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# Simplified design for mechanical connections between precast concrete structural elements in buildings

*Conception simplifiée pour les assemblages mécaniques entre éléments structurels en béton préfabriqué dans les bâtiments*



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# Foreword

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This document was prepared by Technical Committee ISO/TC 71, *Concrete, reinforced concrete and pre-stressed concrete*, Subcommittee SC 5, *Simplified design standard for concrete structures*.

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# Introduction

This document contains a set of practical provisions for the design of the mechanical connections in precast elements under seismic actions. Design of the connections is carried out in terms of strength verifications. Indications are also provided for defining the actions to be used in design.

If national standards provide alternate formulae for the same typology, those can be used instead of the ones given in this document.

# Simplified design for mechanical connections between precast concrete structural elements in buildings

## 1 Scope

This document refers to connections in precast frame systems, either for single-storey or multi-storey buildings. The connections for all orders of joints are considered. Large wall panel and three-dimensional cell systems are not considered.

According to the position in the overall construction and of the consequent different structural functions, the seven following orders of joints are considered:

a) *mutual joints between floor or roof elements* (floor-to-floor) that, in the seismic behaviour of the structural system, concern the diaphragm action of the floor;

b) *joints between floor or roof elements and supporting beams* (floor-to-beam) that give the peripheral constraints to the floor diaphragm in its seismic behaviour;

c) *joints between beam and column* (beam-to-column) that ensure in any direction the required degree of restraint in the frame system;

d) *joints between column segments* (column-to-column) used for multi-storey buildings usually for dual wall braced systems;

e) *joints between column and foundation* (column-to-foundation), able to ensure in any plane a fixed full support of the column;

f) *fastenings of cladding panels to the structure* (panel-to-structure) that ensure the stability of the panels under the high forces or the large drifts expected under seismic action;

g) *joints between adjacent cladding panels* (panel-to-panel) possibly used to increase the stiffness of the peripheral wall system and provide an additional source of energy dissipation.

Simple bearings working by gravity load friction are not considered. Sliding and elastic deformable supporting devices neither, being all these types of connections not suitable for the transmission of seismic actions.

The document provides formulae for the strength design of a large number of joint typologies.



## 2 Normative references

There are no normative references in this document.

## 3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at http://www.electropedia.org/

**3.1**
**union**
generic linking constraint between two or more members

**3.2**
**connection**
local region that includes the *union* (3.1) between two or more members

**3.3**
**connector**
linking device (usually metallic) interposed between the parts to be connected

**3.4**
**node**
local region of convergence between different members

**3.5**
**joint**
equipped interface between adjacent members

**3.6**
**system**
<joint> set of linking practices classified on the basis of the execution technology

**3.7**
**typical joint**
dry *joint* (3.5) with mechanical *connectors* (3.3) generally composed of angles, plates, channel bars, anchors, fasteners, bolts, dowel bars, etc., including joints completed in-situ with mortar for filling or fixing

**3.8**
**emulative joint**
wet *joint* (3.5) with rebar splices and cast-in-situ concrete restoring the monolithic continuity typical of cast-in-situ structures and leading usually to "moment-resisting" *unions* (3.1)

**3.9**
**strength**
maximum value of the force which can be transferred between the parts

**3.10**
**ductility**
ultimate plastic deformation compared to the yielding limit

Note 1 to entry: The ductility values or ranges provided refer to the *connections* (3.2) themselves and, in general, have no direct relation with the global ductility of the structure. Those values are given for the sake of classifying the connection and are not supposed to intervene in the design, which is carried out in terms of *strength* (3.10).

Note 2 to entry: Instead of the plastic deformation of steel element beyond the yield limit, other physical equivalent non-conservative phenomena can be referred to (such as friction).



**3.11**
**dissipation**
specific energy dissipated through the load cycles related to the corresponding perfect elastic-plastic cycle

Note 1 to entry: The values or ranges provided refer to the *connections* (3.2) themselves and, in general, have no direct relation with the global energy dissipation of the structure. Those values are given for the sake of classifying the connection and are not supposed to intervene in the design, which is carried out in terms of *strength* (3.10).

**3.12**
**cyclic decay**
**decay**
*strength* (3.10) loss through the load cycles compared to the force level

**3.13**
**damage**
residual deformation at unloading compared to the maximum displacement and/or evidence of rupture

# 4 Properties

A connection is composed of three parts: two lateral parts A and C, corresponding to the local regions of the adjacent members close to the connector; and a central part B constituted by the connector itself with its metallic components (see Figure 1).

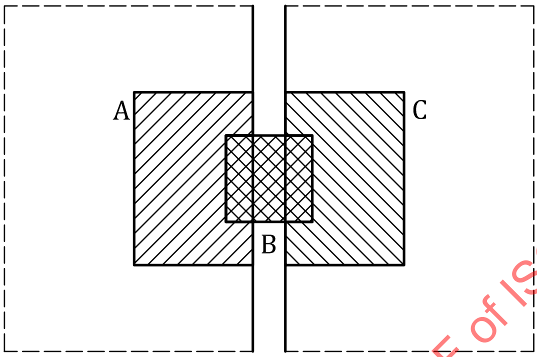



**Figure 1 — Scheme of connection**



The main parameters which characterize the seismic behaviour of the connection, as measured through monotonic and cyclic tests, refer to the six properties of:

— strength;

— ductility;

— dissipation;

— deformation;

— decay;

— damage.

A ductile dissipative behaviour of the connection can be provided by the steel connector B:

— when parts A and C have a non-ductile non-dissipative behaviour characterized by a brittle failure, with small displacements, due to the tensile cracking of concrete; and

— if it is correctly designed for a failure involving flexural or tension-compression modes and not shear modes or by other dissipative phenomena like friction.

In this case, for a ductile connection, in addition to a ductile connector, the criteria of capacity design shall be applied, under-proportioning the connector with respect to the lateral parts.

Also, the geometric compatibility of deformations shall be checked (e.g. against the loss of bearing). Non-ductile connections shall be:

— suitably over-proportioned by capacity design with respect to the resistance of the critical dissipative regions of the structure; or

— proportioned on the basis of the action obtained from a structural analysis that does not account for any energy dissipation capacity.

The ductility of the connections can contribute to the global ductility of the structure or not depending on their position in the structural assembly and on their relative stiffness.

# 5 Classification

## 5.1 General

For any single type of connection, the strength is quantified by the relevant formulae. The other behaviour properties listed in this clause are quantified by specific numerical values. When this precise numerical quantification is not possible, because of lack of experimental data or excessive variability of performances, the type of connection is classified in qualitative terms corresponding to ranges of values.

## 5.2 Strength

For strength, the following information shall be given:

— *behaviour models* corresponding to the working mechanisms of the connection;

— *failure modes* of the resistant mechanisms;

— *calculation formulae* for the evaluation of the ultimate strength for any failure mode;

— *other data* concerning the specific properties of the connection.

Reference is made to the strength obtained from cyclic loading tests.

## 5.3 Ductility

For ductility, the following classification is deduced from the force-displacement diagrams obtained in experiments:

— *brittle connections,* for which failure is reached without relevant plastic deformation;

— *over-resisting connections,* for which failure has not been reached at the functional deformation limit;

— *ductile connections,* for which a relevant plastic deformation has been measured.

In this classification, intentional friction mechanism is equal to plastic deformation. Brittle connections can be used in seismic zones provided they are:

— over-proportioned by capacity design with respect to the critical regions of the overall structure; or

— proportioned with the action deducted from a structural analysis that does not account for any energy dissipation capacity.

### 5.3.1 Ductile connections

Furthermore, ductile connections are divided into:

— *high ductility connections,* with a displacement ductility ratio of at least 4,5;

— *medium ductility connections,* with a displacement ductility ratio of at least 3,0;

— *low ductility connections,* with a displacement ductility ratio of at least 1,5.

With ductility ratio lower than 1,5, the connection is classified as *brittle*.

These definitions refer to the connection itself and, in general, have no direct relation with the global ductility of the structure. For any single order of connections, specific indications are given on this aspect, referring both to ductility and dissipation.



### 5.4 Dissipation

For dissipation, the following classification is deduced from the force-displacement diagrams of cyclic tests and related enveloped area histograms:

— *low dissipation*, with specific values of dissipated energy between 0,10 and 0,30;

— *medium dissipation*, with specific values of dissipated energy between 0,30 and 0,50;

— *high dissipation*, with specific values of dissipated energy over 0,50.

With dissipated energy lower than 0,1, the connection is classified as *not dissipative*.

The theoretical value 1,00 would correspond to the maximum energy dissipated through a perfect elastic-plastic cycle, e.g. by a massive section of ductile steel under alternate flexure, medium dissipation corresponds to well confined reinforced concrete sections under alternate flexure and high dissipation can be achieved with the use of special dissipative devices.

### 5.5 Deformation

For deformation, indications can be given about the order of magnitude of the relative displacements of the connection at certain relevant limits, such as the first yielding of steel devices, the ultimate failure limit or the maximum allowable deformation referred to its functionality.

Indications about *cyclic decay* and *damage* are given if relevant and when specific experimental information is available.

## 6 Floor-to-floor connections

### 6.1 Cast-in-situ topping

#### 6.1.1 General

Figure 2 shows the details of a floor made of precast elements interconnected by a concrete topping cast over their upper surface. The concrete topping, with its reinforcing steel mesh, provides a monolithic continuity to the floor that also involves the precast elements if properly connected to it. The diaphragm action for the in-plane transmission of the horizontal forces to the bracing vertical elements of the structure can be allotted entirely to the topping. Unless greater dimensions are defined from design, for its structural functions, the concrete topping shall have a minimum thickness, $t_{min}$, related also to the maximum aggregate size of the concrete, $d_g$, such as $t_{min} = 2{,}4\ d_g \geq 60$ mm.

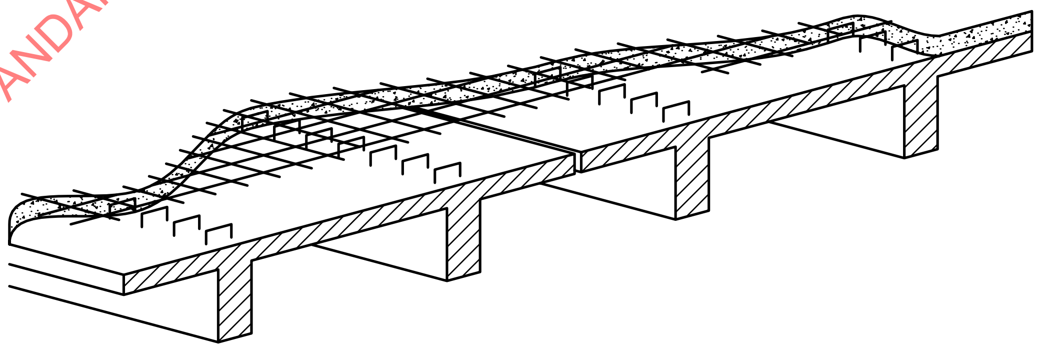


**Figure 2 — Floor made by precast elements**

### 6.1.2 Strength

Interface shear strength of the connection between the precast element and the topping under seismic action can be evaluated neglecting the friction contribution due to gravity loads. Transverse vertical shear at the joint between adjacent floor elements is diverted into the topping. For the good behaviour of the connection, proper steel links crossing the interface shall ensure, with adequate anchorages, an effective shear tie between the two parts (see Figure 3).

**Figure 3 — Precast elements with and without interface connections with topping**

### 6.1.3 Other properties

No specific parameters of seismic behaviour (ductility, dissipation, deformation, decay, damage) have been experimentally measured for this type of indirect connection provided by the cast-in-situ topping that can be calculated like an ordinary reinforced concrete element.

## 6.2 Cast-in-situ joints

### 6.2.1 General

Figure 4 shows the floor-to-floor connection made with the concrete filling of a continuous joint between adjacent elements. It is typical of some precast products like hollow-core slabs. The joint has a proper shape to ensure good interlocking with the transmission of the vertical transverse shear forces, when filled in. For the transmission of the horizontal longitudinal shear forces, the interface shear strength can be improved providing the adjoining edges with vertical indentations. With reference to the diaphragmatic action, this type of connections ensures that the floor has the same performance as a monolithic cast-in-situ floor, provided that a continuous peripheral tie is placed against the opening of the joints. For good filling, the maximum size of the aggregate of the cast-in-situ concrete shall be limited with reference to the joint width.

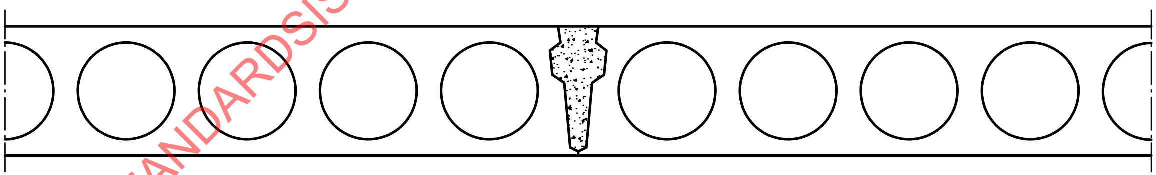


**Figure 4 — Floor-to-floor connection made by concrete filling**

Other types of floor-to-floor cast-in-situ connections, possibly provided with spliced tying steel links, are not considered in this document.

### 6.2.2 Strength

The type of connection of concern is usually intended as a continuous longitudinal hinge. Its strength is ensured following the specifications for the erection of the elements given by the manufacturer.



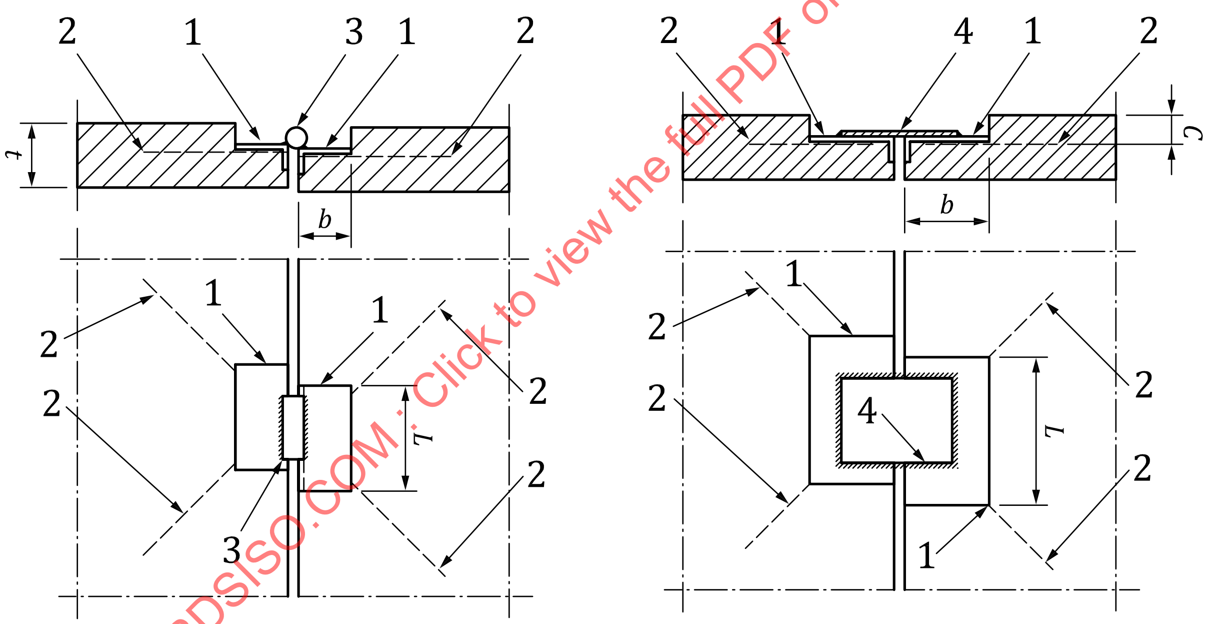
### 6.2.3 Other properties

No ductility and dissipation capacities are expected from the concerned type of connections that are located away from the critical regions of the structure.

