# Connections between precast elements: types, design, application

*Course Design Principles for Precast Concrete Structures*
*South Africa, January 2013*

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**01 February 2013**

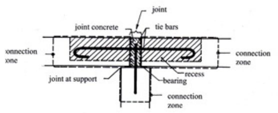




## Basic considerations

In areas with low seismic it is essential that simple and easy to handle solutions are pursued at all stages of the construction process: from design to manufactoring and erection of elements. This is even more valid when it comes to connections in precast concrete. It is a misconception to see the precast concrete technology only as a mere translation of a cast in situ structure into a number of precast concrete elements, which have to be assembled on site in a manner that the initial cast in situ concept is obtained. This misconcept is due to a lack of understanding of the design philosophy and the special characteristics and rules associated with precast concrete design and construction.

In areas with seismic activity additional considerations apply. In this case energy dissipation plays a major role. Frames are designed in such a way that energy dissipation occurs in the beams, which are much more ductile than the columns. In such a case joints should be placed either far from the most stressed regions, or made strong enough to not reach failure first. Structures with shear walls can be designed as strong and stiff members, preventing damage under low intensity earthquakes.







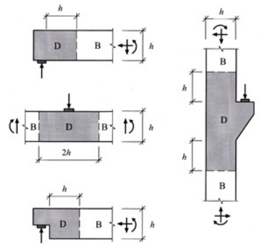

## General considerations

The following design aspects play a role:

- Standardisation
- Simplicity
- Tensile capacity
- Ductility
- Movements
- Fire resistance
- Durability
- Aesthetics

Skeleton frame structure: combination of columns, beams and floor elements

## Role of strut and tie models in the design of connections



Examples of D-regions of structural elements



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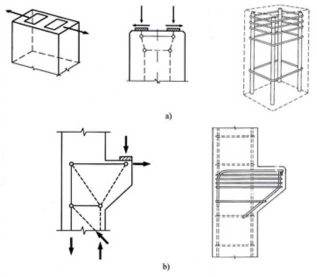
## Role of strut and tie models in the design of connections



The connection zones of precast elements are typically discontinuity regions. The strut and tie method can be used to study the flow of forces and as a basis for designing and detailing:

a) design and detailing of a column head,

b) design and detailing of a column corbel



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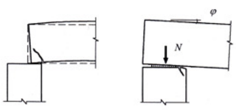
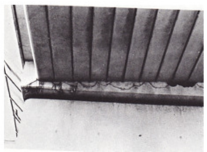
## The "simple" support





Damage due to disregarding deformations

Damage due to incorrect detailing at temporary support (2012)



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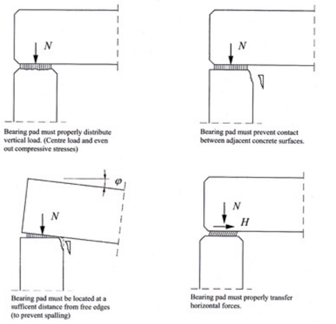
## The "simple" support

The function of bearing pads



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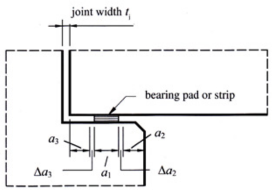
## Types of connections between precast elements




Bearing materials harder than concrete are checked at the ultimate limit state (ULS) whereas bearing materials softer than concrete are checked at the serviceability limit state (SLS).

The design and dimensioning of the support and supported members at a bearing should take into account the anchorage requirements and the necessary dimensions of bends of the reinforcement in the members. Members should be dimensioned and detailed in order to assure correct positioning, accounting for production and assembling tolerances



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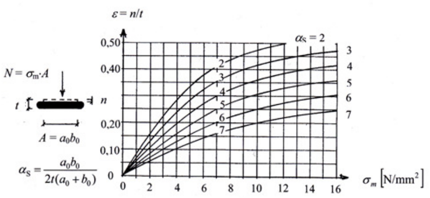
## Types of connections between precast elements

