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Simplified design of connections of concrete claddings to concrete structures

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Foreword

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

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

The current design practice of reinforced concrete buildings, most commonly precast, is based on a frame model, where the peripheral cladding panels enter only as masses without any stiffness. The panels are then connected to the structure with fastenings dimensioned with a local calculation based on their mass for anchorage forces orthogonal to the plane of the panels.

Furthermore, the seismic force reduction in the type of reinforced concrete structures of concern relies on energy dissipation in plastic hinges formed in the columns. Very large drifts of the columns are needed to activate this energy dissipation foreseen in design. However, typically, the capacity of the connections between cladding and structure is exhausted well before such large drifts can develop. Therefore, the design of these connections cannot rely on the seismic reduction factor typically used for design of the bare structure.

This document contains a set of practical provisions for the design of mechanical connections of concrete claddings to concrete structures under seismic actions as well as suggestions for structural analysis for the specified systems.

Simplified design of connections of concrete claddings to concrete structures

1 Scope

The present document refers to the panel-to-structure and panel-to panel connections used for the cladding systems of reinforced concrete frame structures of single-storey buildings, typically precast. They can be used also for multi-storey buildings with proper modifications.

The fastening devices considered in the present document consist mainly of steel elements or sliding connectors. Dissipative devices with friction or plastic behaviour are also considered. Other types of common supports and bond connections are treated where needed.

The use of any other existing fastening types or the connections with different characteristics than those described in the following clauses is not allowed unless comparable experimental and analytical studies do provide the necessary data and verify the design methodology for the particular type.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 20987, *Simplified design for mechanical connections between precast concrete structural elements in buildings*

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
behaviour factor *q*
q factor by which the elastic design spectrum in linear analysis is reduced

Note 1 to entry: Directly or indirectly linked to the ductility and deformation demands on members and connections.

4 Generalities

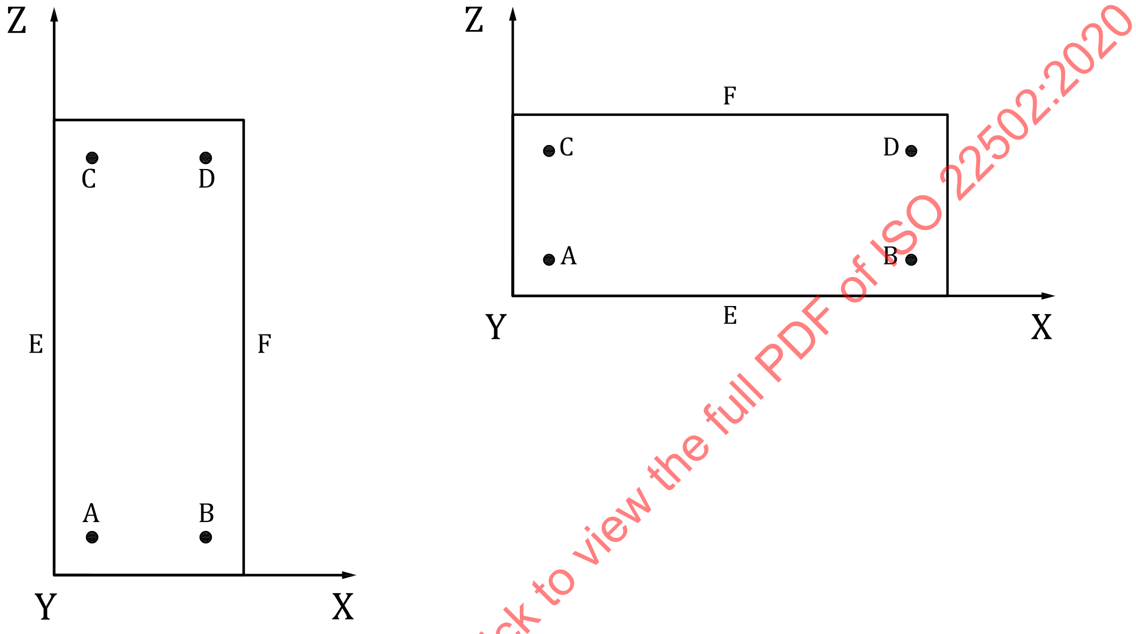
4.1 Cladding panel orientations

Figure 1 a) shows a vertical panel orientation referred to a system of orthogonal axes, where x is oriented horizontally in the panel plane, y is oriented orthogonally to that plane and z is oriented vertically parallel to the gravity loads. The origin is placed in a corner at the base side of the panel.