## 6.3 Welded steel connectors

### 6.3.1 General

In Figure 5 two types of floor-to-floor welded connections are represented. The solution a) is constituted by two steel angles inserted at the edges of the adjacent elements and fixed to them with anchor loops. On the joint lap, a bar is placed welded in site to the angles, compensating the erection tolerances. Two steel plates inserted at the edges of the adjacent elements and fixed to them with anchor loops constitute the solution b). Over the joint, a middle smaller plate is placed, welded in site to the lateral ones. In both solutions, the steel components may be placed within a recess in order to save the upper plane surface of the finishing. In the first solution, the angles may be replaced with plates placed inclined so to leave in the joint the room for the positioning of the middle bar. These kinds of connections are used to join ribbed floor elements without topping. They are also used to join special roof elements when placed in contact one to the other.




a) Steel angles

b) Steel plates

**Key**

1 steel angles

2 anchor loops

3 steel bar

4 steel plates

**Figure 5 — Floor-to-floor welded connections**

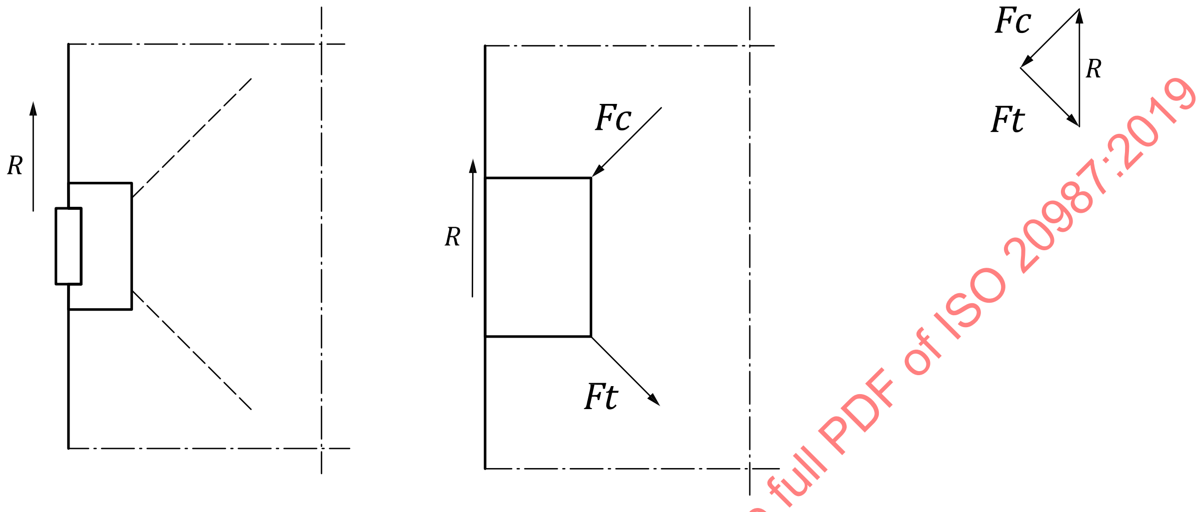
These connections are distributed in some local position along the length of the floor elements. They provide the transverse deflection consistency with the uniform distribution of the load between the elements. Under seismic conditions, they mainly provide the transmission of the diaphragm action with horizontal longitudinal shear forces.

### 6.3.2 Strength

The following indications about the mechanical behaviour of this type of connections leave out of consideration the transverse vertical shear forces that are related to the distribution of the loads between the elements and refer to a non-seismic action. Proper combinations of effects should be added to evaluate the compatibility with the seismic action.

#### 6.3.2.1 Behaviour models

With reference to the transmission of the diaphragm action under seismic conditions, the behaviour model is given in Figure 6 both for the solutions a) and b) described in 6.3.1. The longitudinal shear force, *R*, shall be mainly transmitted, with no relevant transverse normal forces.




**a) Action of the connection** **b) Action with normal forces** **c) Free body diagram**

**Key**

*Fc* normal compression force

*Ft* normal tension force

**Figure 6 — Behaviour models for welded steel connectors**

#### 6.3.2.2 Failure modes

The principal failure modes are as follows:

a) rupture of the welding between the angles and the interposed bar or plate;

b) failure of the interposed plate for solution b);

c) failure of the anchor loops for tensile yielding (see NOTE);

d) failure of the anchor loops for pull-out (see NOTE);

e) spalling of concrete edges due to tensile stresses.

NOTE It is assumed that the anchor loops are fixed to the angles with an adequate welding.

#### 6.3.2.3 Calculation formulae

In expectation of a brittle behaviour of the connection, the action *R* is evaluated through the analysis of the overall structural system with an adequately reduced behaviour factor or through a reliable model of capacity design with respect to the resistance of the critical sections of the structure, using the due overstrength factor, $\gamma_R$.

NOTE Values $\gamma R$ = 1,20 for medium ductility structures and $\gamma R$ = 1,35 for high ductility structures.



a) welding. For the verification of the welding the usual rules shall be applied.

b) plate. Formulae (1) and (2) apply.

$$R \leq R_{vR} \tag{1}$$

with

$$R_{vR} = 0{,}67\, t_p \cdot a \cdot \frac{f_{yd}}{3} \tag{2}$$

where

$t_p$ is the plate thickness;

$a$ is the plate width;

$f_{yd}$ is the design tensile yielding stress of steel.

c) anchor loop (yielding). Formulae (3) and (4) apply.

$$R \leq R_{sR} \tag{3}$$

with

$$R_{sR} = \sqrt{2}\; A_s \cdot f_{yd} \tag{4}$$

where $A_s$ is the bar section.

d) pull-out. Formulae (5) and (6) apply.

$$R \leq R_{bR} \tag{5}$$

with

$$R_{bR} = \sqrt{2}\pi \cdot \phi \cdot I_b \cdot f_{bd} \tag{6}$$

where

$\phi$ is the bar diameter;

$l_b$ is the anchorage length.

$f_{bd} = 2{,}25\, f_{ctd}$

e) spalling. Formulae (7) and (8) apply.

$$R \leq R_{cR} \tag{7}$$

with

$$R_{cR} = 2a \cdot h \cdot f_{ctd} \tag{8}$$



where $f_{ctd}$ is the design value of the concrete tensile strength and

$h = 2c \leq t$

$a = b < l$

for $t$, $b$, $l$ and $c$ see Figure 5.

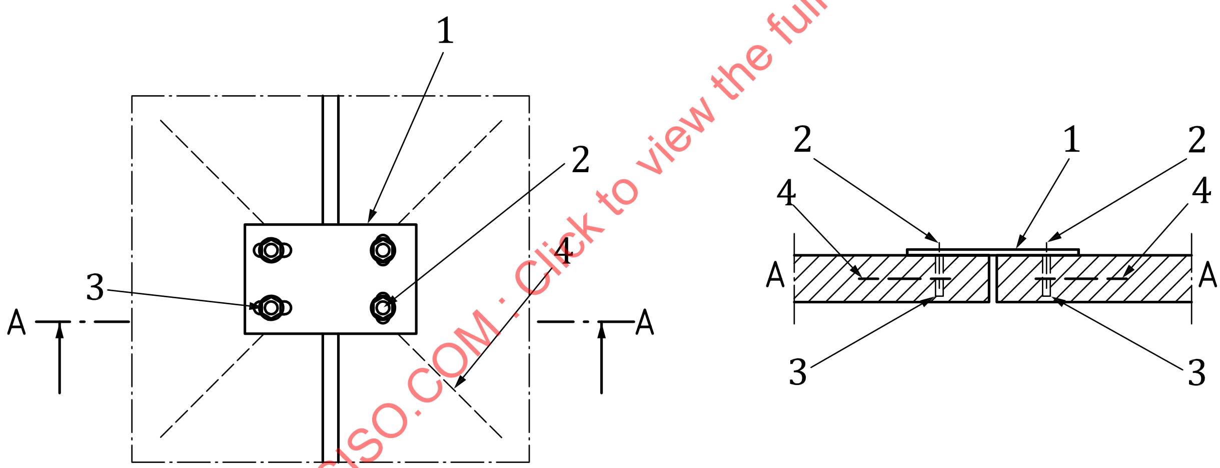
### 6.3.3 Other properties

No ductility and dissipation capacities are expected from the concerned type of connections that are located away from the critical regions of the structure.

## 6.4 Bolted steel connectors

### 6.4.1 General

Figure 7 represents a type of floor-to-floor bolted connection. Over the joint, a plate is placed, bolted in site to the bushes inserted in the lateral parts and fixed to them with anchor loops. The plate has slotted holes to compensate tolerances and may be placed within a recess in order to save the upper plane surface of the finishing. These kinds of connections are used to join ribbed floor elements without topping. They are also used to join special roof elements when placed in contact one to the other.




**Key**

1 connection plate

2 anchor bolts

3 anchor loops



**Figure 7 — Floor-to-floor bolted connection**

These connections are distributed in some local position along the length of the floor elements. They provide the transverse deflection consistency with the uniform distribution of the load between the elements and under seismic conditions. They mainly provide the transmission of the diaphragm action with horizontal longitudinal shear forces.

### 6.4.2 Strength

#### 6.4.2.1 General

The following indications about the mechanical behaviour of this type of connections leave out of consideration the transverse vertical shear forces that are related to the distribution of the loads between the elements and refer to a non-seismic action. Proper combinations of effects should be added to evaluate the compatibility with the seismic action.

#### 6.4.2.2 Behaviour models

With reference to the transmission of the diaphragm action under seismic conditions, the behaviour model is given in Figure 8. The longitudinal shear force, *R*, shall be mainly transmitted, with no relevant transverse normal forces.

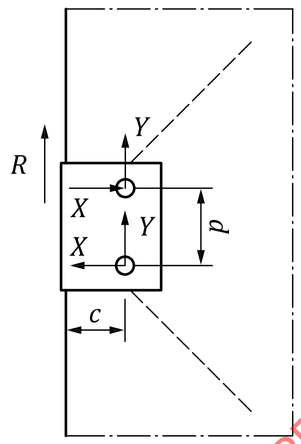



**Figure 8 — Behaviour models for bolted steel connector**

#### 6.4.2.3 Failure modes

The principal failure modes are as follows:

a) shear failure of the anchor bolt;

b) failure of the interposed plate for solution;

c) failure of the anchor loops for tensile yielding;

d) failure of the anchor loops for pull-out;

e) spalling of concrete edges due to tensile stresses.

#### 6.4.2.4 Calculation formulae

In expectation of a brittle behaviour of the connection, the action *R* is evaluated through the analysis of the overall structural system with an adequately reduced behaviour factor or through a reliable model of capacity design with respect to the resistance of the critical sections of the structure, using the due overstrength factor $\gamma_R$.

NOTE Values $\gamma R$ = 1,2 for medium ductility structures and $\gamma R$ = 1,35.

With reference to the symbols of Figure 8 the following effect arises [see Formulae (9) to (23)]:

$$F_d = \sqrt{X^2 + Y^2} \tag{9}$$

where

$$X = R \cdot \frac{c}{d}$$

$$Y = \frac{R}{2}$$

a) bolt

$$F_d \leq F_{vRd} \tag{10}$$

with

$$F_{vRd} = A_b \cdot f_{vd} \tag{11}$$

$$f_{vd} = \frac{0{,}7 \cdot f_{tk}}{\gamma_{M2}} \tag{12}$$

where

$A_b$ is the core section of the bolt;

$f_{tk}$ is the bolt characteristic tensile strength;

$\gamma_{M2}$ = 1,25.

b) plate

$$R \leq R_{vR} \tag{13}$$

with

$$R_{vR} = 0{,}67\ t_p \cdot a \cdot \frac{f_{yd}}{3} \tag{14}$$

where

$t_p$ is the plate thickness;

$a$ is the plate width;

$f_{yd}$ is the design tensile yielding stress of steel.

c) anchor loop (yielding)

$$R \leq R_{sR} \tag{15}$$

with

$$R_{sR} = \sqrt{2}\ A_s \cdot f_{yd} \tag{16}$$

where $A_s$ is the bar section.



d) pull-out

$$R \leq R_{bR} \; R \leq R_{sR} \tag{17}$$

with

$$R_{bR} = \sqrt{2\pi} \cdot \phi \cdot I_b \cdot f_{bd} \tag{18}$$

where

$\phi$ is the bar diameter

$l_b$ is the anchorage length

$f_{bd} = 2{,}25\, f_{ctd}$ and $f_{ctd}$ and is the design value of the concrete tensile strength

e) spalling (when it gives a higher resistance than $R_{sR}$ and $R_{bR}$ of items c and d, the following alternative verification applies)

$$X \leq X_{Rd} \tag{19}$$

with

$$X_{Rd} = \frac{X_{Rk}}{\gamma_c} \tag{20}$$

$$X_{Rk} = 2{,}2\, D^{\alpha} \cdot h^{\beta} \sqrt{f_{ck,cube} \cdot c^3} \tag{21}$$

$$\alpha = 0{,}1 \sqrt{\frac{h}{c}} \tag{22}$$

$$\beta = 0{,}1 \sqrt[5]{\frac{h}{c}} \tag{23}$$

where

$D$ is the bush diameter;

$h$ is the bush effective length;

$f_{ck,cube}$ is the characteristic compressive cubic strength of concrete;

$c$ is the edge distance of the bush axis;

$\gamma_c = 1{,}5$.

### 6.4.3 Other properties

No ductility and dissipation capacities are expected from the concerned type of connections that are located away from the critical regions of the structure.

# 7 Floor-to-beam connections

## 7.1 Cast-in-situ joints

### 7.1.1 General

Figure 9 shows a typical detail of a cast-in-situ connection between floor elements and a supporting beam. Proper links protrude from the upper side of the beam, overlapped to those protruding from the floor elements. Longitudinal bars are added to improve the mutual anchorage. A concrete casting encases the steel links in the joint. This type of connections ensures the transmission of forces without sensible displacements.

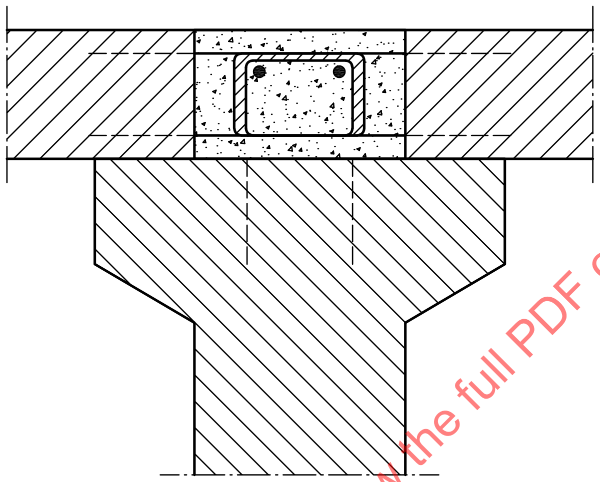


**Figure 9 — Floor-to-supporting beam cast-in-situ connection**

Interface longitudinal shear strength of the connection between the precast beam and the cast-in-situ joint under seismic action can be evaluated neglecting the friction contribution due to gravity loads. Horizontal transverse shear forces between the same parts can be attributed to the shear strength of the steel links protruding from the beam.



### 7.1.2 Other properties

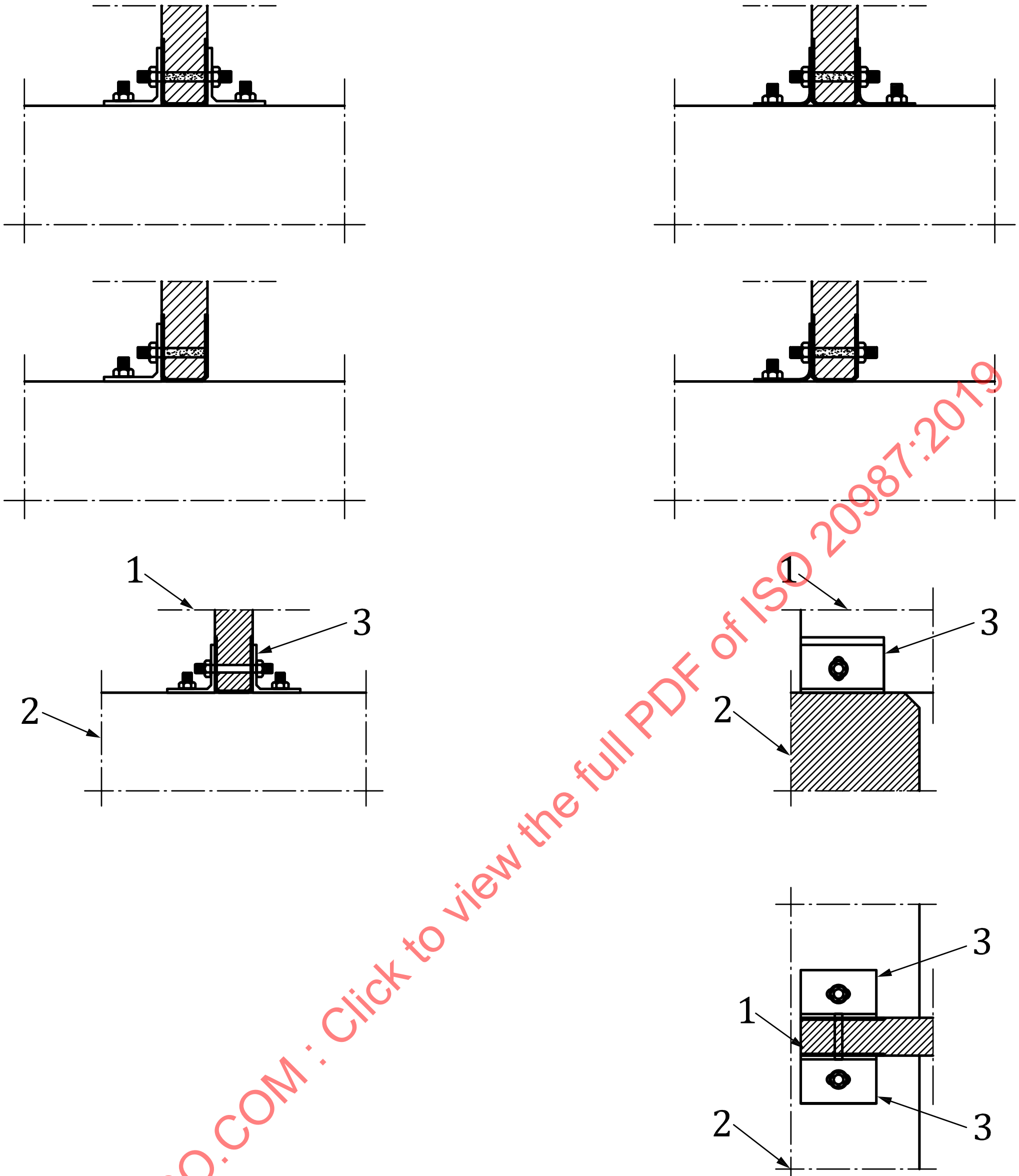
No specific parameters of seismic behaviour (ductility, dissipation, deformation, decay, damage) have been experimentally measured for this type of connection. If designed to transfer transverse moments (in addition to forces), the connection can be assumed as energy dissipating.