Behaviour of plain elastomeric bearing pads




Relation between strain of elastomeric bearing and average compressive stress



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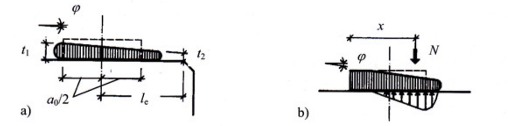
## Types of connections between precast elements




1. The bearing should be designed so that there will be compression over the entire face of the bearing pad: that means:

$$t_1 \le t \qquad \varphi\frac{a_0}{2} \le \varepsilon \cdot t \qquad t \ge \frac{\varphi a_0}{2\varepsilon}$$

2. The thickness t must be designed to prevent direct contact between the surfaces of the concrete members:

$$t_2 = t - \varepsilon \cdot t - \varphi \cdot l_e$$



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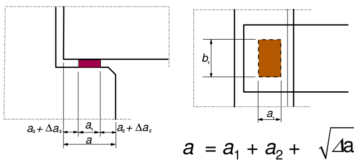
## Bearings: definitions (10.9.5)



$$a = a_1 + a_2 + \sqrt{\Delta a_2^{\,2} + \Delta a_3^{\,2}}$$
$a_3 +$

$a_1$ nett bearing length = $F_{Ed}$ / ($b_1$ $f_{Rd}$), but ≥ minimum value
- $F_{Ed}$ design value of support reaction
- $b_1$ nett value of bearing length
- $f_{Rd}$ design value of maximum bearing pressure

$a_2$ ineffective length from edge of bearing
$a_3$ same for supporting member
$\Delta a_2$ allowable tolerance for distance between bearings (Table 10.5)
$\Delta a_3$ allowable tolerance in length $L_n$ of precast member: $\Delta a_3 = L_n/2500$



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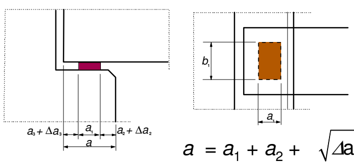
## Bearings (10.9.5)



$$a = a_1 + a_2 + \sqrt{\Delta a_2^{\,2} + \Delta a_3^{\,2}}$$
$a_3 +$

Limits to support pressure

$f_{Rd} = 0{,}4\, f_{cd}$ for dry connections
$f_{Rd} = f_{bed} \le 0{,}85 f_{cd}$ for all other cases

where
$f_{cd}$ lowest value of design strength of supporting member
$f_{bed}$ design strength of bearing material



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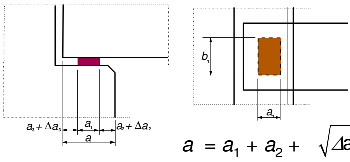
## Bearings (10.9.5)



$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}$

Minimum value $a_1$ in mm

| Relative bearing pressure, $\sigma_{Ed}/f_{cd}$ | ≤ 0,15 | 0.15-0,4 | > 0,4 |
|---|---|---|---|
| Line support (floors, roofs) | 25 | 30 | 40 |
| Ribbed floors and purlins | 55 | 70 | 80 |
| Concentrated supports (beams) | 90 | 110 | 140 |



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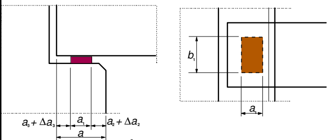
## Bearings (10.9.5)



$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}$

Distance $a_2$ in mm

| Bearing material and – type | $\sigma_{Ed}/f_{cd}$ | ≤ 0,15 | 0,15 – 0.4 | > 0.4 |
|---|---|---|---|---|
| Steel | line support | 0 | 0 | 10 |
| | concentrated | 5 | 10 | 15 |
| Reinforced concrete ≥ C30 | line support | 5 | 10 | 15 |
| | concentrated | 10 | 15 | 25 |
| Plain and reinforced concrete < C30 | line support | 10 | 15 | 25 |
| | concentrated | 20 | 25 | 35 |
| Masonry | line support | 10 | 15 | (-) |
| | concentrated | 20 | 25 | (-) |



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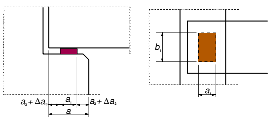
## Bearings (10.9.5)



