Basic Forces Transfer Mechanism for Design of Structural Precast Connections

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The connection characteristics can be categorized by the type of action it is designed to resist:
- Shear
- Tension
- Compression
- Flexure
- Torsion

For many structural connections the behavior is dominated by one of the actions above. Sometimes connections are classified by this dominating action as ‘shear connections’, ‘compression connections’, etc. However, very often the structural connections should be capable to transfer a combination of these basic actions.

Many structural connections should be able to transfer more than one type of basic action. For instance connections at the short ends of floor elements may need, besides the primary support action, both shear resistance along the support and tensile resistance across the joint. In support connections it may also be necessary to combine the ability to transfer forces with the need for movement.

Design of connections with regard to the ability to transfer forces must be based on the knowledge and understanding of basic force transfer mechanisms. Some of these are specific in precast structures. Basic for transfer mechanisms are presented in this article.

Transfer of Compressive Forces

Every precast concrete element has to be supported at one or several locations in order to transfer its own weight and imposed loads down to the foundations. These forces will normally be compressive forces.

Typical connections with compressive forces are shown in fig.1. Small bearing areas lead to small eccentricities, which is normally of great advantage. Large forces, or practical considerations, may however require larger bearing areas.

Compression Joints with Combined Action

Wind load, earthquake load/earth pressure, will in some cases change the compressive forces into tensile forces, or impose horizontal (shear) forces on the connection. Thus, compression joints must often be checked for shear forces, and required reinforcement or other steel components across the joint in addition to the joint bearing material.

Long horizontal members, such as beams or slabs, will rotate at the support following the variation due to temperature change, creep and shrinkage. The rotation often requires bearing pads and strips with special attention to detailed design of pad thickness and edge distance.

Selection of Bearing Type & Material

The bearing material is mainly designed for vertical & horizontal loads, and for rotation and lateral movements. The size of bearing area and joint opening are, however, very often determined by the size of concrete elements, erection tolerances and architectural considerations. The type of
bearing material is also depending upon local availability and economy.

**High Compressive Force without Lateral Movement and/or Rotation**

Connections with high compressive forces without lateral movement and/or rotation require construction steel (steel plates/bars) across the joint with properly designed field – bolting or welding in the connection area. The steel components should be properly anchored in the concrete member to secure transfer of compressive forces to the main reinforcement. This type of solution is mostly needed for connections such as: beam-column, column-column, column-foundation, moment frame or lateral bracing.

**Medium Compressive Forces without Lateral Movement and/or Rotation**

These types of connections are typical for one story columns or load bearing walls. Normally the column or wall is placed on erection shims and the joint is 90-100% grouted. Steel bar or reinforcement across the joint are normally designed for tensile or shear force only, but they can also be utilized as compressive reinforcement.

**High & Medium Compressive Forces with Lateral Movement and/or Rotations**

Connections with high and medium compressive forces with lateral movement and/or rotation are typical for the support of all types of beams, girders and T shape slabs. Typically these type of connections is provided with bearing pads.
Lateral movement will only occur as slippage, which only means that a horizontal force $H=\mu N = 0.2 N$ to $0.5 N$.

Moderate Compressive Forces with Little Rotation And Separate Transfer of Horizontal Forces

These type of connections are typical for the support of the compact slabs or HCS slabs will often require bearing strips sustaining compressive stress of magnitude 1 to $4N/mm^2$. Materials are for example sponge rubber, neoprene pad and hard plastic.

Design

Lateral Expansion

General formulas for lateral expansion

Lateral strain

$$\varepsilon_y = \nu \frac{\sigma_y}{E}$$

Fig.7: Uni-axial compression of concrete cube

Fig.6: Reasons for using bearing pods

Bearing pod must properly distribute vertical load. (Centre load and even out compressive stresses)

Bearing pod must prevent contact between adjacent concrete surfaces

Bearing pod must be located at a sufficient distance from free edges (to prevent spalling)

Bearing pod must properly transfer horizontal forces

Fig.8: Mechanism of ‘confinement’ offered by the surrounding mass of concrete, [CEB-FIP (1992)]