For the transmission of longitudinal shear, no ductility and dissipation capacities are expected from the concerned type of connections that are located away from the critical sections of the structure.

## 7.2 Supports with steel angles

### 7.2.1 General

Figure 10 shows the end connection of a rib of a floor element to a supporting beam. Steel angles are used, applied at one or both sides of the rib and fixed by means of a passing dowel to the rib and by means of anchor bolts (fasteners) to the beam. At the bottom of the rib, a U-shape steel sheet can be inserted with a passing pipe welded to it.




**Key**

1 floor element

2 supporting beam

3 steel angle



**Figure 10 — Floor-to-supporting beam steel angles connection**

The steel angles have a minimum size due to the geometry of the rib with its lower reinforcement and to the workspace for the tightening of the anchor bolts. This leads to minimum sides of about 100 mm. If commercial profiles (hot rolled angles) are used with their minimum thickness, at least an angle L100 × 10 would be chosen, which is very stiff and over-resistant with respect to the expected actions. To allow plastic deformations under cyclic loading, weakened angles can be used, cold formed from thinner steel sheets (e.g. thickness $t$ = 5 mm) with a rounded corner.

In the steel angles, the holes for dowel and bolts should be slotted in orthogonal directions in order to compensate tolerances. This requires the addition of proper knurled plates to ensure grip in the direction of the holes. In the overall model for structural analysis, spherical hinges can simulate this type of connection.

The behaviour in the horizontal transverse direction (see Figure 14) has not been tested.

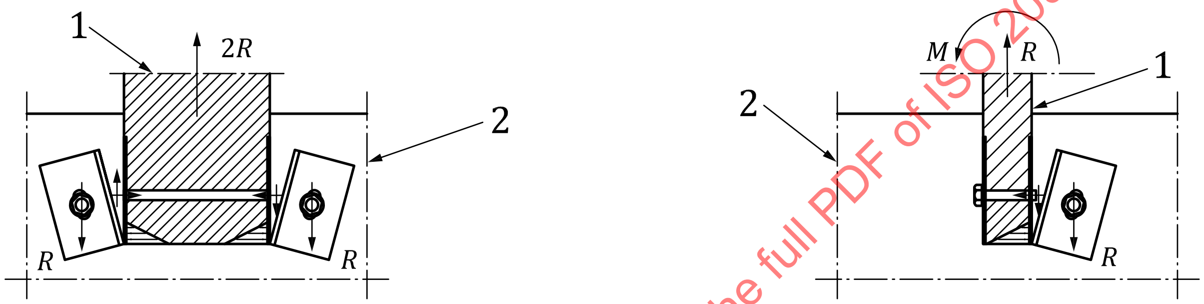
### 7.2.2 Strength

#### 7.2.2.1 General

The following indications about the mechanical behaviour of this type of connection leave out of consideration the friction that sets up between the parts due to the weight of the supported element.

In fact, in seismic conditions, under the contemporary horizontal and vertical shakes the connection shall work instantly also in absence of weight.

With this premise, it has to be pointed out that, in the longitudinal direction of the rib, the constraint given by a steel angle fixed with one bolt to the beam is hypostatic. Only after a finite small rotation, the edge of the steel angle gets in contact with the rib adding, in combination with the tensioned dowel, a rotational constraint to the steel angle for a full isostatic connection of the two parts (see Figure 11).




**Key**

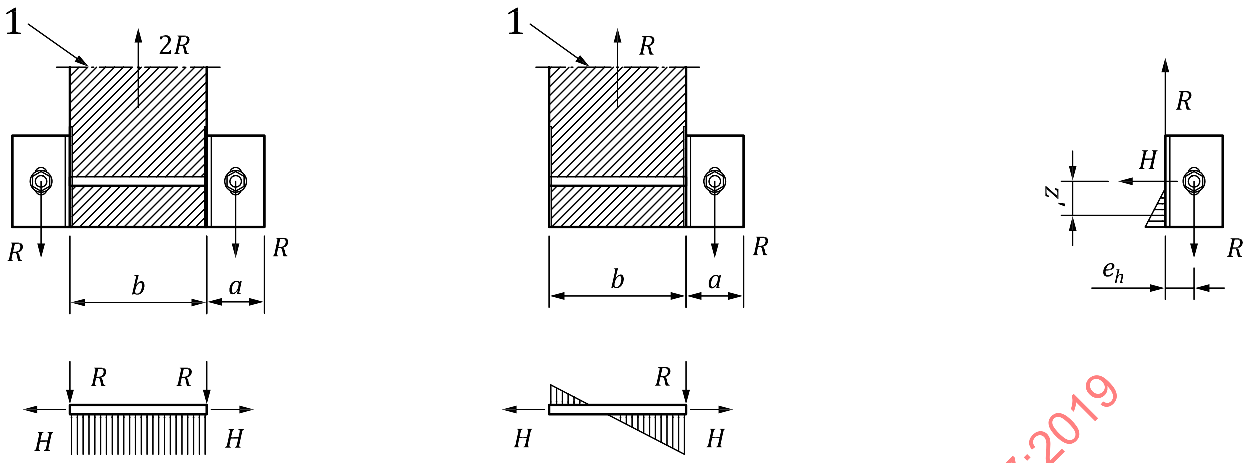
1 floor element

2 supporting beam

**Figure 11 — Kinematic models for floor-to-supporting beam steel angles connections**

#### 7.2.2.2 Behaviour models

Figure 12 shows the details in plan of the resisting mechanism for an action applied in the longitudinal direction of the rib, both for a two side and for a one-side connection. The flow of the force *R*, from the rib to the fastener fixed to the beam goes through the intervention of a couple of transverse forces, *H*, with an arm, *z*, that is related to the dimension, *l*/2, of the steel angle. For the one side connection, the eccentricity of the two forces, *R*, leads to a moment, *M*, the effects of which are compensated by the global system of the opposite connections. These effects are neglected hereafter. The main difference between the two solutions of Figure 12 is the bearing pressure of the pipe containing the dowel on the surrounding concrete: constantly distributed for the two sides connection, variable for the one side connection.

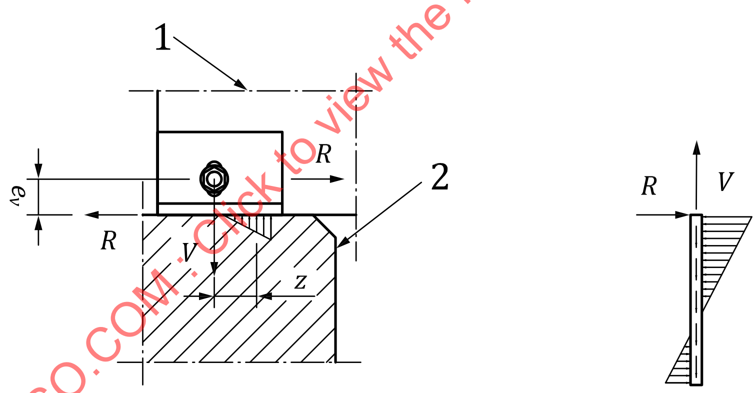



**Key**

1 floor rib

**Figure 12 — Behaviour models for floor-to-supporting beam steel angles connections**

Figure 13 shows the details in elevation of the resisting mechanism for the same longitudinal action. The eccentricity, $e_y$, of the two forces, *R*, is compensated by a couple of vertical forces, *V*, with an arm, *z*, that is related to the dimension, $l/2$, of the steel angle. This couple carries a tensile “pull-out” action to the fastener and a pressure to the concrete.




**Key**

1 floor element

2 supporting beam

**Figure 13 — Floor-to-supporting beam internal actions**

In the horizontal transverse direction, the force, *F*, is transmitted through a direct pressure between the rib and the steel angle in one direction, or through a flexure of the flange of the steel angle indirectly carried by the dowel in tension in the other sense (see Figure 14). In the two-side solution, the two mechanisms are combined together, the first one being expected to be the major because of its greater stiffness. Generally, the one side connection is placed in the opposite sides of the two ribs of a floor element and in this way; the global force is mainly carried by the steel angle in compression.


**Key**

1 floor element

2 supporting beam

**Figure 14 — Transverse behaviour of floor-to-supporting beam steel angles connection**

### 7.2.2.3 Failure modes

The principal failure modes are as follows:

a) rupture of the external section of the dowel subjected to shear and tension;

b) local plastic crushing of the steel angle around the holes due to bearing stresses;

c) breaking of the anchor bolt subjected to shear and tension;

d) spalling of the concrete edges of the rib due to tensile stresses;

e) spalling of the concrete edges of the beam due to tensile stresses.

For ordinary proportioning, the failure of the steel angle subjected to twisting action is not expected.

### 7.2.2.4 Calculation formulae

With reference to the symbols of Figure 12 and Figure 13, for the action of a given force, $R$, evaluated by capacity design with respect to the resistance of the critical sections of the structure using the due overstrength factor, $\gamma_R$[1], the following effects arise [see Formulae (24) to (57)]:

$$M = R\left(e_h + \frac{b}{2}\right) \qquad (24)$$

$$H = R \cdot \frac{e_h}{z} \qquad (25)$$

$$V = R \cdot \frac{e_v}{z} \qquad (26)$$

with

$$z \approx \frac{1}{3} \qquad (27)$$

NOTE Values $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35.



a) dowel

$$\sqrt{2}\frac{H}{H_{Rd}}+\frac{R}{R_{Rvd}}\leq 1 \tag{28}$$

and

$$H\leq H_{Rd} \tag{29}$$

with

$$R_{vRd}=A_b\cdot f_{yd} \tag{30}$$

$$H_{Rd}=0{,}9\cdot A_b\cdot f_{td} \tag{31}$$

$$f_{yd}=0{,}7\frac{f_{tk}}{\gamma_{M2}} \tag{32}$$

$$f_{td}=\frac{f_{tk}}{\gamma_{M2}} \tag{33}$$

where

$A_b$ is the core section of the bolt;

$f_{tk}$ is the bolt characteristic tensile strength; $\gamma_{M2}$ = 1,25.



b) steel angle

$$R\leq R_{bRd} \tag{34}$$

with

$$R_{bRd}=2{,}5\cdot t\cdot\varphi\cdot f_{td} \quad \text{for round holes} \tag{35}$$

or

$$R_{bRd}=1{,}5\cdot t\cdot\varphi\cdot f_{td} \quad \text{for slotted holes perpendicular to the action} \tag{36}$$

where

$t$ is the flange thickness;

$\phi$ is the bolt diameter;

$e$ is the edge distance of the bolt axis;

$f_{tk}$ is the flange characteristic tensile strength;

$\gamma_{M2}$ = 1,25.

c) anchor bolt

$$\left(\frac{R}{R_R}\right)^2 + \left(\frac{V}{V_R}\right)^2 \leq 1 \tag{37}$$

where

$R_R$ is the minimum shear resistance;

$V_R$ is the minimum tensile resistance of the anchor bolt declared by the producer.

d) rib edge

$$R \leq R_{Rd} \tag{38}$$

with

$$R_{Rd} = \frac{R_{Rk}}{\gamma_c} \tag{39}$$

$$R_{Rk} = 1{,}4 \cdot k \cdot d^{\alpha} \cdot h^{\beta} \sqrt{f_{ck,cube} \cdot c^3} \tag{40}$$

$$\alpha = 0{,}1 \sqrt{\frac{h}{c}} \tag{41}$$

$$\beta = 0{,}1 \sqrt[5]{\frac{d}{c}} \tag{42}$$

$$k = \frac{s \cdot h}{4{,}5 \cdot c^2} \leq n \tag{43}$$

$$s = 1{,}5 \cdot c \cdot e_v \leq 3{,}0\, c \tag{44}$$

$$h = \frac{b}{2} \leq 1{,}5c \quad \text{for two sides angles} \tag{45}$$

$$h = \frac{b}{3} \leq 1{,}5c \quad \text{for one side angle} \tag{46}$$

$$h \leq 8d \tag{47}$$

where

$d$ is the pipe diameter;

$f_{ck,cube}$ is the characteristic compressive cubic strength of concrete;

$c$ is the edge distance of the dowel axis;

$\gamma_c$ = 1,5.



In case supplementary edge reinforcement (links or loops) of higher resistance is present, the above verification is applied with:

$$R_{Rk} = f_{yk} \cdot A_s \tag{48}$$

$$R_{Rd} = \frac{R_{Rk}}{\gamma_s} \tag{49}$$

where

$f_{yk} \leq 600$ N/mm$^2$;

$A_s$ is the is the sectional area of the bars parallel to the forcel, $R$, included within a distance less than or equal to 0,75 $c$ from the dowel in its effective length, $h$;

$\gamma = 1{,}5$.

e) beam edge

$$R \leq R_{Rd} \tag{50}$$

with

$$R_{Rd} = \frac{R_{Rk}}{\gamma_c} \tag{51}$$

$$R_{Rk} = 1{,}6 \cdot k \cdot \varphi^{\alpha} \cdot h^{\beta} \sqrt{f_{ck,cube} \cdot c^3} \tag{52}$$

$$\alpha = 0{,}1 \sqrt{\frac{h}{c}} \tag{53}$$

$$\beta = 0{,}1 \sqrt[5]{\frac{d}{c}} \tag{54}$$

$$h \leq 8\varphi \tag{55}$$

where

$\phi$ is the fastener diameter;

$h$ is the dowel effective length;

$f_{ck,cube}$ is the characteristic compressive cubic strength of concrete;

$c$ is the edge distance of the dowel axis;

$\gamma_c = 1{,}5$.

In case supplementary edge reinforcement (links or loops) of higher resistance is present, the above verification is applied with:

$$R_{Rk} = f_{yk} \cdot A_s \tag{56}$$

$$R_{Rd} = \frac{R_{Rk}}{\gamma_s} \tag{57}$$

where

$f_{yk} \leq 600$ N/mm$^2$;

$A_s$ is the is the sectional area of the bars parallel to the force, $R$, included within a distance less than or equal to 0,75 $c$ from the dowel in its effective length, $h$;

$\gamma_s = 1{,}5$.

#### 7.2.2.5 Other properties

Failure modes d and e related to a tensile cracking of the concrete edge generally correspond to the weakest mechanisms. Their strength depends mainly on the edge distance of the dowel or bolt, the properties of the concrete and the reinforcement detailing.

In the ribs of a floor element, the ordinary longitudinal reinforcement made of bars of large diameter does not prevent concrete spalling also if these bars are well anchored by hooks bent at 135°: in fact, the bars come into effect only after the cracking of concrete, but at this point, the support can be jeopardized. In order to control the crack opening and prevent the failure by spalling, effective edge reinforcement shall be added, made of small diameter U-shape horizontal links closely distributed along the lower part of the beam. These horizontal links are particularly important for small $c/d$ ratios as they restrain the dowels after the cracking of concrete. They shall be distributed over a height, $e_v + c$, from the bottom with a spacing not greater than 50 mm and dimensioned for a design resistance equal to the expected action.

To prevent the crack opening and the failure by spalling of the beam edge in case of its local cracking at the floor rib supports, a closely spaced distribution of upper horizontal stirrups or mesh shall be provided with a spacing, $s \leq 1{,}5c \leq 100$ mm, and an included longitudinal bar of diameter $\geq 0{,}12s$ along the corner.

### 7.2.3 Ductility

In testing, the failure limit of the steel connectors has never been reached, showing an over-resisting behaviour. Moreover, the monotonic force-displacement diagram appears to be affected by different contemporary contributions (settlements, friction, elastic and plastic warping deformation and large shape modifications) that do not allow to locate a well-defined yielding limit. Consequently, ductility cannot be quantified.

It is to be noted that, during testing, the load has been applied in such a way that it prevents edge spalling of the concrete rib in tension. Because of possible early failures due to edge spalling, in the real situation on the construction, the actual behaviour of the connection can be brittle.

### 7.2.4 Dissipation

Cyclic tests show that the sum of the different contributions leads to a low dissipation capacity, sensibly higher for the “weakened” thin cold-formed angles than for the “strong” hot-rolled angles.