$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}$

Distance $a_3$ in mm

| Reinforcement detailing | Line support | Concentr. support |
|---|---|---|
| Continuous bars over support | 0 | 0 |
| Straight bars, horizontal loops at end of member | 5 | 15, but ≥ end cover |
| Tendons or straight bars exposed at end of element | 5 | 15 |
| Vertical loop reinforement | 15 | Cover + inner radius of bending |



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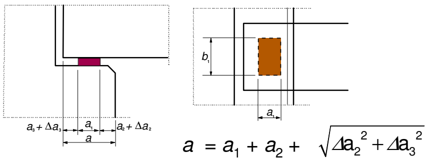
## Bearings (10.9.5)



$$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}$$

Allowance $\Delta a_2$ for tolerances for the clear distance between the faces of the supports:

| Bearing material | $\Delta a_2$ |
|---|---|
| Steel or precast concrete | $10 \le L/1200 \le 30$ mm |
| Masonry or in situ concrete | $15 \le L/1200 + 5 \le 40$ mm |



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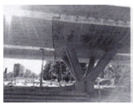
## Problem with "simply" supported precast bridge elements

Precast bridge Utrecht, The Netherlands (2012)





Problem: prestressed precast beams longer than expected. A case for concern?



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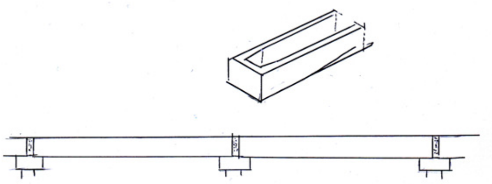
## Proble with "simply" supported prestressed roof beams

Near collapse of the roof of a super market, the Netherlands



Prestressed U-shaped beams supported on transverse beams



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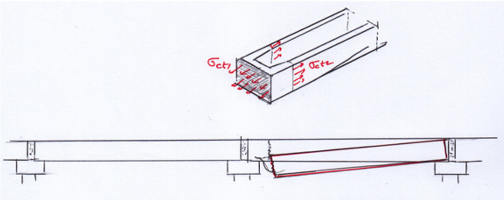
## Types of connections between precast elements

Near collapse of the roof of a super market, the Netherlands: a case of restrained deformation



Tensile strength concrete in joint $f_c$ = 2,5 N/mm$^2$
Tensile strength concrete of slab element $f_c$ = 6,0 N/mm$^2$



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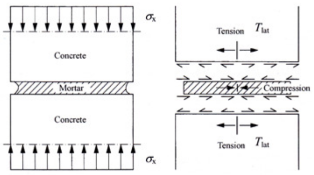
## Connections between vertical elements

mortar joint




Compression through concrete and mortar joint: the mortar quality of the joint will generally be lower than that of the precast column. The mortar, thus having a higher lateral strain ν/E, will cause lateral tensile stresses in the element close to the joint. On the other hand the mortar is confined, which increases its strength.



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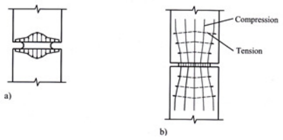
## Connections between vertical elements

mortar joint




The capacity of the joint is influenced by the geometry and the difference in strength between joint mortar and column concrete. The design compressive capacity of the joint is:

$$N_{Rd,joint} = f_{cd,joint} \cdot A_{joint} = \beta \cdot f_{cd,wall} \cdot a_1 \cdot l$$

Where $f_{cd,wall}$ = design compressive strength of wall concrete cylinder
$f_{cd,mortar}$ = design compressive strength of joint mortar cylinder
$f_{cd,joint}$ = design compressive strength of actual joint
$\beta_0 = f_{cd,mortar} / f_{cd,wall}$
$\beta = f_{cd,joint} / f_{cd,wall}$



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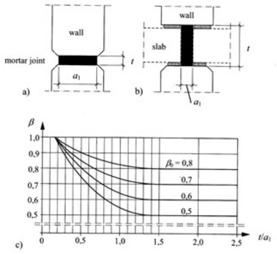
## Connections between vertical elements: plain mortar joint