Four connections are foreseen at the corners of the panel, indicated respectively by A, B, C and D. Any one of these connections is intended to give only translational restraints without any rotational restraint. E and F indicate the possible joint connections with the adjacent panels. Usually, the connections A and B are attached to the foundation beam, the connections C and D are attached to the top beam.

The couple of bottom and top connections may be replaced by single connections placed in the middle of the bottom and top sides for a pendulum arrangement of the panel. In this case, the connections are respectively named A and C, and the symbols B and D are omitted.

In Figure 1 b), the same reference system is associated with a horizontal panel for which the connections A, B, C and D are usually attached to the columns, and E and F refer to the possible joint connections with the adjacent panels, foundation or top beam where the uncertain friction effect can act due to the superimposed panels.



a) Vertical **b) Horizontal**

Figure 1 — Cladding panel orientations

Table 1 — Symbols and graphic schemes for supports

Symbol	Description	Graphic scheme
f	fixed (bilateral)	▼ ►◄,▲
f+	fixed (unilateral in + direction)	▲, ►
f-	fixed (unilateral in - direction)	▼, ◄
s	sliding (bilateral)	↔, ↕
d	dissipative	ΛΛΛ
/	omitted	[empty]

Table 1 gives a general description of the symbols and graphic schemes regarding the effect of the supports along the three directions x, y and z. As an example, Table 2 gives the arrangement matrix indicating the effect of the supports for a vertical panel.

Table 2 — Arrangement matrix – example

Direction	A	B	C	D	E	F
x	f	/	s	/	f	f
y	f	/	f	/	/	/
z	f	/	/	/	d	d

The term “fixed” is used with reference to the restrained linear displacement while the rotational restraints are not provided.

4.2 Design criteria to connect frame and panels

4.2.1 Isostatic approach

An isostatic arrangement of panel connections is able to allow without reactions the large displacements expected for the frame structure under earthquake conditions. Very large displacement capacities are required for connectors with this choice.

The frame deformation demand is allowed by a relative clearance that uncouples the motion of frame and panels. The two systems are kinematically uncoupled, except for the out-of-plane displacements [see Figure 2 a)].

4.2.2 Integrated approach

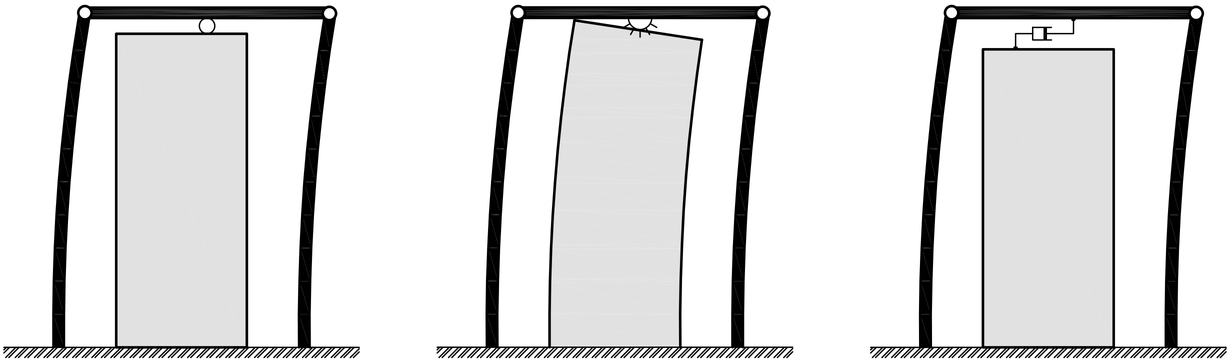
An integrated arrangement relies on fixed connections that integrate the panels in the resistant structural assembly with a dual wall-frame system behaviour. High forces may arise in the connections with this choice.