Triaxial effect

$$fcc^* = f_{cc} + 5p = f_{cc} + 0.5f_{cc} \times \frac{d_2-d_1}{d_1}$$

Compressive Stress Control

$$f_{cd^*} = f_{cd^*} \sqrt{A_2/A_1} < 4.0f_{cd}$$

$$f_{cd} = \text{Concrete design compressive strength}$$

$$f_{cd^*} = \text{Bearing capacity}$$

Joint Bearing Capacity

$$NRd = \beta f_{cd^{wall}} * a_1^* l$$

$$f_{cd^{wall}} = \text{Concrete design compressive strength of wall}$$

$$f_{cd^{mortar}} = \text{Concrete design compressive strength of mortar}$$

$$\beta = \frac{f_{cd^{mortar}}}{f_{cd^{wall}}}$$

$$\beta = \frac{f_{cd^{joint}}}{f_{cd^{wall}}}$$

$$\sigma_y = \frac{N}{A}$$

$\nu = 0.2$ is accepted value for concrete

General Failure Mode of Concrete

Crushing of concrete

$$fcc^* = 4xf_{cc} \text{ for circular or square loaded area}$$

$f_{cc} = \text{compressive strength of concrete under uniaxial stress}$

Splitting Failure
Transfer of Tensile Force

When structural connections are designed to be tensile resistant, it must be presumed that the joint section is cracked. The tensile forces acting across the joint should be resisted by certain tie arrangements.

A tensile resistant connection can be achieved either by ‘continuous’ tie bars that are placed continuously across a joint and anchored in precast units on each side of joint, or by ‘protruding’ tie bar or other tensile resistant devices that are anchored in the respective elements and connected in the joint by bolting, welding or lap splicing.

Reinforcement bars or loops that are protruding from the respective elements can be connected by lap splicing in the intermediate joint.

The connection is activated as the joint is filled with grout or concrete. Lap splicing of reinforcement loops is a classical way to obtain tensile capacity through joints between precast elements. The splitting effect is large in the plane of loop. To prevent premature brittle failure of the connection, transverse reinforcement should be placed through the overlapping part of the loops. With such a solution, a ductile behavior can be obtained.

The tensile capacity of the connection depends on the capacity of tie bars, connection details, welds etc, but also on the anchorage of the steel details in the concrete elements. Anchorage can be obtained by bond along ribbed bars or by various types of end anchors.

Continuous tie bars can also be cast in place with protruding ends in one of the elements. At erection the next element, provided with sleeves, is placed to match the protruding tie bars, which are anchored indirectly in this element by grout, glue etc.

When anchor bars anchored by bond, ordinary ribbed or indented reinforcement bars are normally used. In anchorage by bond, tangential tensile stresses appear in the concrete around the bar. By providing sufficient concrete covers and anchorage length, the anchorage capacity can exceed the tensile capacity of the bar. The anchorage can be lost by splitting failure in the concrete cover or by pullout failure. The anchorage capacity can be estimated by ordinary methods for reinforcing bars. Then the upper limit of bond strength corresponds to the capacity at pullout failure. In design for ductility, enhanced requirements of the anchorage may be needed, since the anchorage capacity in this case should exceed the tensile capacity of the connection at steel rupture.
Anchor Bar Behaviour & Failure Modes

For anchor bars provided with ribs, indentations or threads a tensile force applied to the bar is transferred to the steel/concrete interface to the surrounding concrete by bond. This is in general a favourable way to anchor connection details, since the tensile force is transferred successively along the anchorage length and high stress concentrations can be avoided.

Fig.14: Anchorage failures of ribbed anchor bar, a) splitting failure, b) pullout failure

No anchorage is perfectly rigid, but the bond transfer results in a certain slip between the anchor bar and the surrounding concrete. It should be noted that the bond stress along the steel/concrete interface are not normally uniformly distributed and, accordingly, the slip varies along the anchorage length. This means that the slip at the loaded end of the bar exceeds the slip at the passive end. It can even be the case that the active end has a slip, while the passive end is still firmly fixed without any slip at all. Accordingly, the anchor bar should not be regarded as a rigid body.

The total elongation of the steel bar in relation to the concrete can be recognized as the slip at the loaded end, the so-called ‘end-slip’. In the actual case the active end of the bar has a certain slip, but the passive end of the bar has no slip at all.

Failure can take place in the concrete or in the steel. In case of small concrete covers, the anchorage can fail due to splitting of the concrete. If the concrete cover is sufficient to prevent a splitting failure, the anchor bar can lose its grip in the concrete by a shear failure that develops along the interface starting from the loaded end. This failure mode is referred to as a pullout failure.

It is stated that the anchorage condition can be considered as ‘well confined’, when the concrete cover is 5 times the bar dimension.

In ordinary design the steel strength is based on the yield strength, but in case ductility is important the connection detail should be designed so that the tensile capacity at rupture can be safely anchored. In that case the plastic behavior of the bar can be fully utilized.

Near the free edge, inclined cracks starting from the ribs of the anchor bar develop towards the edge and may cause a local concrete cone failure as indicated in fig. The depth of the cone have been about 2 times the anchor bar diameter.