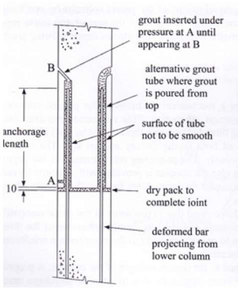



Determination of joint effectiveness factor $\beta$





## Connections between vertical elements: reinforced mortar joint

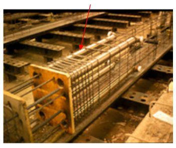


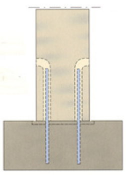






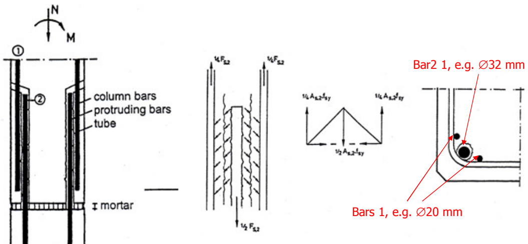
## Connections between vertical elements: reinforced mortar joint

This connection posesses most of the advantages (confinement of concrete, thin dry packed joint, continuity of high tensile reinforcement, easy to manufacture and fix) and a few disadvantages (need for temporary support and accuracy in projecting bar position) associated with precast construction methods. Splices may be made in this way at virtually any level in the frame and are not restricted to column-to-column connections. The grout sleeves may be formed in smooth or corrugated steel tubes.

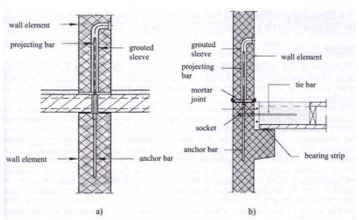
## Reinforced mortar joint: principle of splicing




Bar 1: reinforcement of upper column
Bar 2: reinforcement from lower column, intruded in corrugated steel tube



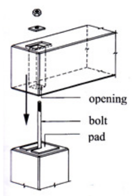
## Reinforced mortar joint: other applications



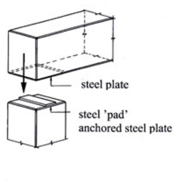

Examples of wall to wall connections with projecting bars in grouted ducts, a) interior connection, b) exterior connection



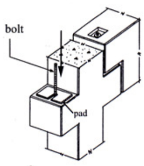
## Connection between vertical and horizontal elements




Beam-column (pad)

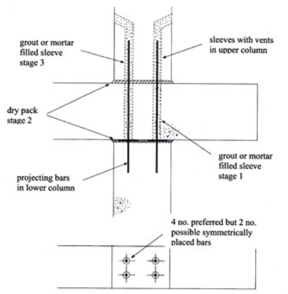



Beam- column (steel)




Beam-corbel (pad)

## Connections between horizontal and vertical elements




This connection is mainly used in portal frames. It may be used as well in skeletal frames where continuous (or cantilevered) beams are required.



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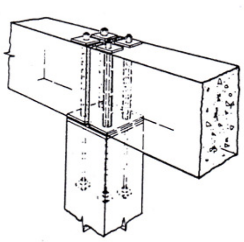
## Connections between horizontal and vertical elements



Torsion resistant connection between column and beam.



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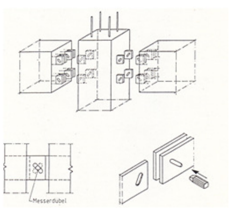
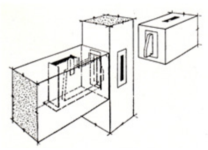
## Connections between vertical and horizontal elements: particular solution: the hidden corbel



Dowel – steel plate cantilever



Spencon system (Norway)



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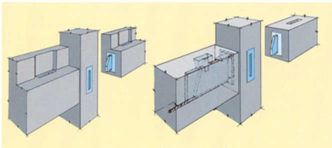
## Hidden corbels: the Spencon system (Norway)





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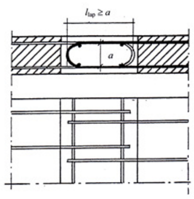
## Tensile connections between horizontal elements

In floors, which should function as a diaphragm which is able to submit tensile forces (e.g. caused by wind tension at the facades) tensile connections between the elements are required.