Panels and frame have a coupled motion: the system is kinematically paired [see Figure 2 b)]. Panels become part of the seismic resisting system and they act as the main restraints in the horizontal direction thanks to their higher stiffness. As a consequence, the connections shall be over-proportioned to carry the higher loads transferred by the frame, according to capacity design rules.

4.2.3 Dissipative approach

An arrangement of dissipative connections between the panels is added to an isostatic system of fastenings to the structure, able to maintain displacements and forces within lower predetermined limits.

Specific devices can balance the overall building response, reducing the displacement and keeping the load below an imposed threshold, determined by the connections themselves [see Figure 2 c)]. Like in the isostatic configuration, the systems are kinematically uncoupled but they are also constrained by inelastic links, like friction or yielding devices. The joints between structure and panels – or among the panels – shall be designed to dissipate energy during the seismic action.



a) b) c)

Figure 2 — Design criteria to connect frame and panels

4.3 Strategies to implement isostatic and dissipative design criteria

4.3.1 Sliding-frame (SF)

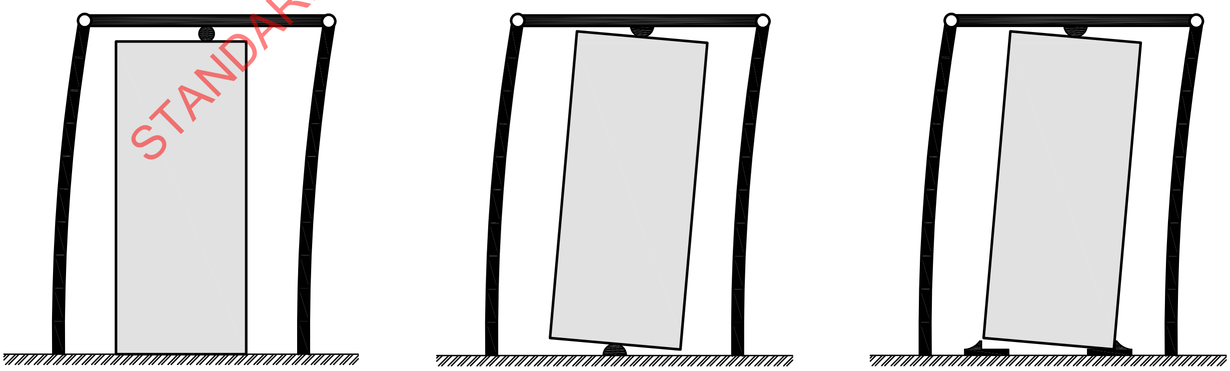
Like an ideal uncoupled system, the isostatic sliding-frame is, in principle, the easiest way to disconnect frame and panels. To achieve this result while avoiding the issues that affect current systems, proper connections (sliders) shall be introduced. They only restrain out-of-plane motions, reproducing the hypothesis typically assumed in the current practice, but in a safer way [see Figure 3 a)].

4.3.2 Double-hinged pendulum (DHP)

The double-hinged pendulum is the proper way to connect the cladding as simple mass without any stiffness contribution [see Figure 3 b)]. This result can be obtained either by connecting panel edges with hinges, or by replacing the top hinge with coupled sliders.

4.3.3 Rocking panel (RP)

Starting from DHP, the rocking panel configuration may be obtained replacing the bottom hinge with a pair of horizontal restraints. These leave the panel free to rock around its bottom corners. Even though this solution looks very similar to the former one, some differences in statics and in kinematic behaviour need to be highlighted [see Figure 3 c)].



a) b) c)

Figure 3 — Isostatic and dissipative design criteria: schemes of design strategies

4.4 Parameters

ISO 20987 shall apply. In addition to the provisions of ISO 20987, the following applies.

Among the main parameters that characterize the seismic behaviour of the connection, the following one is added:

slide - free linear relative displacement capacity with null or negligible reaction.

The main behaviour parameters are provided for each x, y, z direction defined in ISO 20987 specifying possible interaction effects.

4.5 Classification

ISO 20987 shall apply. In addition to the provisions of ISO 20987, the following applies.

Connections present in existing buildings, where sufficient information about their strength and/or ductility is not available, can be classified as unknown.