No ductility or dissipation is expected from this type of connections due to their position in the structural assembly and to their high stiffness in comparison to the column flexibility.



### 7.2.5 Deformation

The functional deformation limit has been set at ±24 mm, being the total longitudinal drift of about 50 mm the maximum compatible with a no support loss requirement for ordinary proportioning

### 7.2.6 Cyclic decay

Cyclic tests show that at the functional deformation limit no relevant strength decay shows after the three cycles.

### 7.2.7 Damage

At the end of the monotonic and cyclic tests taken up to the functional deformation limit, large residual deformations remain as a result of the different non-conservative effects. Plastic warping deformations of the steel angles are much more evident for the "weakened" thin cold-formed angles than for the "strong" hot-rolled angles.

## 7.3 Supports with steel shoes

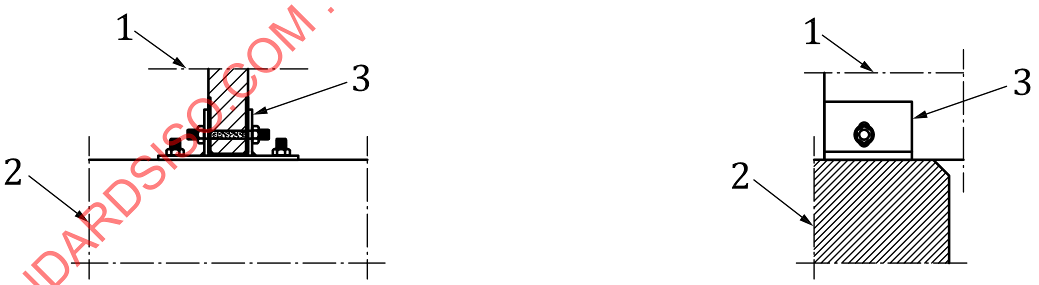
### 7.3.1 General

Figure 15 shows the end connection of a rib of a floor element to the supporting beam. A steel shoe is used, made of a lower horizontal plate with two vertical flanges welded to it. The shoe is placed under the rib, fixed to it with a passing dowel and to the beam with two anchor bolts (fasteners). At the bottom of the rib, a U-shape steel sheet can be inserted with a passing pipe welded to it.

In the steel shoe, the holes for dowel and bolts should be slotted in orthogonal directions in order to compensate tolerances. This requires the addition of proper knurled plates to ensure grip in the direction of the holes.

In the overall model for structural analysis, spherical hinges can simulate this type of connection.

The behaviour in the horizontal transverse direction (see Figure 18) has not been tested.




**Key**

1 floor element

2 supporting beam

3 welded shoe

**Figure 15 — Floor-to-supporting beam steel shoe connection**

### 7.3.2 Strength

The following indications about the mechanical behaviour of this type of connection leave out of consideration the friction that sets up between the parts due to the weight of the supported element. In fact, in seismic conditions, under the contemporary horizontal and vertical shakes, the connection shall work instantly also in absence of weight.

In the longitudinal direction of the rib, the shoe gives an isostatic constraint activated without sensible initial settlements.

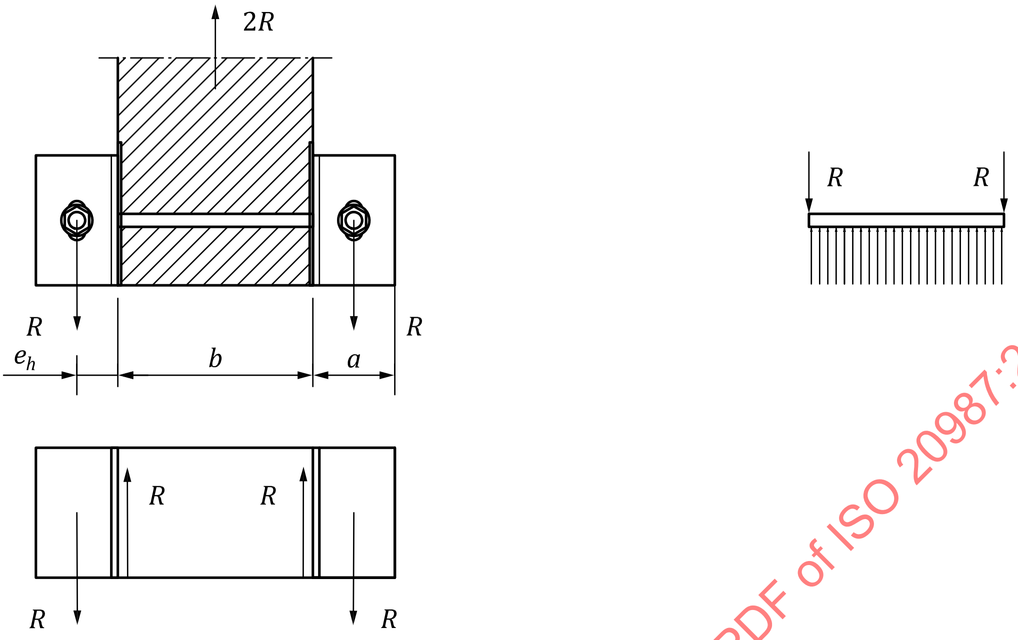



**Figure 16 — Equilibrium of floor-to-supporting beam steel shoe connection**

#### 7.3.2.1 Behaviour models

Figure 16 shows the details in plan of the resisting mechanism for an action applied in the longitudinal direction of the rib. The flow of the forces, *R*, from the rib to the fasteners fixed to the beam goes through a plane stress distribution in the lower plate of the shoe.

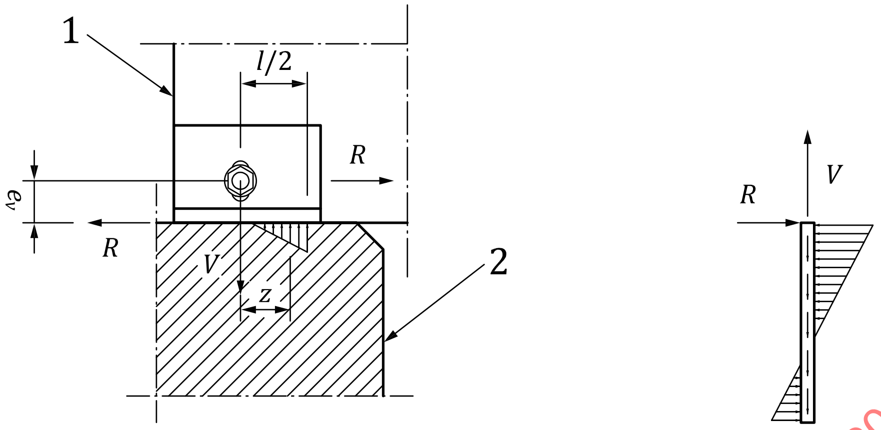
The bearing pressure of the pipe containing the dowel on the surrounding concrete is constantly distributed along the width of the rib.

Figure 17 shows the details in elevation of the resisting mechanism for the same longitudinal action.

The eccentricity, $e_y$, of the two forces, *R*, is compensated by a couple of vertical forces, *V*, with an arm, *z*, that is related to the dimension, $l$ / 2, of the steel angle. This couple carries a tensile pull-out action to the fastener and a pressure to the concrete corner.





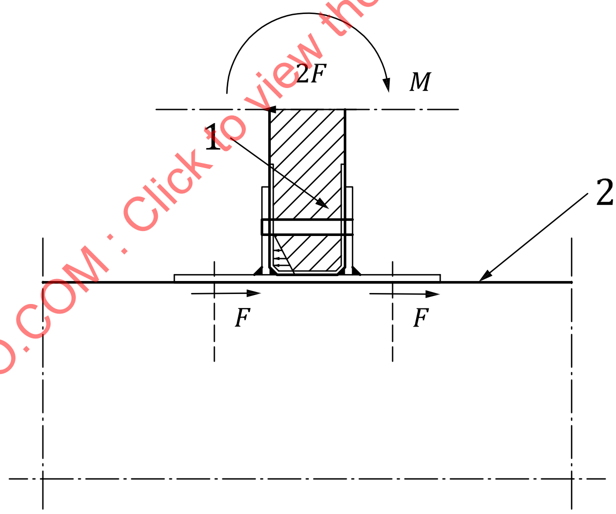

**Key**

1 floor element

2 supporting beam

**Figure 17 — Behaviour models for floor-to-supporting beam steel shoe connection**

In the horizontal transverse direction, the force, *F*, is transmitted through a direct pressure between the rib and one flange combined with a flexure of the opposite flange, indirectly carried by the dowel in tension (see Figure 18), where the first effect is expected to be the major because of its greater stiffness. At the base, the force is equally distributed on the two fasteners by the in-plane stiffness of the lower plate.




**Key**

1 floor element

2 supporting beam

**Figure 18 — Transverse behaviour of floor-to-supporting beam steel shoe connection**

#### 7.3.2.2 Failure modes

The principal failure modes are as follows:

a) rupture of the external section of the dowel subjected to shear;

b) local plastic crushing of the steel flanges or plate around the holes due to bearing stresses;

c) breaking of the anchor bolt subjected to shear and tension;

d) spalling of the concrete edges of the rib due to tensile stresses;

e) spalling of the concrete edges of the beam due to tensile stresses.

For ordinary proportioning, the failure of the steel angle subjected to twisting action is not expected.

#### 7.3.2.3 Calculation formulae

With reference to the symbols of Figure 17, for the action of a given force, $R$, evaluated by capacity design with respect to the resistance of the critical sections of the structure using the due overstrength factor, $\gamma_R$, the following effect arises [see Formulae (58) to (89)]:

$$V = R \cdot \frac{e_v}{z} \tag{58}$$

with

$$z \approx \frac{1}{3} \tag{59}$$

NOTE Values $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35.

a) dowel

$$R \leq R_{vRd} \tag{60}$$

with

$$R_{vRd} = A_b \cdot f_{yd} \tag{61}$$

$$f_{yd} = 0.7 \frac{f_{tk}}{\gamma_{M2}} \tag{62}$$

where

$A_b$ is the core section of the dowel;

$f_{tk}$ is the dowel characteristic tensile strength;

$\gamma_{M2}$ = 1,25.



b) steel shoe

$$R \leq R_{bRd} \tag{63}$$

$$\frac{1}{2} \geq 2{,}5\ \varphi \tag{64}$$

$$e \geq 2{,}0\ \varphi \tag{65}$$

with

$$R_{bRd} = 2{,}5 \cdot t \cdot \varphi \cdot f_{td} \quad \text{for round holes} \tag{66}$$

or

$$R_{bRd} = 1{,}5 \cdot t \cdot \varphi \cdot f_{td} \quad \text{for slotted holes perpendicular to the action} \tag{67}$$

$$f_{td} = \frac{f_{tk}}{\gamma_{M2}} \tag{68}$$

where

$t$ is the flange or plate thickness;

$\phi$ is the bolt diameter;

$e$ is the edge distance of the bolt axis;

$f_{tk}$ is the flange or plate characteristic tensile strength;

$\gamma_{M2} = 1{,}25$.

c) anchor bolt

$$\left(\frac{R}{R_R}\right)^2 + \left(\frac{V}{V_R}\right)^2 \leq 1 \tag{69}$$

where

$R_R$ is the minimum shear resistance;

$V_R$ is the minimum tensile resistance of the anchor bolt declared by the producer.

d) rib edge

$$R \leq R_{Rd} \tag{70}$$

with

$$R_{Rd} = \frac{R_{Rk}}{\gamma_c} \tag{71}$$

$$R_{Rk} = 1{,}4 \cdot k \cdot d^{\alpha} \cdot h^{\beta} \sqrt{f_{ck,cube} \cdot c^3} \tag{72}$$

$$\alpha = 0{,}1 \sqrt{\frac{h}{c}} \tag{73}$$

$$\beta = 0{,}1 \sqrt[5]{\frac{d}{c}} \tag{74}$$

$$k = \frac{s \cdot h}{4{,}5 \cdot c^2} \leq n \tag{75}$$

$$s = 1{,}5 \cdot c \cdot e_v \leq 3{,}0\, c \tag{76}$$

$$h = \frac{b}{2} \leq 1{,}5c \quad \text{for two sides angles} \tag{77}$$

$$h = \frac{b}{3} \leq 1{,}5c \quad \text{for one side angle} \tag{78}$$

$$h \leq 8d \tag{79}$$

where

$d$ is the pipe diameter;

$f_{ck,cube}$ is the characteristic compressive cubic strength of concrete;

$c$ is the edge distance of the dowel axis;

$\gamma_c$ = 1,5.

In case supplementary edge reinforcement (links or loops) of higher resistance is present, the above verification is applied with:

$$R_{Rk} = f_{yk} \cdot A_s \tag{80}$$

$$R_{Rd} = \frac{R_{Rk}}{\gamma_s} \tag{81}$$

where

$f_{yk}$ ≤ 600 N/mm$^2$;

$A_s$ is the is the sectional area of the bars parallel to the force, $R$, included within a distance ≤0,75 $c$ from the dowel in its effective length, $h$;

$\gamma_s$ = 1,5.

e) beam edge

$$R \leq R_{Rd} \tag{82}$$

with

$$R_{Rd} = \frac{R_{Rk}}{\gamma_c} \tag{83}$$

$$R_{Rk} = 1{,}6 \cdot k \cdot \varphi^{\alpha} \cdot h^{\beta} \sqrt{f_{ck,cube} \cdot c^3} \tag{84}$$

$$\alpha = 0{,}1 \sqrt{\frac{h}{c}} \tag{85}$$



$$\beta = 0{,}1 \sqrt[5]{\frac{d}{c}} \tag{86}$$

$$h \leq 8\varphi \tag{87}$$

where

$\phi$ is the fastner diameter;

$h$ is the dowel effective length;

$f_{ck,cube}$ is the characteristic compressive cubic strength of concrete;

$c$ is the edge distance of the dowel axis;

$\gamma_c = 1{,}5$.

In case supplementary edge reinforcement (links or loops) of higher resistance is present, the above verification is applied with:

$$R_{Rk} = f_{yk} \cdot A_s \tag{88}$$

$$R_{Rd} = \frac{R_{Rk}}{\gamma_s} \tag{89}$$

where

$f_{yk} \leq 600$ N/mm$^2$;

$A_s$ is the is the sectional area of the bars parallel to the force, $R$, included within a distance ≤0,75 $c$ from the dowel in its effective length, $h$;

$\gamma_s = 1{,}5$.

#### 7.3.2.4 Other properties

Failure modes d and e related to a tensile cracking of the concrete edge generally correspond to the weakest mechanisms. Their strength depends mainly from the edge distance of the dowel or bolt, from the properties of the concrete and from the reinforcement detailing.

In the ribs of a floor element, the ordinary longitudinal reinforcement made of bars of large diameter does not prevent concrete spalling, also if these bars are well anchored by hooks bent at 135°. In fact, the bars come into effect only after the cracking of concrete, but at this point, the support can be jeopardized. In order to control the crack opening and prevent the failure by spalling, effective edge reinforcement shall be added, made of small diameter U-shape horizontal links closely distributed along the lower part of the beam. These horizontal links are particularly important for small $c/d$ ratios as they restrain the dowels after the cracking of concrete. They shall be distributed over a height, $e_v + c$, from the bottom with a spacing not greater than 50 mm and dimensioned for a design resistance equal to the expected action.

To prevent the crack from opening, and to avoid failure because of spalling of the beam edge in case of local cracks at the floor rib supports, closely distributed upper horizontal stirrups or mesh shall be provided with a spacing $s \leq 1.5c \leq 100$ mm and an included longitudinal bar of diameter more than or equal to 0,12$s$ along the corner.

### 7.3.3 Ductility

In testing, the failure limit of the steel connectors has never been reached, showing an over-resisting behaviour. Moreover, the monotonic force-displacement diagram does not show a well-defined yielding limit. As a consequence, ductility cannot be quantified.

It is to be noted that, during testing, the load has been applied in such a way that it prevents edge spalling of the concrete rib in tension. Because of possible early failures due to edge spalling, in the real situation on the construction, the actual behaviour of the connection can be brittle.

### 7.3.4 Dissipation

Cyclic tests show a low dissipation capacity. Regardless, no ductility and dissipation is expected from this type of connections due to their position in the structural assembly and to their high stiffness in comparison to the column flexibility.

### 7.3.5 Deformation

The functional deformation limit has been set at ±24 mm, being the total longitudinal drift of about 50 mm the maximum compatible with a no support loss requirement for ordinary proportioning.

### 7.3.6 Cyclic decay

Cyclic tests show that, at the functional deformation limit, no relevant strength decay shows after the three cycles.