The "loop"- connection is able to transfer tensile forces, bending moments and shear forces. It is used for solid slabs where continuity is demanded.
The connection can fail due to rupture of the reinforcing bars, crushing or splitting of the joint concrete in the plane of the overlapping loops. The design aims at preventing concrete failures to occur before the reinforcement loops yield. Transverse reinforcement within the overlap is necessary inorder to achieve an acceptable behaviour. If properly designed, the loop connection can exhibit substantial ductility.



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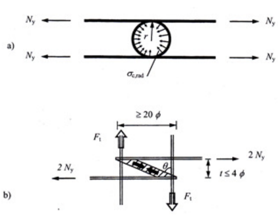
## Tensile connections between horizontal elements




It is recommended that the detailing should be such that :

$$r \geq \frac{\pi\phi}{4} \cdot \frac{f_{sy}}{4\sigma_{c,rad}}$$

The tensile force in one U-bar is balanced in the joint by the radial concrete stresses. From equilibrium it follows that

$$\sigma_{c,rad} = \frac{\pi\phi}{4r} f_{sy}$$

where r = radius of bend of the U-bar
∅ = diameter of the U-bar

The radial concrete stress $\sigma_{c,rad}$ should be limited to:

$$\sigma_{c,rad} \leq f_{cc}\sqrt{\frac{b_i}{\phi}}$$ not greater than $3f_{cc}$

where $b_i = 2 \cdot (c_e + \frac{\phi}{2})$ not less than $t$

and $c_e$ = concrete cover between U-bar and edge of element.



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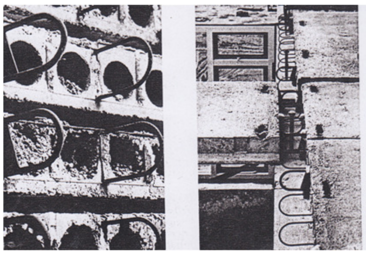
## Types of connections between precast elements

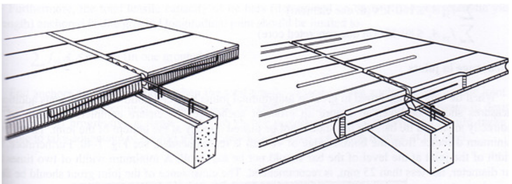


Loop connections in solid concrete precast decks





## Tensile connections between horizontal elements

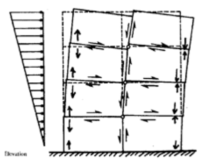


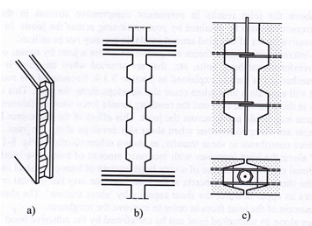
Indirectly anchored tie bars in hollow core floors, a) anchorage in longitudinal joints, b) anchorage in cores opened by a slot in the top





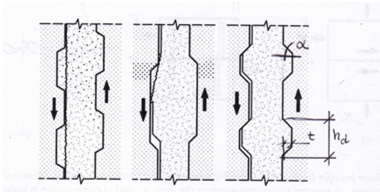
## Shear joints

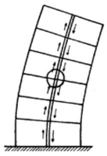



Connection at vertical joint between wall elements
a) indented joint face of wall element,
b) transverse, tying reinforcement concentrated to the end of the wall element (in the horizontal joint),
c) transverse, overlapping loops distributed along the joint

## Shear joints between precast concrete panels

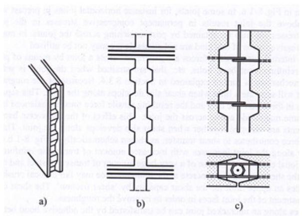


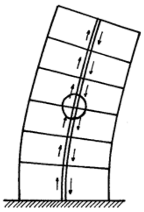



Failure modes:
- (left) Crack formation due to shrinkage: resistance depends on shear friction of rough crack areas.
- (middle) Crushing of concrete in compression: this will most probably not occur if $h_d/t < 6$, where $h_d$ = height of indentation, t = depth of indentation
- (right) Sliding: will not occur if $N \geq V \cdot \tan(\alpha - \arctan\mu)$, where $\alpha$ = angle of indentation, $\mu$ = coefficient of friction concrete to concrete