Existing connections can be classified as insufficient when a specific calculation under the expected seismic action shows their inadequate strength.

5 Isostatic systems

5.1 General

For buildings with isostatic arrangements of cladding panel connections, the structural analysis under seismic action shall refer to the frame system following the conventional design practice of such structures. In expectation of large displacements, the second order effects, $P\Delta$, should be taken into account. In addition to the ordinary output data used for the design of member resistance at ultimate limit state (ULS[1)]), the sliding or rotation displacements shall be provided for the design of the pertinent capacities of panel connection devices.

5.2 Analysis of the building

5.2.1 General

For the frame systems considered, capacity design criteria for the proportioning of the connection are applied. It is assumed, as a rule, that the beam-to-column and column-to-foundation connections are properly over-proportioned with respect to the bending moment ultimate capacities of the columns. Floor connections involved in the diaphragm action can refer to some approximate methods.

In any case, the structural connections can be over-proportioned, referring to the forces obtained from a structural analysis performed with behaviour factor $q = 1{,}5$

Figure A.1 shows a simplified design flowchart. It shows the required steps to design a cladding to concrete structure connection. Specific suggestions regarding the isostatic systems structural model analysis are given in 5.2.2 and 5.2.3.

5.2.2 Suggestions for the structural model

For the numerical model of the structure, the ordinary linear elements (beam type) can be used, positioned along the axis of the members. Different eccentricities between the members should be reproduced using link rigid elements at their joints. The connections between the elements shall be faithfully represented with their degrees of freedom in the different planes. It should be considered that, if the connections are modelled with no deformability (e.g. fixed built-in full support or hinged

1) ULS: state at which the material stresses are limited to the point at which the bearing elements can withstand the design loads and maintain the safety and integrity of the structure.

support), the results of the analysis can lead to very high joint forces. The actual deformability of the connections, even small, can substantially lower these forces. More reliable results can be obtained if the actual deformability of the connections is reproduced in the model.

The floor elements can be modelled as linear elements concentrating their mechanical properties along the axis. To reach the actual points of their connections, link rigid elements can protrude from the axis. The diaphragm action of the floors shall be properly represented, implicitly by the layout of their members or explicitly through the options provided by the computation code.

If the cladding panels are introduced as members in the model, they can be reproduced as linear elements distributing their weight along the axis. Their supports shall faithfully reproduce the isostatic arrangement of the connections. To reach the actual points of the connections, where some response parameters are needed, rigid link elements can protrude from the panel axis.

If the cladding panels are introduced as masses in the model, their total mass, M, shall be transferred to the sustaining members in a ratio, R_y, depending on the connection arrangement.

For one-storey structures with vertical panels in the horizontal orthogonal y direction, this ratio, R_y, is given by Formula (1):

$$R_y = \frac{0{,}67\,Mh}{h_o} \tag{1}$$

where

h is the height of the panel;

h_o is the elevation of its upper support connected with the roof deck;

M is the total mass of the cladding panels in the model.

In the in-plane horizontal x direction, the same ratio, $R_x = R_y$, can be assumed for a pendulum support arrangement. A null ratio, $R_x = 0$, can be assumed for a cantilever arrangement with upper sliding connections.

For one-storey structures with horizontal panels in the orthogonal y direction, their mass, M, shall be shared between the two lateral supporting columns, amplified as a function of their elevation h_i [see Formula (2)]:

$$R_y = \frac{0{,}5\,Mh_i}{h_o} \tag{2}$$

where h_o is the elevation of the roof deck.

In the in-plane horizontal x direction, their mass shall be transferred to the lateral columns with the same amplification, based on the constraint degree of the corresponding support.

5.2.3 Rocking systems

The vertical panel of Figure 4 keeps its stability in its plane until the horizontal top force, H_o [see Formula (3)], is smaller than the limit force

$$H_o = \frac{Gb}{2h} \tag{3}$$

where

G is the weight of the panel;

b and h are geometrical quantities indicated in Figure 4.

If $H > H_0$, the panel starts rocking around its lower corner like an inverted pendulum with a restoring force, H_0, that remains constant for small displacements. At the reverse motion, the panel sits back on the base side and starts a new opposite cycle similar to the previous one. To capture such vibration motion, a refined dynamic analysis should be carried out for the solution of the non-linear algorithms inclusive of the unilateral effects of the base supports.