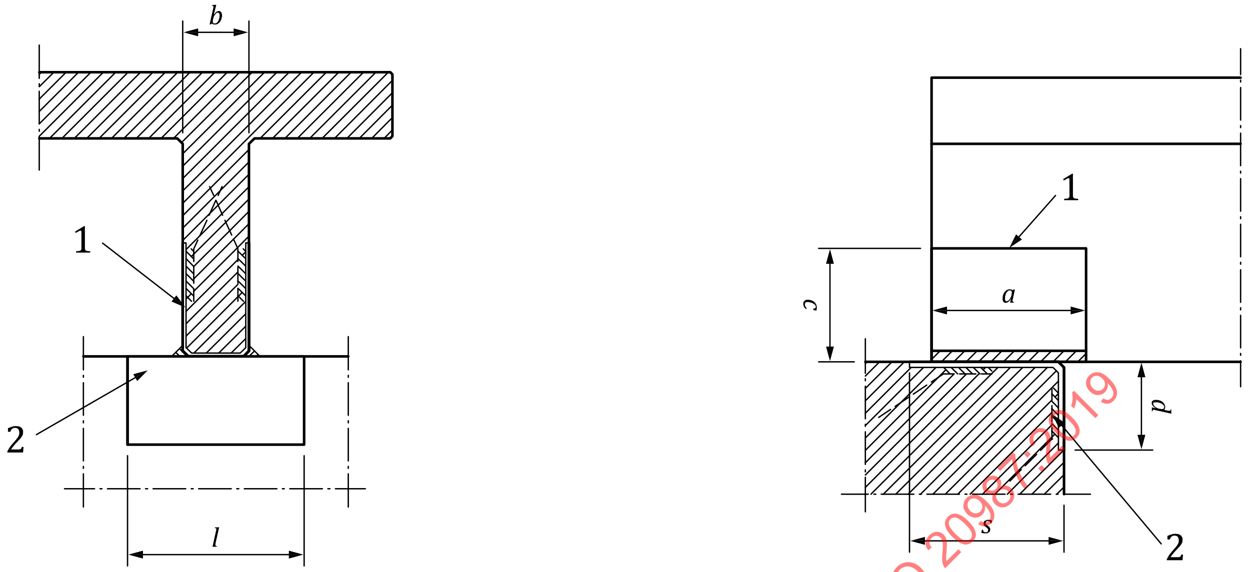
### 7.3.7 Damage

At the end of the monotonic and cyclic test taken up to the functional deformation limit, large residual deformations remain because of different non-conservative effects. Residual plastic warping deformations of the steel shoe are evident.

## 7.4 Welded supports

### 7.4.1 General

Figure 19 shows the welded connection of the rib of a floor element to a supporting beam. A U-shape steel sheet is inserted at the bottom of the rib, anchored to it with proper fasteners. An L-shape steel sheet is inserted at the edge of the beam, anchored to it with proper fasteners. Welding is made on site to connect the two parts. The number (one or two) of the welding is determined by the possibility for the welder to access from the sides of the floor element's rib.




**Key**

1 U-shaped steel plate

2 L-shaped steel plate

**Figure 19 — Welded supports**



The thickness of the steel sheets shall be proportioned with reference to the throat thickness of the welding. Adequate anchor loops shall ensure their full anchorage to the concrete parts.

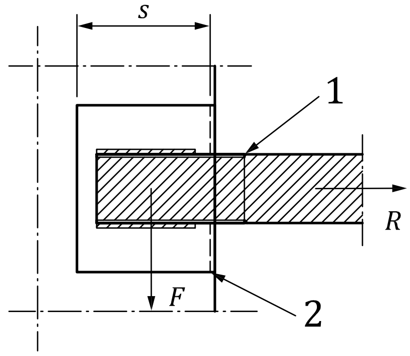
### 7.4.2 Strength

#### 7.4.2.1 General

The following indications about the mechanical behaviour of this type of connection leave out of consideration the friction that sets up between the parts due to the weight of the supported element. In fact, in seismic conditions, under contemporary horizontal and vertical shakes, the connection shall work instantly also in the absence of weight.

#### 7.4.2.2 Behaviour models

Figure 20 shows the details in plan of the connection with an indication of the two components of the horizontal action expected in seismic conditions, *R* and *F*. In structural analysis, the connection is assumed to be a spherical hinge. Actually, unintended small moments can be transmitted. They can be neglected in the design of the connected elements (beam and floor). They have effects on the connection itself, additional to those considered in 7.4.2.3, that can be taken over by its ductility resources.




**Key**

1 U-shaped steel plate

2 L-shaped steel plate

**Figure 20 — Plan of a welded supports**

### 7.4.2.3 Failure modes

The principal failure modes are as follows:

a) rupture of welding;

b) failure of the fastenings anchored in the rib of the floor element;

c) failure of the fastenings anchored in the beam;

d) spalling of the concrete edges of the rib due to tensile stresses;

e) spalling of the concrete edges of the beam due to tensile stresses.

### 7.4.2.4 Calculation formulae

With reference to the symbols of Figure 19, for the action of a given force, $R$, evaluated by capacity design with respect to the resistance of the critical sections of the structure using the due overstrength factor, $\gamma_R$, the following effect arises [see Formulae (90) to (97)]:

$$V = R \cdot \frac{e_v}{z} \tag{90}$$

with

$$z \approx \frac{1}{3} \tag{91}$$

NOTE Values $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35.

a) Welding

For the verification of the welding, the usual rules shall be applied.

b) fastenings anchored in the rib of the floor element

The anchoring system shall be properly designed referring to the specific arrangement of the fasteners.

c) fastenings anchored in the beam

The anchoring system shall be properly designed referring to the specific arrangement of the fasteners.

d) rib edge

$$R \leq R_{rR} \tag{92}$$

with

$$R_{rR} = 0{,}25 b \cdot h \cdot f_{ctd} \tag{93}$$

$$h = a + c \leq 2a \tag{94}$$

for $t$, $b$, $l$ and $c$, see Figure 19.

e) beam edge

$$R \leq R_{bR} \tag{95}$$

with

$$R_{bR} = 0{,}25 t \cdot h \cdot f_{ctd} \tag{96}$$

$$h = l + s \leq 2s \tag{97}$$

for $l$, $s$, $d$ and $t$, see Figure 19.

The resistance of the steel sheets is assumed to be verified if their thickness is not less than the throat thickness of the welding.

#### 7.4.2.5 Other properties

Failure modes d and e, related to tensile cracking of the concrete edge, generally correspond to the weakest mechanisms. Their strength depends mainly from the edge distance of the dowel or bolt, from the properties of the concrete and from the reinforcement detailing.

In the ribs of a floor element, the ordinary longitudinal reinforcement made of bars of large diameter does not prevent concrete spalling also if these bars are well anchored by hooks bent at 135°. In fact, the bars come into effect only after the cracking of concrete, but at this point the support is jeopardized. In order to control the crack opening and prevent spalling, an effective edge reinforcement can be added made of small diameter U-shape horizontal links closely distributed along the lower part of the rib edge.

The use of steel fibre reinforced concrete at the end of the rib can be equally as effective on controlling the crack opening.

The presence of pre-tensioned adherent wires or strands can contribute to improving the local behaviour reducing tensile stresses in the concrete.

### 7.4.3 Other properties

No specific parameters of seismic behaviour (ductility, dissipation, deformation, decay, damage) have been experimentally measured for this type of connection, for which no ductility and dissipation capacities are expected.



## 7.5 Hybrid connections

### 7.5.1 General

Figure 21 shows the end connections of floor ribbed elements to a supporting beam. The term “hybrid” refers to the connection arrangement made at the upper part with additional bars and cast-in-situ concrete proper of an emulative joint and at the lower part with mechanical steel devices proper of a typical joint. The upper cast-in-situ slab is connected to the precast elements by the protruding stirrups that resist the longitudinal shear transmitted through the interface. The lower connection can be made with one of the solutions described in 7.2, 7.3 and 7.4. In this subclause, the solution of welded support described in Figure 19 is referred to.

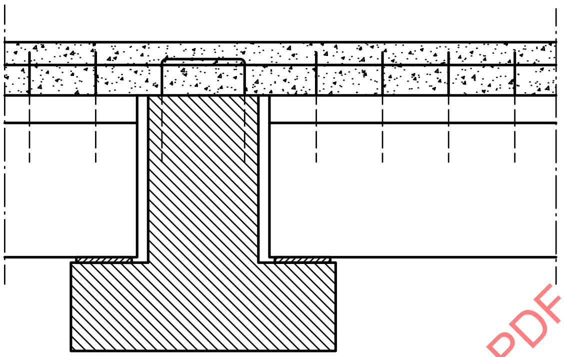


**Figure 21 — Hybrid connection: floor ribbed elements to a supporting beam**



### 7.5.2 Strength

This type of connection, after the hardening of the cast-in-situ concrete topping, provides a moment resisting support between the parts, with an asymmetrical behaviour for positive and negative moments. The loads applied after the hardening of the topping take their action on this moment resisting connection. The self-weight of the floor elements, included the concrete topping, acts on a simple hinged support arrangement.

#### 7.5.2.1 Behaviour models

In both stages of hinged and fixed support, the shear force coming from the floor element is assumed to go entirely on the flange standing out from the beam web.

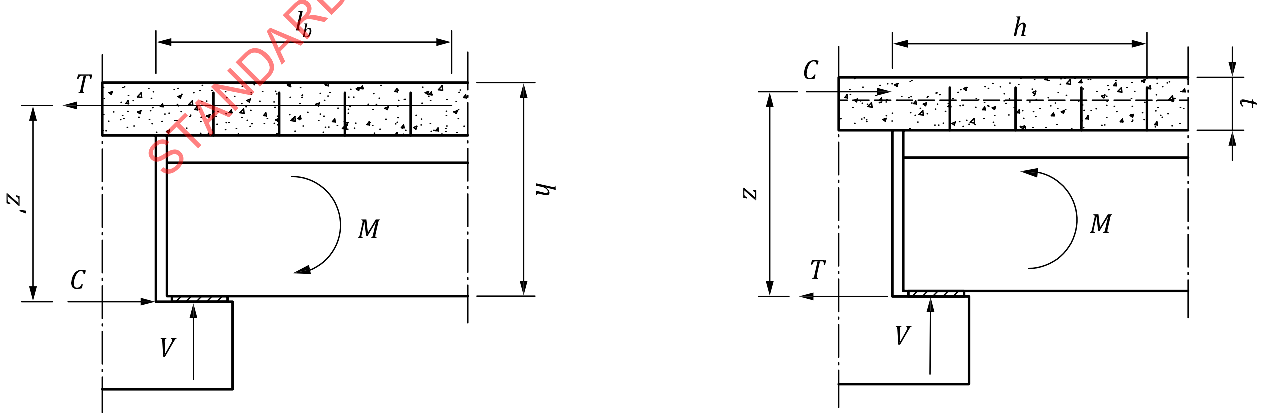



**Figure 22 — Resisting mechanisms for hybrid connections**

Figure 22 shows the resisting mechanisms respectively for negative and positive moments. In the first mechanism, the tensile force, *T*, acts in the longitudinal bars added in the cast-in-situ topping and the compressive force, $C$ (=$T$), comes from the bottom weldings with a lever arm, $z'$. In the second mechanism, the compressive force, $C$, acts in the cast-in-situ topping and the tensile force, $T$ (=$C$), comes from the bottom weldings with a lever arm, $z$.

#### 7.5.2.2 Failure modes

For a negative moment, the principal failure modes are as follows.

a) flexural failure of the connection referred to the yielding of the longitudinal upper bars;

b) bond failure of the anchorage of the upper bars;

c) longitudinal shear failure at the interface between precast element and cast-in-situ slab;

d) failure of the bottom connection between the rib and the supporting flange.

For a positive moment, the principal failure modes are:

e) flexural failure of the connection referred to the rupture of the bottom connection;

f) longitudinal shear failure of the interface between precast element and cast-in-situ slab.

#### 7.5.2.3 Calculation formulae

For a negative moment (see Figure 22), the ultimate resisting moment, $M_{Rd}$, can be calculated by:

$$M_{Rd} = A_{st} \cdot f_{yd} \cdot z' \tag{98}$$

where

$A_{st}$ is the total sectional area of the longitudinal upper bars;

$f_{yd}$ is the design tensile yielding stress of steel;

$z'$ is the lever arm.

a) flexure

$$M_{Rd} \geq M_{Ed} \tag{99}$$

where

$M_{Ed}$ is the design value coming from the structural analysis and for seismic action condition could be calculated by capacity design with a proper overstrength factor $\gamma_R$;

$t_p$ is the plate thickness;

$A$ is the plate width;

$f_{yd}$ is the design tensile yielding stress of steel.

NOTE Values $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.

b) bar anchorage

$$l_b \cdot u \cdot f_{bd} \geq \gamma_R \cdot A_s \cdot f_{ym} \tag{100}$$

with

$$f_{bd} = 2.25 f_{ctd} \quad \text{is the ultimate bond strength} \tag{101}$$

$$f_{ym} = 1.08 f_{yk} \quad \text{is the mean yielding stress of the steel} \tag{102}$$

where

$l_b$ is the anchorage length of a bar in the upper slab;

$u$ is the perimeter of a bar in the upper slab;

$A_s$ is the bars area in the upper slab;

$f_{ctd}$ is the design tensile strength of the cast-in-situ concrete;

$f_{yk}$ is the characteristic yield strength of steel.

c) longitudinal shear

$$A_{ss} \cdot f_{yd} \geq \gamma_R \cdot A_{st} \cdot f_{ym} \tag{103}$$

where

$A_{SS}$ is the total area of protruding stirrups available in the end segment long h of the beam;

$f_{yd}$ is the design tensile yielding stress of steel;

Values $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.

d) bottom connection

Verifications *a*, *b* and *c* of point 7.4.2.3 shall be applied referring to an acting force *R*.

$$R = \gamma_R \cdot A_{st} \cdot f_{ym} \tag{104}$$

For a positive moment (see Figure 22) the resisting value can be calculated by

$$M_{Rd} = R_R \cdot z \tag{105}$$

where

$R_R$ is the minimum resistance of the bottom connection calculated from all the failure modes covered by 7.4.2.3 and $z \approx h\text{-}t/2$;

$z$ is the lever arm.

e) flexure

$$M_{Rd} \geq M_{Ed} \tag{106}$$

where $M_{Ed}$ is the design value coming from the structural analysis and for seismic action condition could be calculated by capacity design with a proper overstrength factor $\gamma_R$.

f) longitudinal shear

$$A_{ss} \cdot f_{yd} \geq \gamma_R \cdot R_R \tag{107}$$



#### 7.5.2.4 Other properties

The overstrength factor $\gamma_R$ of the formulae given in 7.5.2.3 shall be properly quantified evaluating the role of the connection behaviour on the seismic response of the structure. If no relevant role is played by the connection of concern, $\gamma_R$ = 1,0 can be assumed. Otherwise, the values $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures shall be assumed.

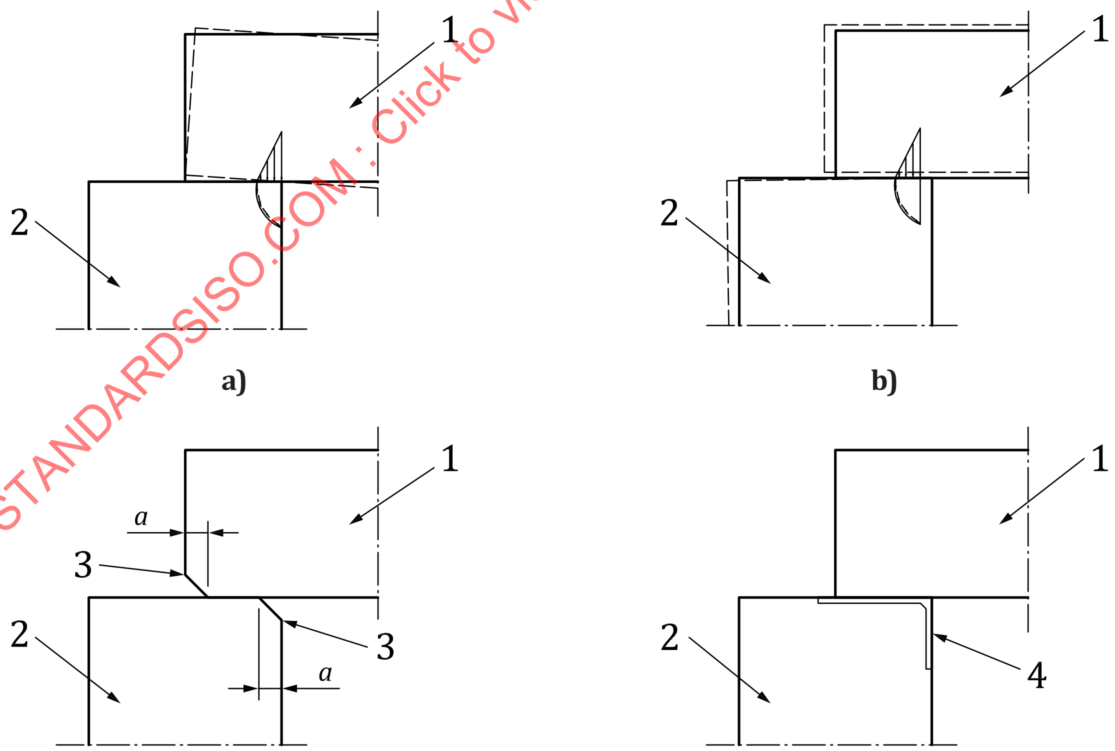
### 7.5.3 Other properties

No specific parameters of seismic behaviour (ductility, dissipation, deformation, decay, damage) have been experimentally measured for this type of connection, for which no ductility and dissipation capacities are normally expected. A general indication can be given about the flexural failure modes a and e, the first one related to negative moments being expected to be ductile, the second one related to positive moments being expected to be brittle.

