Connections between precast elements: types, design, application

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 manufacturing 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 misconception 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.

Basic considerations

General considerations

The following design aspects play a role:

- Standardization
- 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

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

The "simple" support

Damage due to disregarding deformations

Damage due to incorrect detailing at temporary support (2012)
The "simple" support

The function of bearing pads

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.

Types of connections between precast elements

Behaviour of plain elastomeric bearing pads

Relation between strain of elastomeric bearing and average compressive stress
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:
   \[ t_s \leq \frac{v}{2} \] and \[ t \geq \frac{v}{2} \]

2. The thickness \( t \) must be designed to prevent direct contact between the surfaces of the concrete members:
   \[ t_s = t - \Delta t - \varphi \cdot \frac{v}{2} \]

Bearings: definitions (10.9.5)

\[ a = a_1 + a_2 + \sqrt{a_3^2 + a_4^2} \]

- \( a_1 \): nett bearing length = \( F_{Ed} / (b_1 f_{Rd}) \), but \( \geq \) minimum value
- \( b_1 \): design value of support reaction
- \( n_1 \): 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: \( \Delta a_3 = L_n / 2500 \)

Limits to support pressure

\[ f_{Rd} = 0.4 f_{cd} \quad \text{for dry connections} \]
\[ f_{Rd} = f_{bed} / 0.85 \quad \text{for all other cases} \]

where
- \( f_{cd} \): lowest value of design strength of supporting member
- \( f_{bed} \): design strength of bearing material
Bearing material and – type | $n_{p/a}$ | ≤ 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 | (-)

Distance $a_2$ in mm
- Steel: line support 0, concentrated 5
- Reinforced concrete ≥ C30: line support 5, concentrated 10
- Plain and reinforced concrete < C30: line support 10, concentrated 20
- Masonry: line support 10, concentrated 20

Distance $a_3$ in mm
- Reinforcement detailing: Line support 0, Concentr. support 0
- Continuous bars over support 0
- Straight bars, horizontal loops at end of member 5, but ≥ end cover
- Tendons or straight bars exposed at end of element 5
- Vertical loop reinforcement 15, Cover + inner radius of bending
Problem with "simply" supported precast bridge elements
Precast bridge Utrecht, The Netherlands (2012)

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

Bearing (10.9.5)

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

<table>
<thead>
<tr>
<th>Bearing material</th>
<th>$\Delta a_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel or precast concrete</td>
<td>10 ≤ $L/1200 ≤ 30$ mm</td>
</tr>
<tr>
<td>Masonry or in situ concrete</td>
<td>15 ≤ $L/1200 + 5 ≤ 40$ mm</td>
</tr>
</tbody>
</table>

Problem with "simply" supported prestressed roof beams
Near collapse of the roof of a supermarket, the Netherlands

Prestressed U-shaped beam supported on transverse beams
Types of connections between precast elements

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

![Diagram of connections between precast elements](image)

Tensile strength concrete in joint $f_t = 2.5 \text{ N/mm}^2$
Tensile strength concrete of slab element $f_t = 6.0 \text{ N/mm}^2$

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 $\nu/E$, will cause lateral tensile stresses in the element close to the joint. On the other hand, the mortar is confined, which increases its strength.

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:

$$ N_{j,\text{design}} = f_{c,d,\text{mort}} A_{j,\text{design}} = \beta f_{c,d,\text{wall}} A_{j,\text{design}} $$

Where:
- $f_{c,d,\text{wall}}$ = design compressive strength of wall concrete cylinder
- $f_{c,d,\text{mort}}$ = design compressive strength of joint mortar cylinder
- $\beta$ = $f_{c,d,\text{mort}}/f_{c,d,\text{wall}}$
Connections between vertical elements: plain mortar joint

Determining joint effectiveness factor $\beta$

Connections between vertical elements: reinforced mortar joint

This connection possesses most of the advantages (confinement of concrete, 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

Bar 1, e.g. ∅20 mm
Bar 2, e.g. ∅32 mm

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.

Connections between horizontal and vertical elements

Torsion resistant connection between column and beam.

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

Dowel – steel plate cantilever
Spencer system (Norway)
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 failure to occur before the reinforcement loops yield. Transverse reinforcement in the overlap is necessary inorder to achieve an acceptable behaviour. If properly designed, the loop connection can exhibit substantial ductility.

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

\[
\sigma_{\text{c,rad}} = \frac{f_{\text{crad}}}{\pi \phi}
\]

where \( r \) = radius of bend of the U-bar  
\( \phi \) = diameter of the U-bar

The radial concrete stress \( \sigma_{\text{c,rad}} \) should be limited to

\[
\sigma_{\text{c,rad}} \leq \frac{f_{\text{r}}} {\pi \phi}
\]

not greater than \( f_{\text{r}} \)

where \( b = 2 \cdot \phi \)  
and \( f_{\text{c}} = \) concrete cover between U-bar and edge of element.
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 $h_d = \text{height of indentation}$, $t = \text{depth of indentation}$
- (right) Sliding: will not occur if $N \geq V/\tan(\alpha - \arctan(\mu))$, where $\alpha = \text{angle of indentation}$, $\mu = \text{coefficient of friction concrete to concrete}$

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 stress at height $x$:
$$\tau_{\text{shear}}(x) = \frac{V_x}{b}$$

Shear force per shear key:
$$V_{\text{shear}} = \frac{V_x}{b}$$
Shear joints between precast concrete panels

Sufficient shear capacity if:

1. No sliding:
   \[ N \geq \Delta V_{dy} \tan(\alpha - \arctan \mu) \]
   where \( N = A_{s, key} f_{yd} \)
   and \( \arctan \mu = 30^\circ \)

2. No concrete crushing in joint
   \[ \frac{h}{\bar{t}} \leq 6 \]
   \[ \bar{t} \leq 0.6 f_{cd} \]

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

Maximum shear stress of cracked area:
   \[ \Delta V_{dy} \leq c (A_{s, key} f_{yd}) \mu \]
   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 resisting 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

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

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

This connection is investigated frequently. Although often classified as “semi-rigid”, in that the moments of resistance are accompanied by beam to column rotations, the stiffness 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.

Load transfer mechanism through beam-end to column connection:
- hogging moment
- sagging moment

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

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

Columns in pockets

Column in pocket foundation: basic mechanical behaviour

Pocket foundations

Indirectly
Column to foundation connections

a) Column-base connection with steel shoes
b) Moment-rotation relationship for various detailing solutions

(Bergman, 2004)