Considering this, the small value of the limit force, H_0, can prevent the rocking motion only for low actions. For practical design applications, a simplified approach can be used, based on a linear elastic structural analysis for each of the two possible structural schemes: integrated and isostatic. The design approach can therefore adopt a first model corresponding to the integrated system with cantilever panels fully fixed at their base and connected with an equivalent hinge to the roof and a second model referred to the isostatic system with pendulum panels connected with two end hinges.

Starting with the integrated system, the first analysis refers to the serviceability limit state (SLS[2]) seismic action, evaluated using the pertinent elastic response spectrum. Its outcome provides the forces and displacements. If the corresponding connection forces are not greater than H_0, the calculated displacements are used for the verification of the drift limits. If they are greater, the analysis of the isostatic model is necessary.

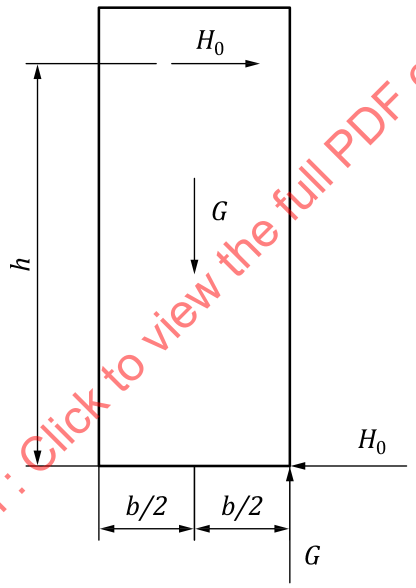


Figure 4 — Vertical panel force equilibrium

The second analysis of the integrated system refers to the ultimate limit state seismic action (no-collapse requirement), evaluated using the pertinent design response spectrum with the behaviour factor $q = 1{,}0$. Its outcome provides the forces and displacements. If the corresponding connection forces are not greater than H_o, the forces calculated in the structure are used for the strength design. If they are greater, the analysis of the isostatic model is necessary.

When necessary, the isostatic system is analysed neglecting the restoring force, H_0. The panels only contribute to the response of the structure as masses without any stiffness. Thus, they can be modelled as indicated in 5.2.2. For SLS, the elastic response spectrum is used. The resulting displacements are verified against the required drift limits. For the ULS, the design response spectrum is used. The behaviour factor q, of the frame systems and the resulting forces are used in the strength designs.

In any case, the forces in the panel connections for their strength design, taking into account the impulsive effects of the dynamic action, shall be taken at least equal to $2H_0$.

2) SLS: state at which the structure is supposed to be comfortable and usable taking into account vibrations, deflections and cracks.

5.3 Analysis of conventional systems

5.3.1 General aspects

The term “conventional systems” used in these rules is for the reinforced concrete buildings with the existing fastening systems of cladding panels, which have been extensively used in the past and may still be used at present in zones with low to medium seismicity.

The existing design practice for the conventional systems usually has been based on the model that is not explicitly considering the interaction between the main structural system and the claddings in the plane of the façade. Such approach cannot identify eventual complex interaction between the structure and the panels leading to possible failure of the fastening system and to the fall of the panels during strong earthquakes. Some of such systems, in case of small seismic demand and/or structures with large over-strength and stiffness, can still provide sufficiently safe design solutions.

A suitable design procedure is provided in 5.3.2 and 5.3.3. It can be used for strengthening existing structures as well as for the design of new ones.

5.3.2 General design methodology

These rules are strictly limited to those fastening systems described in Clause 6. When the applied fastening system is different from those presented in Clause 6, the system shall be experimentally and analytically investigated (taking into account the 3D behaviour of the structure) to provide the basic data needed in the proposed methodology. These data include, but are not limited to, the mechanism of the structure-to-panel interaction, deformation and strength capacity, equivalent stiffness, and, in the case when refined inelastic response analysis is chosen, the hysteretic models for the structure and the fastening system.