## 8 Beam-to-column connections

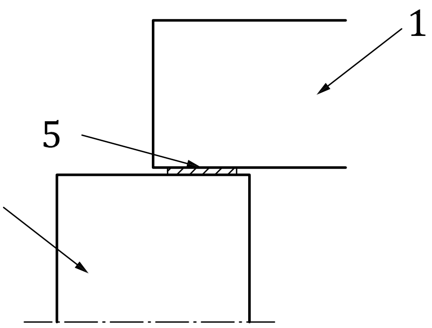
In order to protect the concrete edges of column and beam against spalling, due to the concentration of stresses under the flexural deformation of beam and column [see Figure 23 a) and b)], proper provisions shall be adopted. These provisions shall prevent the application of strong pressures on a strip of the bearing area close to the corner. The width, *a*, of this strip should correspond to the concrete cover to the confining reinforcement and indicatively should be not lesser than 20 mm.

Figure 23 c) shows a first possible solution with a chamfered edge. Figure 23 d) shows a second possible solution with the edge protected by a cold formed steel angle properly anchored to the column. Figure 23 e) shows a third possible solution with an interposed deformable rubber pad. Figure 23 f) finally shows a fourth possible solution with an interposed rigid steel plate.

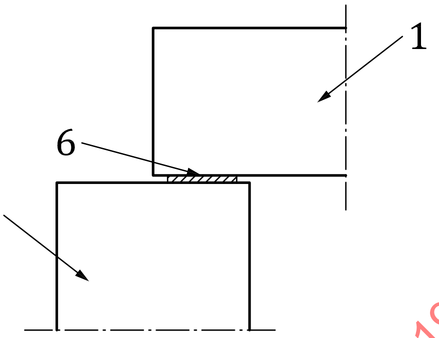


c)

d)




e)




f)

**Key**

1 beam
2 column
3 chamferred edge
4 cold-formed steel angle
5 deformable rubber pad
6 rigid steel plate

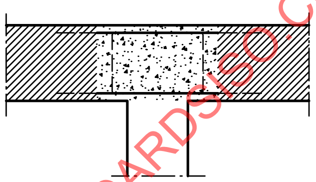
**Figure 23 — Beam-to-column connections**



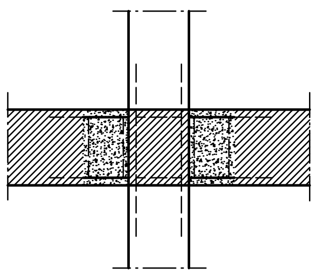
## 8.1 Cast-in-situ joints

### 8.1.1 General

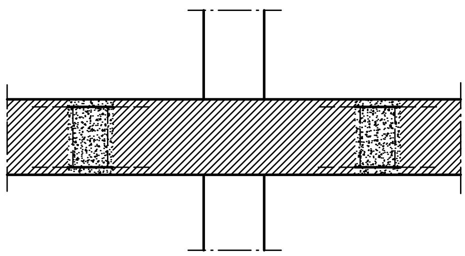
Figure 24 shows typical cast-in-situ connections between beams and columns placed in different positions. In case a), the connection is placed on the top of the column, from which the longitudinal bars protrude into the joint and overlap with those protruding from the beams. A concrete casting encases the overlapped bars in the joint. The size of the joint shall provide the room necessary for the required overlapping lengths. This type of connection ensures the transmission of forces and moments among the elements without large displacements. It falls within the possible critical regions of the resisting frame under seismic actions.



a)



b)



c)

**Figure 24 — Typical Beam-to-columns cast-in-situ connections**

In case b), the connection is placed at an intermediate storey and is divided into two separate parts, one at each side of the column. Proper bars protrude from the column into the lateral joints and overlap for the necessary length with those protruding from the beams. This way, the continuity of the column with its reinforcing bars is saved. Connections of this type ensure the transmission of forces and moments among the elements without large displacements. They fall within the possible critical regions of the resisting frame under seismic actions.

In order to move the connections out of the possible critical regions of the beams, solution c) may be adopted. In all three cases described above, proper temporary props shall be provided to the beams in the transient situations of the execution stages.

The connection can be moved into the size of the column as shown in Figure 24 a). Within the depth of the floor in transient situation, only the passing longitudinal bars give the continuity of the column.


**Key**

1 beam

**Figure 25 — Beam-to-columns cast-in-situ connections**

The necessity of temporary props during erection can be avoided if the continuity bars are moved to an inner position so as to leave room for the sitting of the precast beams, as shown in Figure 24 b). If continuity bars of the same diameter of the current longitudinal ones of the column are used in the joint, this solution weakens locally the flexural capacity of the column with detrimental effects on the frame behaviour of the structure under seismic action. Like the use of superimposed segments of column jointed at the floors levels, the latter solution can be used in structures braced by walls or cores (wall systems) where the columns are mainly subjected to axial action without relevant bending moments. To save the uniform continuity of the flexural resistance of the column through the joint in frame systems, continuity bars of a bigger diameter can be used.

With respect to Figure 24 and Figure 25, proper stirrups shall be added within the joints. The detailing of the joint may be differently laid out depending on the type of connected elements and on the arrangement of the connection.

### 8.1.2 Strength

#### 8.1.2.1 General

Strength verifications of the connections of concern refer mainly to the adequate anchorage of the overlapped bars within the joints. To avoid a brittle bond failure, it is necessary to over-proportion the anchorage length by capacity design with respect to the full tensile strength of the overlapped bars. Shear verification of the beam end shall be made following the ordinary calculation model.

The following sub-clauses give more detailed rules for a tested arrangement, which is described in Figure 25. In particular, the tested prototypes had only one beam laterally connected to a passing column.

#### 8.1.2.2 Behaviour models

This type of connection provides a monolithic union of the beam on the joint, ensuring a full support with the transmission of internal forces and moments. The usual models for the verification of shear and bending moment of cast-in-situ structural elements apply.

#### 8.1.2.3 Failure modes

The principal failure modes are as follows.

a) flexural failure of the connection referred to the yielding of the longitudinal tensioned bars;

b) bond failure of the tensioned bars;

c) longitudinal shear failure at the interface between the precast beam and the cast-in-situ slab.

The shear strength of the beam shall be over-proportioned by capacity design with respect to the flexural strength of its end sections.

#### 8.1.2.4 Calculation formulae

The ultimate resisting moment (positive or negative) can be calculated by:

$$M_{Rd} = A_{st} \cdot f_{yd} \cdot z \tag{108}$$

with

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{109}$$

$$z = d - \frac{x}{2} \tag{110}$$

$$x = \frac{A_{st} \cdot f_{yd}}{b \cdot f_{cd}} \tag{111}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \tag{112}$$

where

$A_{st}$ is the sectional area of the tensioned reinforcement;

$f_{yk}$ is the characteristic yielding stress of the tensioned reinforcement;

$d$ is the effective depth of the beam section;

$b$ is the width of beam compressed chord;

$f_{ck}$ is the concrete characteristic compressive strength;

$\gamma_s = 1{,}15$;

$\gamma_c = 1{,}5$.

a) flexure

$$M_{Rd} \geq M_{Ed} \tag{113}$$

where $M_{Ed}$ is the design value coming from the structural analysis. For the design of the frame resisting system under seismic action, the resisting moment $M_{Rd}$ could enter in the capacity design calculation together with the competent resisting moments of the other members convergent in the node.

b) bar anchorage

$$l_b \cdot u \cdot f_{bd} \geq \gamma_R \cdot A_s \cdot f_{ym} \tag{114}$$

with

$$f_{bd} = 2{,}25\, f_{ctd} \quad \text{is the ultimate bond strength} \tag{115}$$

$$f_{ym} = 1{,}08\, f_{yk} \quad \text{is the mean yielding stress of the steel} \tag{116}$$

where

$l_b$ is the anchorage length of a bar in the upper slab;

$u$ is the perimeter of a bar in the upper slab;

$A_s$ is the bars area in the upper slab;

$f_{ctd}$ is the design tensile strength of the cast-in-situ concrete;

$f_{yk}$ is the characteristic yield strength of steel.

c) longitudinal shear

$$A_{ss} \cdot f_{yd} \geq \gamma_R \cdot A_{st} \cdot f_{ym} \tag{117}$$

where

$A_{SS}$ total area of protruding stirrups available in the end segment long h of the beam;

$f_{yd}$ is the design tensile yielding stress of steel;

$\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.



#### 8.1.2.5 Other properties

For the arrangements described in Figure 24, due to the uncertain stressing of the bars within the overlapping length, it is difficult to evaluate a precise yielding limit moment of the sections in the joint. For this reason, it is preferable to over-design the connections.

### 8.1.3 Ductility

For the arrangement of Figure 25, in testing a displacement ductility over 4,0 has been always measured. This refers to the testing arrangement that includes a relevant part of the beam so that the measurements refer mainly to the flexural contribution of the beam.

In general, it can be assumed that this type of connection, if properly designed following the rules given above, saves the full capacities of the beam (between medium and high ductility).

### 8.1.4 Dissipation

Cyclic tests performed on the arrangement of Figure 25 show a medium dissipation capacity that is to be attributed to the beam.

### 8.1.5 Deformation

In cyclic tests, drifts of about 1,5 % have been reached for positive moments (upper slab in compression), of about 2,0 % for negative moments (upper slab in tension).

### 8.1.6 Cyclic decay

Limited strength decay has been measured after the three cycles of any amplitude before failure.

### 8.1.7 Damage

For drifts larger than 1 %, relative rotations have been observed between the beam and the column. Plastic flexural deformations occurred in the beam for higher drifts with the yielding of the longitudinal tensioned bars. Shear cracks penetration into the joint has also been observed.

## 8.2 Dowel connections

### 8.2.1 General

Figure 26 shows the end connection of a beam to a supporting column. In case a), two dowels protrude from the top of the column and enter into the sleeves inserted in the beam. The sleeves are filled with no-shrinkage mortar of adequate strength to ensure by bond the anchorage of the dowels. The anchorage can also be ensured providing the dowels with a cap fixed at the top with a screwed nut. In any case, the sleeve shall be filled in with mortar to avoid hammering under earthquake conditions. Case b) refers to the same technology but with only one dowel. In the transverse direction, the use of two dowels improves the resistance against overturning moments. Due the much lower stability against overturning moments, the use of one only dowel is not recommended especially with reference to the uneven load conditions during the construction stages.

The beam usually is placed over a pad to distribute the load (see Figure 23). If deformable rubber pads are used, due to their much lower stiffness, all the loads applied after their bond anchorage are conveyed into the steel dowels. In addition, this causes a local splitting damage of the concrete around the dowels.

The use of rigid steel pads prevents this effect. To avoid local splitting damage, rubber pads can be used with non-adherent dowels, but this requires a different device to transfer horizontal seismic actions without hammering. The rules given in the following subclauses are based on tests made only on connections with flexible rubber pads and adherent dowels.


a) b)



**Key**

| | | | |
|---|---|---|---|
| 1 | beam | 3 | mortar infill |
| 2 | column | 4 | dowel bar |

**Figure 26 — Beam-to-column connections**

As a general rule, a proper confinement shall be provided at the column top with additional stirrups and steel links. At the beam edge horizontal anchored hooks in front of the dowels shall be provided in order to restrain them in case of spalling of the concrete cover.

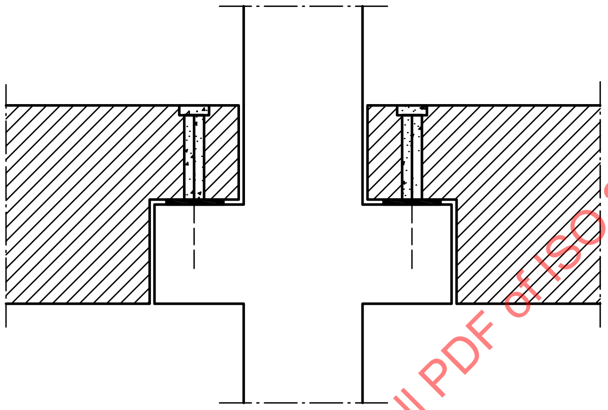


**Figure 27 — Beam-to-column connections with supporting corbels**

A similar behaviour with the same design criteria has the beam-to-column connections placed at the intermediate floors over corbels standing out from the column. In particular, Figure 27 shows one of these connections with the solution of half joints that keep the corbel within the depth of the supported beam.

### 8.2.2 Strength

#### 8.2.2.1 General

The following indications about the mechanical behaviour of this type of connection leave out of consideration the friction that sets up between the parts due to the weight of the supported element. In fact, in seismic conditions, under the contemporary horizontal and vertical shakes, the connection shall work instantly also in absence of weight.

#### 8.2.2.2 Behaviour models

This type of connection provides a hinged support in the vertical plane of the beam and a full support in the orthogonal vertical plane. In the longitudinal direction of the beam, the horizontal force, *R*, is transmitted through the shear resistance of the connection (see Figure 28), which is given by the shear resistance of the dowels and their local flexure between the elements corresponding to the bearing pad. In the transverse direction, omitting the vertical gravity loads, the connection transmits a shear force, *V*, together with the corresponding moment, *M* (see Figure 28).

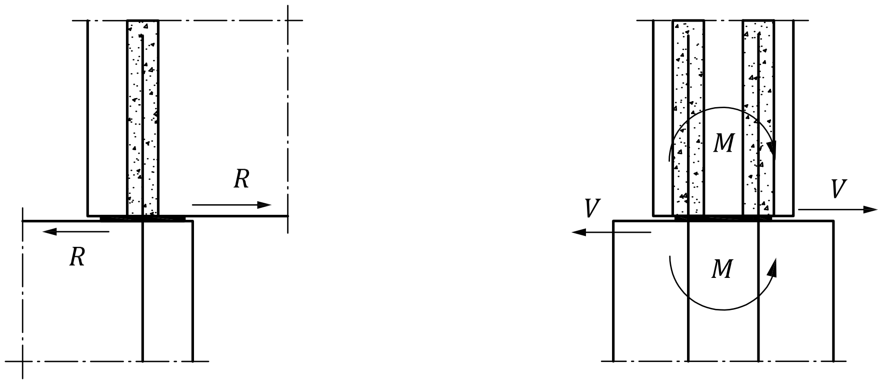



**Figure 28 — Behaviour models for beam-to-column connections**



### 8.2.2.3 Failure modes

The principal failure modes for longitudinal action are as follows:

a) breaking of the dowel connection due to combined shear, tension and flexure on steel bar and bearing stresses on concrete;

b) spalling of the concrete edge of beam due to tensile stress;

c) spalling of the concrete edge of column due to tensile stress.

The principal failure modes for transverse action are as follows:

a) flexural failure of the bearing section due to the action of *M*;

b) pull-out of the tensioned dowel under the action due to *M*;

c) sliding shear failure under the action of *V*.

### 8.2.2.4 Calculation formulae

With reference to Figure 28 a), for the action of a given force, *R*, evaluated by capacity design with respect to the resistance of the critical sections of the structure using the due overstrength factor, $\gamma_R$, the following shall be verified.

NOTE If the rotation of the joint is prevented by the stiffness of the connected elements, the numerical factor can be taken to 1,0.

For spalling of the concrete edges, the formulae taken from the literature are suggested in b) and c) as possible resistance verification.

a) dowel

$$R_{Rd} \geq R \quad (118)$$

with

$$R_{Rd} = 0{,}90^1 \cdot n \cdot \varphi^2 \sqrt{f_{yd} \cdot f_{cd}\left(1-\alpha^2\right)} \quad (119)$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \quad (120)$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \tag{121}$$

$$\alpha = \frac{\sigma}{f_{yk}} \tag{122}$$

where

$n$ is the dowels number;

$\phi$ is the dowels diameter;

$f_{yk}$ is the characteristic yield strength of steel;

$f_{ck}$ is the characteristic compressive strength of concrete;

$\sigma$ is the normal tensile stress due to other possible contemporary effects on the dowel.

b) beam edge

$$R_{Rd} \geq R \tag{123}$$

with

$$R_{Rd} = \frac{R_{Rk}}{\gamma_c} \tag{124}$$

$$R_{Rk} = 1{,}4 \cdot k \cdot \varphi^{\alpha} \cdot h^{\beta} \sqrt{f_{ck,cube} \cdot c^3} \tag{125}$$

$$k = \frac{b}{3c} \leq n \tag{126}$$

$$\alpha = 0{,}1 \sqrt{\frac{h}{c}} \tag{127}$$

$$\beta = 0{,}1 \sqrt[5]{\frac{\varphi}{c}} \tag{128}$$

$$h \leq 8\varphi \tag{129}$$

where

$\phi$ is the dowel diameter;

$h$ is the dowel effective length;

$f_{ck,cube}$ is the characteristic compressive cubic strength of concrete;

$c$ is the edge distance of the dowel axis;

$\gamma_c = 1{,}5$.

