided the joints are prevented from opening up.



a) Vertical ducts and lapping of ties by grouting

b) Simple dowel pin connection for horizontal shear forces only

c) Mechanical bolting of vertical tie reinforcement

d) Welded joint

Figure 3.40 Isolated Connection In Wall Panels