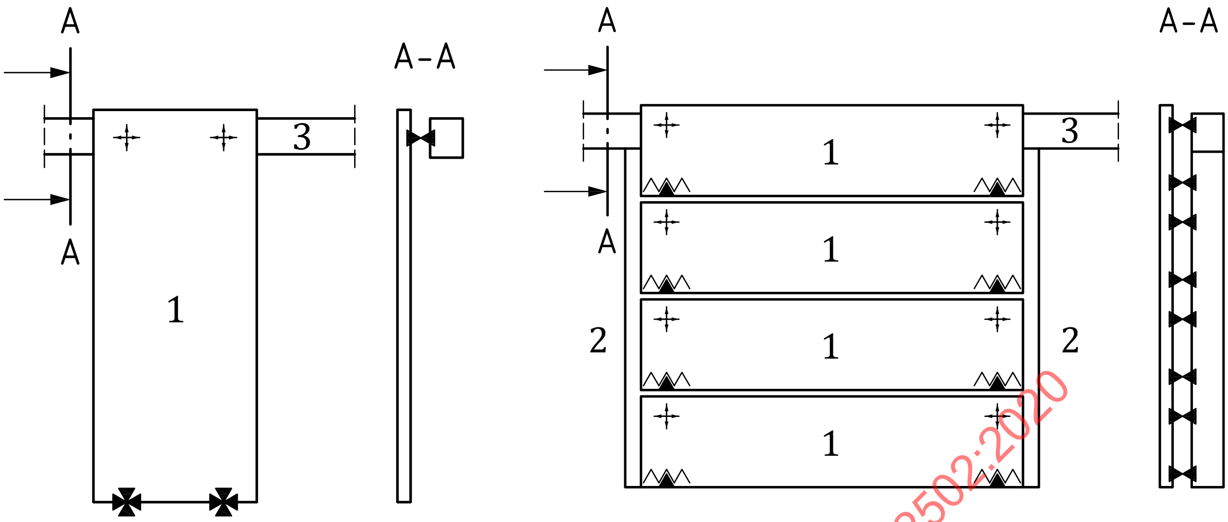
Furthermore, these rules are limited to fastening schemes presented in Figure 5 a) for vertical panels and Figure 5 b) for horizontal panels. In particular, the vertical panels are attached to the upper beam with two connections giving bilateral restraint in y (orthogonal) direction and bilateral essentially sliding freedoms in x (horizontal) and z (vertical) directions, while at the base they are supported with two pinned connections providing restraints in all the three directions. Any horizontal panel is attached to the lateral columns with two connections at the upper part and with two connections at the lower part giving bilateral restraint in y (orthogonal) direction, unilateral support in z (vertical) direction and bilateral partially sliding freedom in x (horizontal) direction.

The approach has two possible levels of complexity and it is based on the following main considerations:

a) weak interaction between the panel and the bare frame (i.e. the stiffness of the fastening devices is small compared to the stiffness of the structure itself) can be expected in conventional systems until certain deformation threshold is exhausted. Until this deformation limit is reached, the system behaves essentially as isostatic and relatively simple traditional structural models can be used, neglecting the structure-to-cladding interaction. The relevant deformation capacity of the addressed conventional systems is provided in 6.3;

b) after the deformation limit is reached, more complex model shall be used considering the interaction between the panels and the bare structure through the fastening system. Relevant input parameters for the addressed conventional systems are provided in 6.3;

c) if the more refined model does not prove the adequacy of the system, a different cladding connection system shall be chosen.



a) Vertical configuration

b) Horizontal configuration

Key
1 panel
2 column
3 beam

Figure 5 — Cladding elements fastening schemes for conventional systems

Design of the connections in the direction perpendicular to the plane shall always be done.

The most critical problem in the case of the conventional systems is their quite limited deformation capacity. Below this limit, the conventional connections behave essentially as isostatic connections.

Since the deformation threshold and the interacting mechanism are highly uncertain, back-up devices, such as restrainers shall always be used (see 6.4.1).

5.3.3 Application procedure

5.3.3.1 Practical application

5.3.3.1.1 General

The practical design application can be made through steps 1 to 4.

Step 1:

The bare frame (without panels) is analysed taking into account the local seismicity at the site. The modal response spectrum analysis is used.