Fig.15: Typical anchorage behaviour of an anchor bar loaded in tension and typical distributions of steel stress and bond stress for a) a small tensile force, b) an intermediate tensile force. Dotted lines indicate possible effect of local concrete failure near the free edge

Transfer of Shear Force

Principle for Shear Force Transfer

Shear force can be transferred between concrete elements by adhesion or friction at joint interfaces, shear-key effect at indented joint faces, dowel action of transverse steel bars, pin & bolts, or by other mechanical connection devices. The frictional resistance can be enhanced by the pullout resistance of tie bars properly placed across the joint.

When a joint face has a certain roughness, shear force can be transmitted by friction even if the joint is cracked. In some joints, for instance horizontal joints in precast walls, the weight of the wall above the joint results in permanent compressive stresses in the joint. Permanent compressive stresses can also be obtained by post-tensioning across the joints. In many applications, external compressive forces of this kind are not available and may not be utilized.

Generally, internal compressive forces are generated across a joint by means of pull resistance of transverse reinforcement bars, bolts, etc. that are strained when the joint is subjected to shear sliding. Because of the roughness of the joint faces, the joint will separate a little when shear slip develops along the joint. This separation results in tensile stresses in the transverse bars and the resulting tensile forces must be balanced by a compressive force of the same magnitude action across the joint. This effect of the transverse bars means that the adjacent elements are clamped together when shear slip develops along the joint.
This self-generated compressive force contributes to shear transfer, as shown in Fig. The shear force capacity along the joint increases with increased amount of transverse reinforcement and with increased frictional coefficient. In case of a very large amount of transverse steel and depending on the magnitude of shear action, the concrete at the joint interface may fail in local crushing. This failure mode constitutes an upper limit for the shear capacity by ‘shear friction’. The shear capacity can also increase by treatment of the joint faces in order to improve the roughness.

Shear force along an uncracked joint can be transferred by the adhesive bond between joint grout and the adjacent concrete elements. The adhesive bond, however, depends to a large extent on the workmanship and cleanliness of the joint faces during grouting. If the joint faces are dirty from sand, cement or oil wastes, the adhesive bond can be entirely lost.

This means that in practice, it is not possible to rely on adhesive bond for shear transfer, but the joint must be assumed to be cracked and the shear transfer must be secured by shear friction, shear keys, or mechanical devices.

Shear keys are generally formed by providing the precast members with indented joint faces. When this type of connection is loaded in shear along the joint, the shear resistance depends on the strength of the shear keys on condition that transverse reinforcement or other tie arrangements are provided. The shear keys work as mechanical locks preventing any significant slip along the joint. To function in the intended way, the shear key must fulfill certain minimum requirements concerning tooth length, tooth depth and tooth inclination. The joint should also be prevented from controlled joint separation by transverse reinforcement or other transverse tie arrangements. The transverse steel can be distributed along the joint or, under certain conditions, be concentrated to the ends of joint, according to Fig.

A connection with indented joint faces has a very stiff behavior until the shear-key effect is destroyed by cracking or local crushing at the heaviest loaded contact areas. When the shear-key effect decreases due to this degradation of the shear keys, the behavior changes to a frictional phase associated with a significant shear slip along the cracked section.

Reinforcement bars, bolts, studs, etc., which are placed across joints, can also contribute to the shear resistance by their dowel action due to imposed shear displacements. The ‘dowel’ is loaded by shear in front of the joint and is supported by a contact pressure along the part that is embedded in the concrete element. This loading condition normally results in considerable flexural deformation and flexural stresses in the ‘dowel’. Various failure modes are possible. For normal dimensions and strengths, a collapse mechanism develops by formation of one or more plastic hinges in the dowel. Simultaneously, local crushing occurs in the surrounding concrete where the contact pressure is high. If the dowel is anchored by bond in the concrete or by an end-anchor, a combined mode of behavior develops with both dowel and shear friction. Some common types of connections where shear forces are transferred by dowel action in dowel pins and bolts are shown in fig.
Mechanical devices for shear transfer can be steel details that are welded or bolted to steel plates, which in turn are embedded and anchored in the concrete elements.

Dowel Action

Dowel action of partly embedded steel bars is a basic mechanism in the transfer of shear force. The simplest case is when a bar embedded at one end is loaded by a shear force acting along the joint face or at some distance from the joint face, see fig. When this load case is studied by theory of elasticity as a beam on elastic foundation, the concrete stresses in a plane through the dowel pin as indicated in fig. As a result there will be high bearing stresses under the dowel pin near the joint face, and the dowel pin will be subjected to a shear force, which changes sign along the dowel pin, and a bending moment with a maximum value at some distance below the joint face.