In case supplementary edge reinforcement (links or loops) of higher resistance is present, the above verification is applied with:

$$R_{Rk} = f_{yk} \cdot A_s \tag{130}$$

$$R_{Rd} = \frac{R_{Rk}}{\gamma_s} \tag{131}$$

where

$f_{yk} \leq 600$ N/mm$^2$;

$A_s$ is the is the sectional area of the bars parallel to the force, $R$, included within a distance less than or equal to 0,75 $c$ from the dowel in its effective length, $h$;

$\gamma_s = 1{,}5$.

c) column edge

$$R_{Rd} \geq R \tag{132}$$

with

$$R_{Rd} = \frac{R_{Rk}}{\gamma_c} \tag{133}$$

$$R_{Rk} = 1{,}4 \cdot k \cdot \varphi^{\alpha} \cdot h^{\beta} \sqrt{f_{ck,cube} \cdot c^3} \tag{134}$$

$$k = \frac{b}{3c} \leq n \tag{135}$$

$$\alpha = 0{,}1 \sqrt{\frac{h}{c}} \tag{136}$$

$$\beta = 0{,}1 \sqrt[5]{\frac{\varphi}{c}} \tag{137}$$

$$h \leq 8\varphi \tag{138}$$

where

$\phi$ is the dowel diameter;

$h$ is the dowel effective length;

$f_{ck,cube}$ is the characteristic compressive cubic strength of concrete;

$c$ is the edge distance of the dowel axis;

$\gamma_c = 1{,}5$.

In presence of supplementary edge reinforcement (links or loops) of higher resistance, the above verification is applied with:

$$R_{Rk} = f_{yk} \cdot A_s \tag{139}$$

$$R_{Rd} = \frac{R_{Rk}}{\gamma_s} \tag{140}$$

where

$f_{yk} \leq 600$ N/mm$^2$;

$A_s$ is the is the sectional area of the bars parallel to the force, $R$, included within a distance less than or equal to 0,75 $c$ from the dowel in its effective length, $h$;

$\gamma_s = 1{,}5$.

With reference to Figure 28 b), for the action of a force, $V$, and a moment, $M$, evaluated by capacity design with respect to the resistance of the critical sections of the structure using the due over strength factor, $\gamma_R$, the following shall be verified.

NOTE The values are $\gamma_R = 1{,}20$ for medium ductility structures and $\gamma_R = 1{,}35$ for high ductility structures.

d) flexure

$$M_{Rd} \geq M \tag{141}$$

with

$$M_{Rd} = A_s \cdot f_{yd} \cdot z \tag{142}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{143}$$

where

$A_s$ is the sectional area of the dowel;

$f_{yk}$ is the characteristic yield strength of steel;

$z$ is the lever arm of the couple of forces in the bearing print;

$z \approx d$ may be assumed with $d$ spacing of the two dowels;

$\gamma_s = 1{,}15$.

e) pull-out

$$l_b \cdot u \cdot f_{bd} \geq \gamma_R \cdot A_s \cdot f_{ym} \tag{144}$$

with

$$f_{bd} = 0{,}45\ f_{md} \quad \text{is the ultimate bond strength} \tag{145}$$

$$f_{ym} = 1{,}08\ f_{yk} \quad \text{is the mean yielding stress of the steel} \tag{146}$$

where

$l_b$ is the anchorage length of the dowels in the beam;

$u$ is the perimeter of the dowels in the beam;

$A_s$ is the sectional area of a dowel;

$f_{ctd}$ is the design cylinder compressive strength of the mortar;

$f_{yk}$ is the characteristic yield strength of steel.

f) sliding shear

$$V_{Rd} \geq V \tag{147}$$

with

$$V_{Rd} = V_{dd} + V_{fd} \tag{148}$$

$$V_{dd} = 1{,}3 A_s \sqrt{f_{cd} \cdot f_{yd}} \tag{149}$$

$$V_{fd} = 0{,}25\ b \cdot x \cdot f_{cd} \tag{150}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \tag{151}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{152}$$

where

$V_{dd}$ is the resistance of the shear resisting (compressed) dowel;

$V_{fd}$ is the sliding resistance of the compressed concrete;

$A_s$ is the sectional area of dowels not yielded by the contemporary flexure;

$b$ is the width of the bearing print;

$x$ is the depth of bearing print (compressed part);

$f_{ck}$ is the characteristic compressive strength of beam concrete (or column if lower);

$f_{yk}$ is the characteristic yield strength of steel;

$\gamma_c$ = 1,5;

$\gamma_s$ = 1,15.

#### 8.2.2.5 Other properties

Failure mode b related to a tensile cracking of the concrete edge of the beam corresponds to the weakest mechanisms indicatively for $c/\phi < 6$ where $c$ is the edge distance of the dowel axis. For $c/\phi > 6$, failure mode a related to the dowel strength is the weakest one.

In the beam, the ordinary longitudinal reinforcement made of bars of large diameter does not prevent concrete spalling also if these bars are well anchored by hooks bent at 135°. In fact, the bars come into effect only after the cracking of concrete, but at this point, the support can be jeopardized. In order to control the crack opening and prevent the failure by spalling, effective edge reinforcement shall be added made of small diameter U-shape horizontal links closely distributed along the lower part of the beam. These horizontal links are particularly important for small $c/\phi$ ratios as they restrain the dowels after the cracking of concrete. They shall be distributed over $8\phi$ from the bottom with a spacing not greater than 50 mm and dimensioned for a design resistance equal to the expected action.

To prevent the failure by spalling of the column, a closely spaced distribution of confining stirrups shall be placed at the top end together with steel links to the dowels and a grid of bars at the upper face. For a height equal to the larger side of the cross-section, the stirrups shall be distributed with a spacing not greater than 100 mm and a confining volumetric mechanical ratio of at least 0,08. The steel links shall be placed in the direction of the action, distributed within $8\phi$ from the top of the column for a $2c$ width around the dowels and dimensioned for a design resistance equal to the expected action.

Calculation formulae of cases a, b and c refer to the use of flexible rubber bearing pads. Rigid steel pads have not been tested in the present research.

### 8.2.3 Ductility

In testing, failure mode a of dowel connection displayed a local shear ductility, $\mu$, due to the flexural and tensional deformation of the dowels within the joint gap, evaluated in:

$$4{,}0 < \mu < 6{,}0 \quad \text{for } c/\phi \geq 6 \tag{153}$$

$$2{,}5 < \mu < 3.5 \quad \text{for } c/\phi < 6 \tag{154}$$

Often, an early brittle spalling of the beam edge occurred. With a sufficient bearing length, spalling can be eliminated from consideration for the stability of the support.

### 8.2.4 Dissipation

Cyclic tests performed in the longitudinal direction of the beam show a medium dissipation capacity due to the alternate deformations of the dowels within the joint gap. For small thicknesses of this gap, the dissipation capacity decreases sensibly. Also, crushing of the concrete around the dowels occurs for large shear displacements reducing the energy dissipation capacity.

In any case, due to their location in the structural assembly and to their high stiffness in comparison to the column flexibility, no contribution of ductility and dissipation are expected from this type of connections to the global ductility of the structure. They shall be over-proportioned by capacity design with respect to the critical sections of the column base.

### 8.2.5 Deformation

In cyclic tests, relative displacements up to ±36 mm have been reached at the ultimate amplitudes before failure.

### 8.2.6 Cyclic decay

Cyclic tests show that, at any displacement level before failure, strength decay occurs after each of the three cycles. At the third cycle, this strength decay can reach the 25 % of the value of the first cycle.



### 8.2.7 Damage

Damage is associated with spalling of the concrete beam end, splitting of the concrete at both beam and column and breaking of the dowels. Typically, the first two modes of damage lead to a reduction of the shear resistance but not to failure.

Where bearing pads are used, failure occurs with the tensile rupture of the dowels after large plastic deformations. Depending on the diameter of the dowels and the ratio $c/\phi$, dowels can break after or before significant spalling at the concrete edges. The breaking point is usually located at a depth, approximately equal to $2\phi$ within the beam or the column. As a result, breaking of the dowels does not necessarily lead to a total loss of resistance, since a portion of the broken dowels extrudes from the column or the beam inside the opposite element and continues to pose significant resistance against horizontal movement.

## 8.3 Mechanical coupler connections

### 8.3.1 General

This type of connection refers to over-designed mechanical devices that achieve the flexural continuity between the connected members through high resistance bolts. The gap is filled with high resistance no-shrinkage mortar to ensure the continuity with the members' concrete. The mortar shall have at least the same resistance of the concrete. Figure 29 shows in plan and elevation the end connection of a beam to a column in the typical arrangement of a half-joint

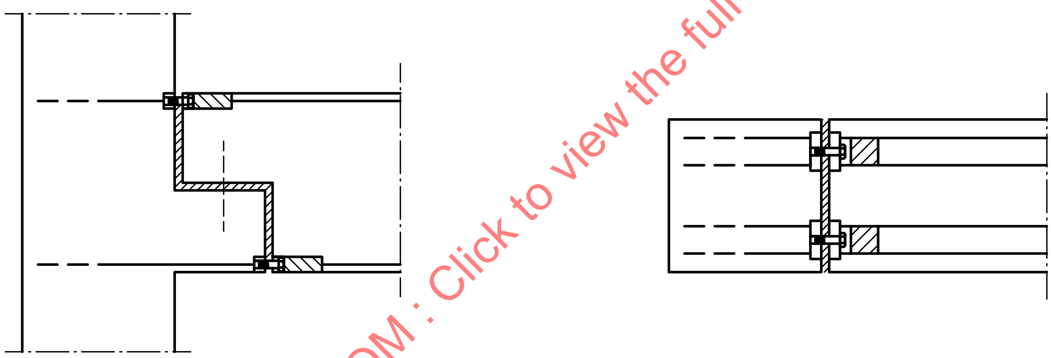


**Figure 29 — Moment resisting beam-to-column connection**

The details of the coupling devices are shown in Figure 30. In case a) of this figure the reinforcing bars are connected to two plates placed in each member. Bolts are placed between the plates to achieve the connection. In case b), one plate, to which the reinforcing bars are fixed, is placed on the beam, while the column is provided with a reinforcement with a threaded end, in which the coupling bolt is directly screwed.

If the concrete section is sensibly weakened by slots for the installation of the bolt(s), the section shall be restored and properly confined. This type of connection is normally used in combination with dowels (see 8.2) and can be activated in a second stage during erection. In the latter case, transitory construction phases shall be checked.



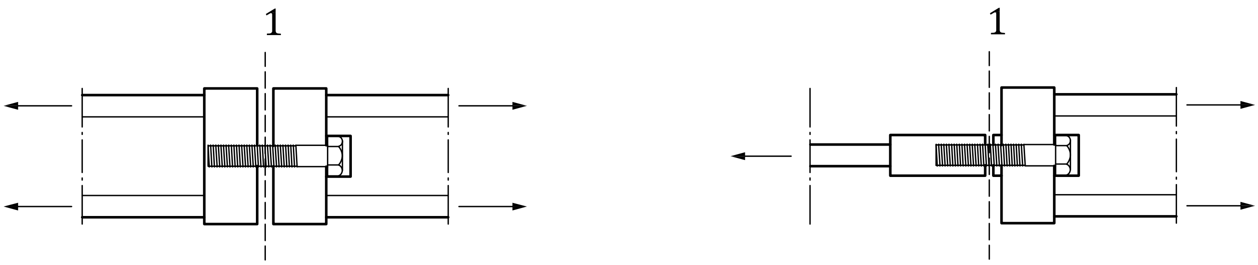



**Figure 30 — Details of the coupling devices**

### 8.3.2 Strength

#### 8.3.2.1 Behaviour models

This type of connection provides a clamped support. The bolts are mainly acting in tension. In some cases they can also act in compression (if a proper counter-nut is provided). In the longitudinal direction, the horizontal force due to the bending moment is directly transmitted to the reinforcing bars through the connection. The mortar filling acts in compression under flexure.

The shear force coming from the beam is assumed to go entirely on the corbel standing out from the column. Proper detailing shall be provided for the reinforcement of the corbel and the beam end following concrete design rules.

Since this type of connection is normally used in addition to dowels, the horizontal shear transmission and the flexural transverse resistance are still carried by the dowels. When the mortar filling is hardened, the union between the beam and the joint can be considered as monolithic.

#### 8.3.2.2 Failure modes

The principal failure modes are as follows:

a) breaking of the coupler (bolt);

b) excessive deformation of the supporting plate(s);

c) detachment of the reinforcing bars.

#### 8.3.2.3 Calculation formulae

This type of connection is placed in critical zones. With reference to Figure 30, for the action of a force evaluated by capacity design with respect to the greater between the resistance of the two reinforcements connected by the coupler using the due over-strength factor, the following shall be verified.

a) coupler (non ductile)

provided that the threaded length and the washers are correctly designed, the coupler shall be over-designed as follows:

$$F_{R\min} \geq \gamma_R \cdot A_s \cdot f_{ym} \tag{155}$$

with

$$f_{ym} = 1{,}08\, f_{yk} \quad \text{is the mean yielding stress of the steel} \tag{156}$$



where

$F_{Rmin}$ is the minimum steel ultimate capacity of the fastener declared by the producer;

$A_s$ is the maximum sectional area between the two corresponding upper reinforcements;

$f_{yk}$ is the characteristic yield strength of steel;

$\gamma_R$ = 1,20 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.

b) plate(s)

plate(s) shall be over-proportioned in thickness in order to avoid sensible deformation at failure limit.

c) reinforcement

proper connection between the reinforcement and the plate(s) shall be designed by initial type testing. In case of direct welding, special care and controls are suggested to avoid weakening of the reinforcement.

#### 8.3.2.4 Other properties

Since this type of connection is over-resisting, failure is expected to occur away from the connection (in the current reinforcement). Since high resistance bolts are usually adopted as couplers, their cross-section can be less than the one of the reinforcement they are linking. Thus, flexural cracks are expected to open also within the connection. The cracked stiffness of the member should be calculated considering the cross-section in correspondence of the couplers.

Special care is suggested for the mortar filling process, especially if dealing with complex three-dimensional surfaces and small gaps.

### 8.3.3 Ductility

The ductility of the member is not influenced by the connection, which is over-resisting and designed to remain in elastic range.

### 8.3.4 Dissipation

The energy dissipation of the structure does not depend on the contribution of the connection itself. Local damage of the mortar or other effects can create a permanent gap in the joint, affecting the cyclic performance of the connection. Double nuts are suggested to be used to reduce this effect (making the bolt acting also in compression). High tightening moments can be used to delay the decompression and the opening of the joint.

### 8.3.5 Deformation

A slightly larger flexural deformation of the member with this type of connection, if compared to a cast-in-situ solution, is expected, due to the elastic elongation of the coupling bolts.

### 8.3.6 Cyclic decay

Cyclic tests show that at any displacement level before failure no relevant strength decay shows after the three cycles.

### 8.3.7 Damage

Failure is expected to occur out of the connection, while damage (cracking) can occur at the joint.



## 8.4 Hybrid connections

### 8.4.1 General

Figure 31 shows the end connections of beams to the corbels standing out from a supporting column. The term “hybrid” refers to the connection arrangement made at the upper part with additional bars and cast-in-situ concrete proper of an emulative joint and at the lower part with mechanical steel devices proper of a typical joint. The upper cast-in-situ slab is connected to the precast beams by the protruding stirrups that resist the longitudinal shear transmitted through the interface. The lower connection can be made with the welded solution described in 7.4 for a rib of a floor element providing for the proper dimensional adaptations.

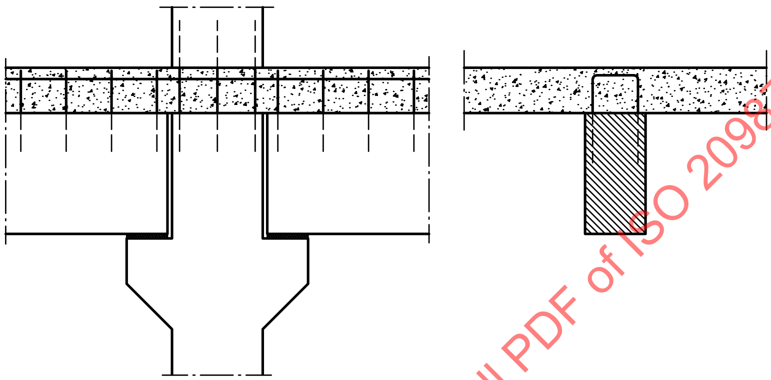


**Figure 31 — Beam-to-column hybrid connection with corbels**



### 8.4.2 Strength

This type of connection, after the hardening of the cast-in-situ concrete slab, provides a moment resisting support between the parts, with an asymmetrical behaviour for positive and negative moments. The loads applied after the hardening of the slab take their action on this moment resisting connection. The self-weight of the beam, included the upper concrete slab, acts on a simple hinged support arrangement.