## Shear joints between precast concrete panels

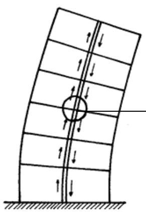


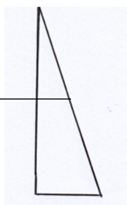



The clamping force N can be provided in two ways:
- Reinforcing bars in the horizontal joints between the panels (b)
- Looped bars in the vertical (shear joints) between the panels (c)



## Shear joints between precast concrete panels





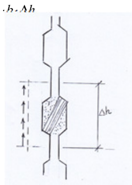
Shear force $V_{ed,max}$

Shear stress at height x:

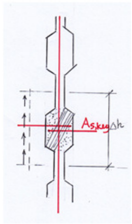
$$v_{Ed} = \frac{V_{Ed,x} \cdot S}{b \cdot I}$$

Shear force per shear key:

$$V_{Ed,key} = \left(\frac{V_{Ed,x} \cdot S}{b \cdot I}\right) \cdot b \cdot \Delta h$$

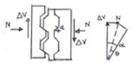


## Shear joints between precast concrete panels



Sufficient shear capacity if:

1. No sliding:

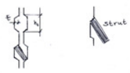


$$N \geq \Delta V_{Ed} \cdot \tan(\alpha - \arctan \mu)$$

where $N = A_{s,key} f_{yd}$

and $\arctan\mu = 30^0$

2. No concrete crushing in joint

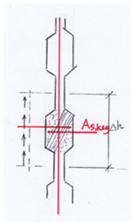



$$\frac{h_t}{t} \leq 6$$

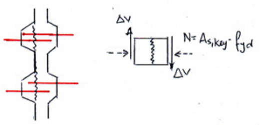
ir $\sigma_c \leq 0{,}6 f_{cd}$



## Shear joints between precast concrete panels



3. No shear failure of cracked interfaces $N = A_{s,key} f_{yd}$



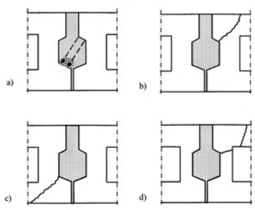

Maximum shear stress of cracked area:

$$\Delta V_{Ed} \leq c + (A_s f_{yd})\mu$$

According to EC2: $c$ = 0,40 and $\mu$ = 0,70



## Shear capacity of joints between horizontal elements



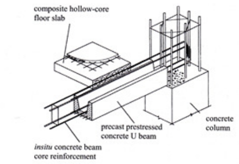

In general: mortar in confined condition (enclosed in joint) is not governing for shear capacity.

Possible failures at longitudinal floor connections subjected to vertical shear:

a) Crushing of joint grout
b) Failure in upper corner of slab element
c) Failure in lower corner of slab element
d) Failure influenced by core

## Moment resisting connections

Moment resistant connections should be proportioned such that ductile failures will occur and that the limiting strength of the connection is not governed by shear friction, short length of weld, plates embedded in thin sections, or other similar details which may lead to brittleness. Many of the principles behind these requirements have evolved through years of seismic R&D, and the common practice in the U.S., Japan and New Zealand is often to design and construct moment resisting connections in the perimeter of the frame, where there are less size restrictions on beams and columns. Moment resisting portals, such as the multi-legged U-frames shown in Fig. 9-1 may be used to provide peripheral framework to what is otherwise a pinned-joint structure.