Failure Modes

- Steel shear failure
- Concrete splitting failure
- Steel flexural failure

Steel Shear Failure

The shear capacity of a steel bar loaded in pure shear can be estimated by adopting the yield criteria, which is expressed as

\[ F_{\text{vld}} = \alpha_g \cdot f_{yd} \cdot A_s \]

Where \( \alpha_g = 0.6 \) in normal case

Concrete Splitting Failure

The load case gives rise to a highly concentrated reaction in the concrete under the dowel pin. The connection zone must be designed and detailed so that this concentrated reaction is safely spread and transferred into the element. The concentrated reaction tends to split the element, but the splitting can be controlled by reinforcement designed to establish an equilibrium system in cracked reinforced concrete.

Steel Flexural Failure

When the dowel pin is not very weak in relation to the surrounding concrete, the steel bar fails when a plastic hinge is ultimately formed in the section with the maximum bending moment. This failure mode is associated with a significant settlement of the dowel bar in the surrounding concrete that crushes under the high compressive stresses.

For concrete subjected to high bearing stresses under a local loading area, a tri-axial state of stresses is obtained. For such a case compressive stresses can reach values that are several times the uniaxial concrete compressive strength.

\[ f_{cd}^* = 4 \cdot f_{cd} \]

The bearing stress under the dowel pin can be assumed to reach a similar stress level in case when splitting failure is avoided.

The ultimate shear capacity can be solved

\[ F_{\text{vld}} = \alpha_g \cdot \Phi_s \cdot \sqrt{f_{cd} \cdot f_{yd}} \]

\( \alpha_g = 1 \) in design

Influence of Non-Symmetrical Conditions

Plain dowel pin without end anchors, different concrete strengths
When dowel action is used in practical applications, it happens quite often that the conditions are different on each side of the joint, for instance due to quite different concrete strengths. A typical case is a bolted beam-column connection where the bolt is cast in place in the supporting member and protrudes into a vertical recess in the supported member, where the recess is filled with grout. This load case is not any longer symmetrical, but the connection has a stronger and a weaker side. Accordingly, the plastic hinges will not develop simultaneously, and the ultimate load is reached at the formation of the second plastic hinge, which turns the resisting system to a collapse mechanism.

This means that for a certain shear force, a plastic hinge formed in the dowel pin at the weaker side, while the dowel still has an elastic behavior at the stronger side. Hence the load can be increased further and not until a plastic hinge is formed also at the other side, a failure mechanism is obtained. However, the stiffness of the shear connection is reduced by the formation of the first plastic hinge.

Since the ultimate capacity of the connection is determined by the formation of the second hinge, the shear capacity in case of no eccentricity can be calculated as

\[
F_{vRd} = \alpha_0 \cdot \phi_2 \cdot \sqrt{f_{\text{cd,max}} \cdot f_{\text{yd}}}
\]

Where \(f_{\text{cd,max}}\) = design concrete compressive strength at stronger side.

**Combination of Dowel Action and Friction**

Plain dowel pin with end anchors

In a shear connection where the dowel pin is plain and without end anchors, the shear displacement is possible to obtain without any significant axial restraint in the dowel pin. The dowel pin can slide inside the concrete and will successively adapt itself by bending deformations to the actual shear displacement. When the dowel is plain the bond stresses along the dowel can be assumed to be without significance. It means that only flexural stresses will appear in the critical section with the maximum bending moment.

However, for a plain dowel pin with end anchors, a substantial axial restraint develops, when the shear connection is loaded in shear. If the end anchors are firmly fixed in the concrete, the dowel pin must elongate to adapt itself to the shear deformation. It means that overall axial stresses as well as flexural stresses develop when the connection is loaded in shear. The final failure depends still on a mechanism with plastic hinges, but the dowel capacity is influenced by the axial restraint.

Since this steel stresses is used for the overall elongation, it is not any more available for the flexural resistance of the dowel bar and the dowel capacity will be reduced. On the other hand the tensile force in the dowel bar must be balanced by an equal compressive force at the interface and a frictional force develops along the interface, which contribute shear resistance

\[
F_{vR} = \alpha_0 \cdot \phi_2 \cdot \sqrt{f_{\text{cd,max}} \cdot f_{\text{y,red}}} + \mu \cdot \sigma_{sn} \cdot A_s
\]

\(f_{\text{cd,max}}\) = concrete compressive strength at the stronger side

\(f_{\text{y,red}}\) = strength available for dowel action

\(\mu\) = friction coefficient

\(\sigma_{sn}\) = normal stress

\(A_s\) = cross-sectional area of the dowel bar

**Reference**

- Fib bulletin 43- Structural Connections for Precast Concrete Buildings

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