#### 8.4.2.1 Behaviour models

In both stages of hinged and fixed support, the shear force coming from the beam is assumed to go entirely on the corbel standing out from the column. The local detailing and design calculation of the corbel shall be made following the provisions of concrete design.

Figure 32 shows the resisting mechanisms respectively for negative and positive moments. In the first mechanism, the tensile force, $T$, acts in the longitudinal bars added in the cast-in-situ topping and the compressive force, $C$ (=$T$), comes from the bottom welding with a lever arm, $z'$. In the second mechanism the compressive force, $C$, acts in the cast-in-situ topping and the tensile force, $T$ (=$C$), comes from the bottom welding with a lever arm, $z$.

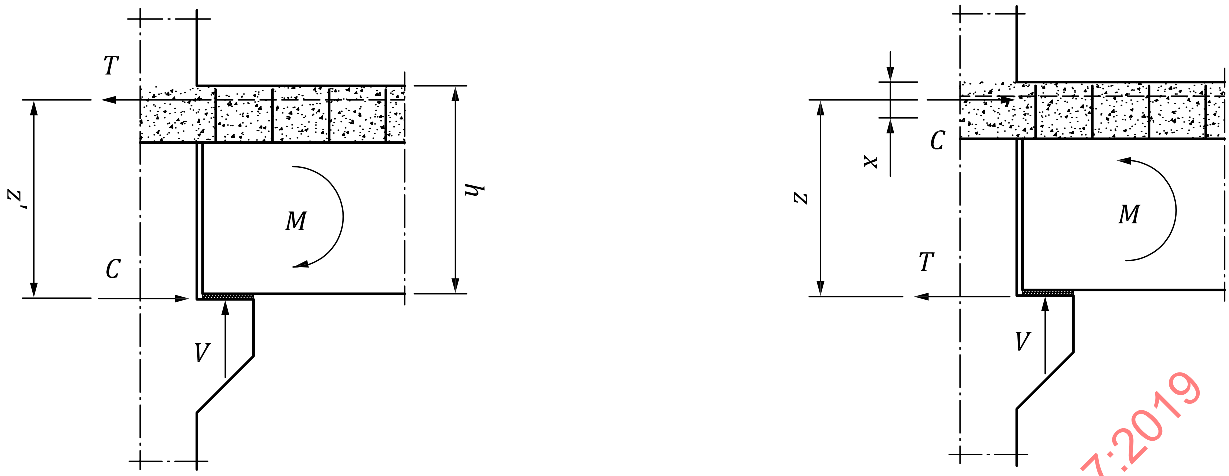



Figure 32 — Behaviour models for hybrid beam-to-column connections



#### 8.4.2.2 Failure modes

For a negative moment the principal failure modes are as follows.

a) flexural failure of the connection referred to the yielding of the longitudinal upper bars;

b) bond failure of the anchorage of the upper bars;

c) longitudinal shear failure at the interface between precast element and cast-in-situ slab;

d) failure of the bottom connection between the rib and the supporting corbel.

For a positive moment, the principal failure modes are as follows.

a) flexural failure of the connection referred to the rupture of the bottom connection;

b) longitudinal shear failure of the interface between precast beam and cast-in-situ slab.

#### 8.4.2.3 Calculation formulae

For a negative moment [see Figure 32 a)], the ultimate resisting can be calculated by:

$$M_{Rd} = A_{st} \cdot f_{yd} \cdot z' \tag{157}$$

with

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{158}$$

where

$A_{st}$ is the total sectional area of the longitudinal upper bars;

$f_{yk}$ is the characteristic yield strength of steel;

$z'$ is the lever arm;

$\gamma_s$ = 1,15.

a) flexure

$$M_{Rd} \geq M_{Ed} \tag{159}$$

where $M_{Ed}$ is the design value coming from the structural analysis. For the design of the frame resisting system under seismic action, the resisting moment $M_{Rd}$ could enter in the capacity design calculation together with the competent resisting moments of the other members convergent in the node.

b) bar anchorage

$$l_b \cdot u \cdot f_{bd} \geq \gamma_R \cdot A_s \cdot f_{ym} \tag{160}$$

with

$$f_{bd} = 2{,}25\, f_{ctd} \quad \text{is the ultimate bond strength} \tag{161}$$

$$f_{ym} = 1{,}08\, f_{yk} \quad \text{is the mean yielding stress of the steel} \tag{162}$$

where

$l_b$ is the anchorage length of a bar in the upper slab;

$u$ is the perimeter of a bar in the upper slab;

$A_s$ is the bars area in the upper slab;

$f_{ctd}$ is the design tensile strength of the cast-in-situ concrete;

$f_{yk}$ is the characteristic yield strength of steel.

c) longitudinal shear

$$A_{ss} \cdot f_{yd} \geq \gamma_R \cdot A_{st} \cdot f_{ym} \tag{163}$$

where

$A_{SS}$ total area of protruding stirrups available in the end segment long h of the beam;

$f_{yd}$ is the design tensile yielding stress of steel;

$\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.



d) bottom connection

Verifications *a*, *b* and *c* of point 7.4.2.3 shall be applied referring to an acting force *R*.

$$R = \gamma_R \cdot A_{st} \cdot f_{ym} \tag{164}$$

For a positive moment (see Figure 32b) the resisting value can be calculated by

$$M_{Rd} = R_R \cdot z \tag{165}$$

with

$$z = h - \frac{x}{2} \leq 0{,}96h \tag{166}$$

$$x = \frac{R_R}{f_{cd} \cdot b} \tag{167}$$

where

$R_R$ minimum resistance of the bottom connection calculated from all the failure modes covered by 7.4.2.3;

$z$ is the lever arm;

$b$ is the collaborating width of the upper slab;

$f_{cd}$ is the design compressive strength of the cast-in-situ concrete.

e) flexure

$$M_{Rd} \geq \gamma_R \cdot M_{ER} \tag{168}$$

where $M_{Ed}$ is the design value coming from the structural analysis and for seismic action condition could be calculated by capacity design with a proper overstrength factor $\gamma_R$.

f) longitudinal shear

$$A_{ss} \cdot f_{yd} \geq \gamma_R \cdot R_R \tag{169}$$

where

$A_{SS}$ total area of protruding stirrups available in the end segment long h of the beam;

$f_{yd}$ is the design tensile yielding stress of steel;

$\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.

#### 8.4.2.4 Other properties

For positive moments, a ductile flexural behaviour can be provided by the end segment of the beam with its lower longitudinal reinforcement, the welded connection being overdimensioned.

### 8.4.3 Ductility

In testing, a displacement ductility over 3,5 has been always measured. This refers to the testing arrangement that includes a relevant part of the beam so that the measurements refer mainly to the flexural contribution of the beam. The connection itself is expected to display a good ductility for negative moments, coincident with the beam ductility, and display no relevant ductility for positive moments. If the bottom connection is overdimensioned by capacity design, the latter behaviour does not endanger the ductility capacities of the beam.

In general, it can be assumed that this type of connection, if properly designed following the rules given above, saves the full capacities of the beam (between medium and high ductility).

### 8.4.4 Dissipation

Cyclic tests performed show a medium dissipation capacity that is to be attributed to the beam.

### 8.4.5 Deformation

In cyclic tests, drifts of about 2 % have been reached for positive moments (upper slab in compression), of about 1 % for negative moments (upper slab in tension).

### 8.4.6 Cyclic decay

Limited strength decay has been measured after the three cycles of any amplitude before failure.

### 8.4.7 Damage

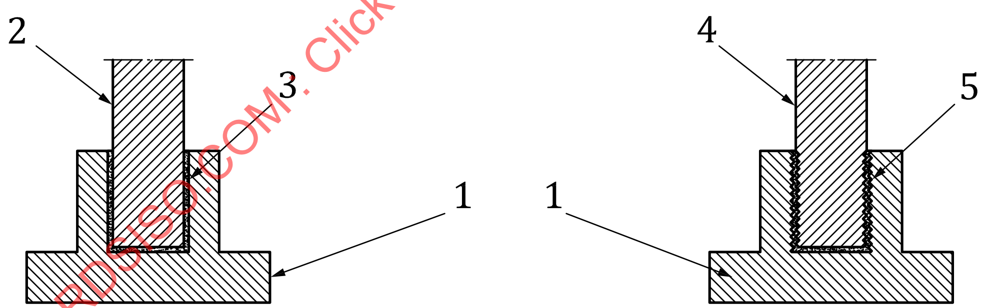
For drifts larger than 1 %, relative rotations have been observed between the beam and the column. Plastic flexural deformations occurred in the beam for higher drifts with the yielding of the longitudinal tensioned bars.

# 9 Column-to-foundation connections

## 9.1 Pocket foundations

### 9.1.1 General

Figure 33 shows two possible solutions for the connection of a column to the supporting foundation. For both solutions the column is inserted within the pocket delimited by the four walls of the foundation. It is placed on a pad over the bottom footing slab. After the centring of the column, fixed with proper provisional bracing props, the bottom gap to the footing and the peripheral gap to the walls are filled with no-shrinkage mortar. The pocket shall be large enough to enable a good compacted filling below and around the column. In the left solution, the surfaces of column and foundation within the joint are smooth. In the right solution, they are wrought with indentations or keys so to increase the adherence of the mortar.




**Key**

1 pocket foundation
2 column
3 smooth surfaces
4 column
5 rough surfaces

**Figure 33 — Pocket foundations**

For sway frames, where the stability of the structure relies on the flexural strength of the column, a minimum insertion depth of the column is recommended with $l \geq 1.2h$, where $l$ is the insertion depth and $h$ is the maximum side of the column section.

### 9.1.2 Strength

For sway frames, the connection shall be verified for the action of the (plastic) ultimate moment $M_{Rd} = M_{Rd}(N)$ of the adjacent column section with the corresponding contemporary axial force, $N$, and of shear, $V$. This can be calculated in the two main directions independently. The due overstrength factor, $\gamma_R$[1], shall be added with $\gamma_R \cdot M_{Rd}$, N and $\gamma_R \cdot V$.

In particular pocket foundations with wrought, surfaces are assumed as monolithic and the verification refers mainly to the proper overlapping of the vertical bars of column and pocket walls. For pocket foundations with smooth surfaces a behaviour model referred mainly to a system of reaction forces orthogonal to the adjacent surfaces can be adopted.

NOTE The values are $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.

### 9.1.3 Other properties

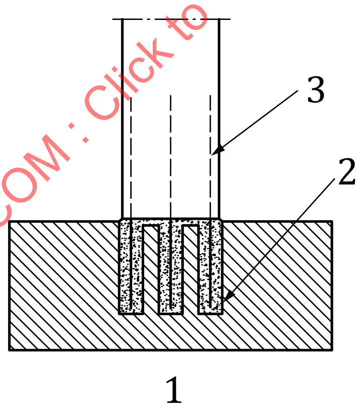
No specific parameters of seismic behaviour (ductility, dissipation, deformation, decay, damage) have been experimentally measured for this type of connection, for which no ductility and dissipation capacities are expected.

## 9.2 Foundations for columns with protruding bars

### 9.2.1 General

Figure 34 shows the connection of a column to the foundation obtained by the anchorage of the reinforcing longitudinal bars protruding from the base of the column within the corrugated sleeves inserted in the foundation and filled with no-shrinkage mortar. Due to their size (80 mm to 100 mm diameter), the sleeves jut out of the column profile in the wider dimension of the foundation element so that the longitudinal bars can enter without deviating from their straight peripheral position in the column.

The column itself settles on a bed of mortar that fills up the joint. This bed shall be sufficiently thin to avoid the buckling of the bars within the gap when subjected to strong compression. Otherwise, a proper confining reinforcement shall be added.




**Key**

1 foundation element

2 sleeves

3 reinforcing bars

**Figure 34 — Foundations for columns with protruding bars**

The steel reinforcement inside the column does not need any special adaptation for the connection. The length of the protruding part of the bars shall be over-proportioned by capacity design to avoid brittle bond failure of the anchorage before the yielding of the bars in the critical region at the base of the column. The protruding bars shall be protected during transportation to avoid accidental distortion.

An alternative solution keeps the protruding parts of the bars separated. Just before the installation in site of the column, these cropped parts are screwed in a bush previously fixed at the end of the internal part of the longitudinal reinforcement. The threading weakens the bars and can jeopardize the strength of the connection, leading to early brittle failure. Special technological provisions shall be adopted to maintain the strength hierarchy of the connection devices that allows exploiting the full ductility resources of the column.

Proper reinforcement shall be placed in the foundation element to confine the concrete around the sleeves and anchor them against pull-out. The sleeves shall be filled with fluid mortar just before placing the column, which verticality shall be adjusted with timber wedges driven at the base and ensured by lateral provisional props until the mortar has hardened and sufficiently aged.

### 9.2.2 Strength

#### 9.2.2.1 General

The connection shall be verified for the action of the (plastic) ultimate moment, $M_{Rd} = M_{Rd}(N)$, of the adjacent column section with the corresponding axial force, $N$, and of shear, $V$. This can be calculated in the two main directions independently. The due overstrength factor and, $\gamma_R$, shall be added as specified 9.2.2.3.

#### 9.2.2.2 Behaviour models

Figure 35 shows the details of the resisting mechanism of the foot section of the column subjected to the combined bending moment, $M_{Rd}$, and axial action, $N$, and to the shear, $V$. Assuming that, at this level of action, the tensile reinforcement is yielded, the anchorage verification shall refer to a pull-out force corresponding to the mean yielding stress, $f_{ym}$, of the bar and to a fully confined mortar.

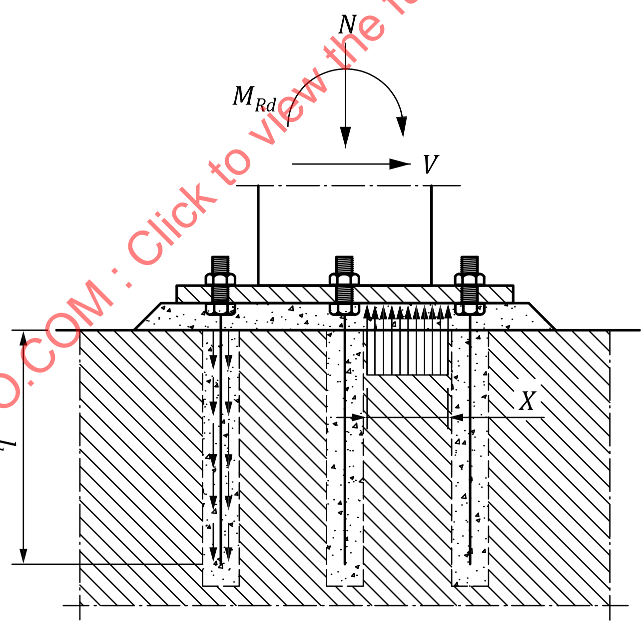



**Figure 35 — Resisting mechanism of Column-to-Foundation connection**

#### 9.2.2.3 Failure modes

The principal failure modes are as follows.

a) pull-out of the tensioned bars of the foot section under the combined action of $\gamma_R \cdot M_{Rd}$, and $N$;

b) sliding shear failure at the foot section in the design situation corresponding to $\gamma_R \cdot M_{Rd}$, $N$ and $\gamma_R \cdot V$.

#### 9.2.2.4 Calculation formulae

With reference to the symbols described in Figure 35 and with $\gamma_R$ overstrength factor, the following shall be verified [see Formulae (170) to (180)].

NOTE The values are $\gamma_R$ = 1,2 for medium ductility structures and $\gamma_R$ = 1,35 for high ductility structures.

a) pull-out

$$l_b \cdot \mu \cdot f_{bd} \geq \gamma_R \cdot A_s \cdot f_{ym} \tag{170}$$

with

$\mu = \pi\varphi'$ is the area of the upper bar section (171)

$A_s = \dfrac{\pi\varphi^2}{4}$ is the area of the upper bar section (172)

$f_{bd} = 0{,}45 f_{md}$ is the ultimate bond stress (173)

$f_{ym} = 1{,}08 f_{yk}$ is the mean yielding stress of the steel (174)

where

$l_b$ is the anchorage length of a bar;

$\phi$'is the diameter of the protruding bar;

$\phi$ is the diameter of the upper bar;

$f_{md}$ is the design cylinder compressive strength of the mortar;

$f_{yk}$ is the characteristic yield strength of steel.

b) sliding shear

$$V_{Rd} \geq V \tag{175}$$

with

$V = V(\gamma_{Rd} \cdot M_{Rd})$ is the shear corresponding to $\gamma_R \cdot M_{Rd}$ (176)

$$V_{Rd} = V_{dd} + V_{fd} \tag{177}$$

$V_{dd} = 1{,}3 A_d \sqrt{f_{cd} \cdot f'_{yd}}$ is the dowel resistance of the shear resisting bars (178)

$V_{fd} = 0{,}50\, b \cdot x \cdot f_{cd}$ is the sliding resistance of the compressed mortar (or concrete) (179)

$$f'_{cd} \approx 0{,}5 f_{cd} \tag{180}$$