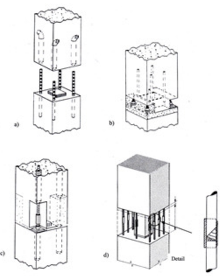



Mixed precast and in-situ concrete used to create moment resisting connections





## Moment resistant connections



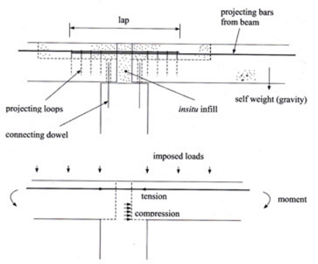

| Connection identification | On site jointing action | Advantages | Disadvantages | Approximate max. moment capacity* |
|---|---|---|---|---|
| Column base plate | Bolted | Immediate column stability. | High accuracy in setting bolt position. Cost of bolts | 400 kNm |
| Column pocket | Grouted | Low cost of foundation and column. No additions to column | Temporary propping required | 250 kNm |
| Column grouted sleeve | Grouted | Low cost of foundation. Easy to construct | Accuracy in projecting rebar positions. Temporary propping required | 150 kNm |
| Coupled column splice | Coupler | Immediate column stability. | Expensive. Difficult to construct | 100 kNm |
| Welded plate column splice | Bolted | Immediate column stability. Easy to construct | Expensive | 50 kNm |
| Grouted sleeve column splice | Grouted | Low cost. Easy to construct | Accuracy in projecting rebar positions. Temporary propping required | 100 kNm |
| Grouted sleeve coupler splice | Grouted | Easy to construct. Splice length short | Expensive. Temporary propping required. Pressure grouting | 100 kNm |
| Steel shoe column splice | Bolted | Immediate column stability. Easy to construct. Grout quantity small | Expensive | 175 kNm |
| Welded plate beam connector | Welded | Immediate fixity. Large tolerances | Site weld inspection. Large volume of grout | 300 kNm ** |
| Steel billet beam connector | Bolted | Immediate fixity. Easy to fix. Grout quantity small | Tolerances | 200 kNm ** |
| Bolted plate beam connector | Bolted | Immediate fixity. Grout quantity small | Expensive. Relatively difficult to fix. Tolerances | 250 kNm ** |
| Portal frame eaves connection | Bolted | Inexpensive. No grouting. | Tolerances | 150 kNm |
| Wall to floor connection | Grouted | Inexpensive. Large tolerances. Grout quantity small | Wet bedding and temporary propping required. | 50 kNm/m |

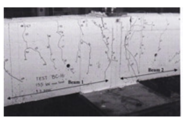
* based on members up to about 400 x 400 mm in cross section. ** includes 200 mm deep floor slab and ties.





## Moment resistant connection between horizontal elements

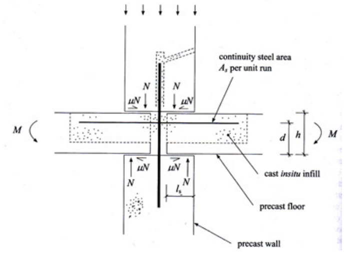


Full scale test

Principle of moment-resisting connections in precast frames. Moment-continuity exists only for imposed loads after the in-situ infill has matured to full strength

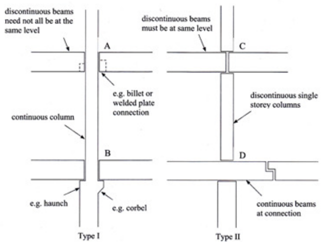
## Beam column connection




Typical example of floor–wall-floor connection designed to have full or partial capacity for moment in the floor



## Moment resistant beam-column connections

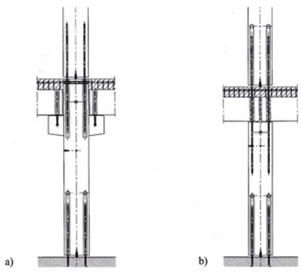



Generic types of beam-column connections :

A) Beam end hidden connection to continuous column
B) Beam end to column corbel
C) Discontinuous beams and column
D) Column to continuous beam



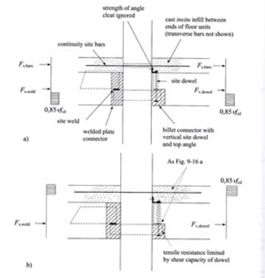
## Moment resistant beam-column connections




Examples of internal moment resisting beam column connections:

a) Beam-end connection to continuous column with corbels
b) Beam to column head connection with discontinuous beam and column

## Moment-resistant steel column connections

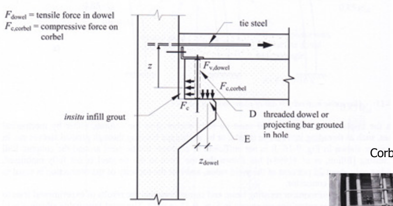



Load transfer mechanism through beam-end to column connection:
a) Hogging moment, b) Sagging moment

This connection is investigated frequently. Although often classiefied as "semi-rigid", in that the moments of resistance are accompanied by beam to column rotations, the stiffnesss is sufficiently large to make sure that the connection is efficiently fully-rigid. In many cases the rotational ductility of the connection is equal to or greater than the curvature capacity of the beams and the columns.

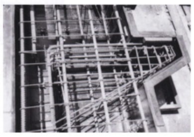


## Moment resistant steel column connections



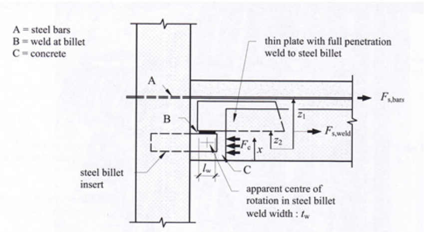

Structural mechanism for the beam end connection with a concrete corbel

Corbel reinforcement



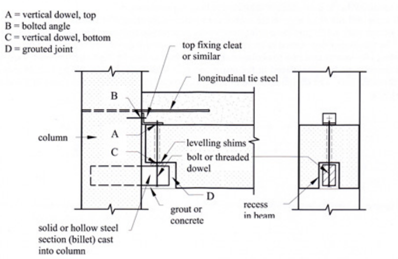


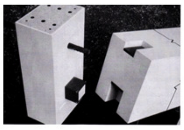
## Moment resistant steel column connection




Structural mechanism for the beam-end connector with the welded plate connector

## Moment resistant steel column connection



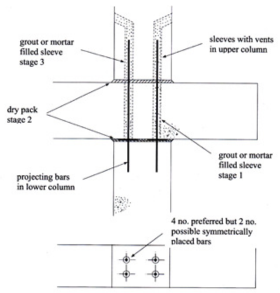
Model construction of a single billet connection with welded top bar (boltet cleat or grouted joint options possible)

Structural mechanism for the beam-end connection with steel billet connector





## Moment resistant steel column connection

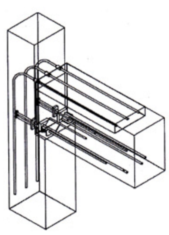



This connection is mainly used in portal frames. It may be used as well in skeletal frames where continuous (or cantilevered) beams are required.

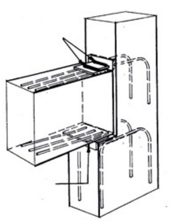




## Moment resistant beam column connections

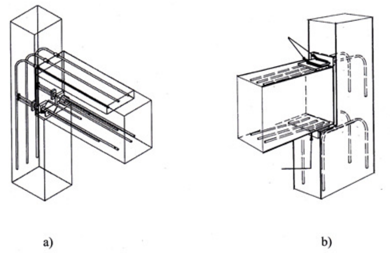


a)



b)

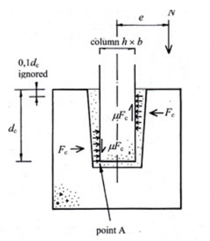
## Tensile connections between horizontal elements



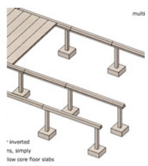

Column haunch connections: a) threaded bars and bolting b) weld plates
(PCI Design Manual Connections)



## Columns in pockets



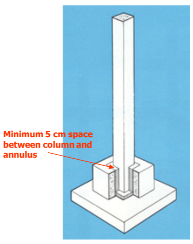

Column in pocket foundation: basic mechanical behaviour

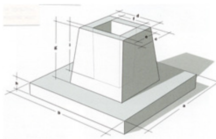


Precast concrete pocket foundation, with castellations for enhanced bond to the column



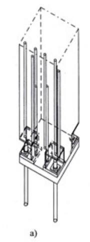
## Pocket foundations

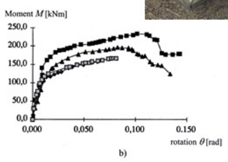


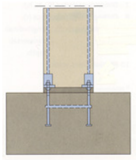



Indirectly

## Column to foundation connections





a) Column-base connection with steel shoes
b) moment-rotation relationship for various detailing solutions
(Bergström, 1994)



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