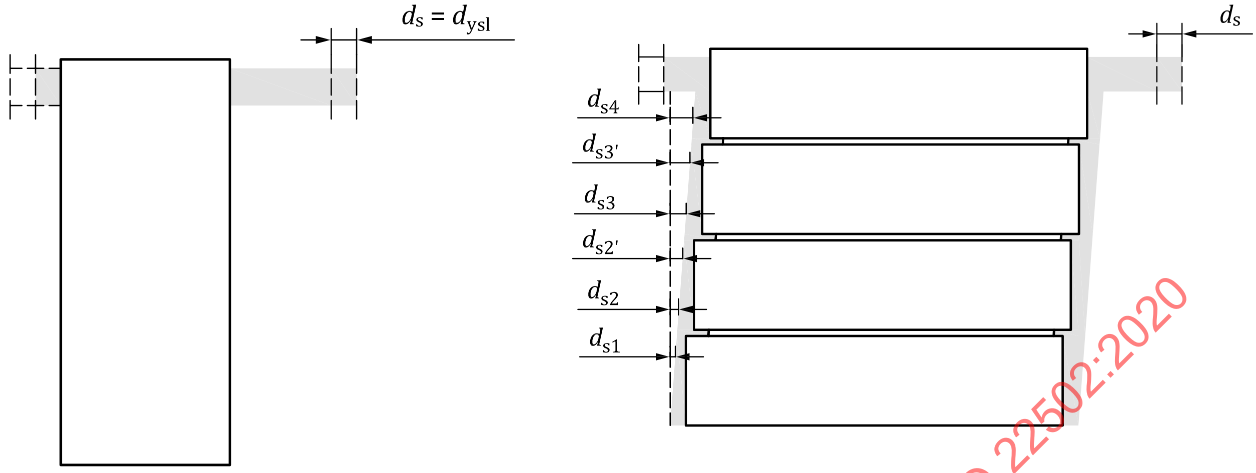
Estimation of the required strength of the structure can be based on the un-cracked cross-section of columns. For calculating the displacements, the cracked cross-sections shall be considered.

Step 2:

The displacement capacity of the connections in the plane of the panel is compared with the displacement demand defined in Step 1 as it is stated below:

a) vertical panels: the displacement capacity of the top connections is compared with the displacement of the beam [see Figure 6 a)]. The size of the gap, corresponding to the displacement demand shall be checked according to the procedure presented in 6.3;

b) horizontal panels: the displacement capacity of the connections is compared with the displacements of the columns at the level of these connections [Figure 6 b)].



a) Vertical configuration

b) Horizontal configuration

Figure 6 — Displacement demand of cladding elements

If the displacement capacity of vertical panel connections is larger than the displacement demand, the analysis is completed unless the gap between the beam and the panel is closed. If the displacement capacity of horizontal panel connections is larger than the displacement demand, the analysis of this step is completed. In such cases, it can be assumed that the interaction between panels and the bare frame is weak.

If the displacement demand exceeds the capacity of the connections and/or the gap between the vertical panels and the beam is closed, the cladding-to-structure interaction shall be considered in the analysis (see Step 4) or different cladding system shall be chosen.

Step 3:

In the direction perpendicular to the panel, the strength of all connections shall be verified with respect to the demand, evaluated according to normal rules for non-structural elements. The capacity of the connections in the direction perpendicular to the panel is estimated according to the data provided by the manufacturer. If the strength of the connections is inadequate, a different connection system shall be chosen.

Step 4:

The cladding-to-structure interaction is taken into account by means of a more refined structural analysis. Rules regarding this analysis are provided in 5.3.3.4.

If the displacement demand exceeds the capacity of the connection and/or the gap between the vertical panel and the beam is closed, a different connection system should be chosen. If not, go to Step 3.

5.3.3.2 Suggestions for the structural model

Any type of structural model and analysis can be used for the evaluation of the existing reinforced concrete buildings. While the equivalent elastic modal spectrum analysis is the first choice in the conventional design practice, more refined inelastic analysis method can be adopted in some cases due to the highly complex non-linear behaviour considering panel-to-structure interaction.

In the general case of existing reinforced concrete structures, the frame or dual wall-frame analysis should be performed by means of software computation. The ordinary methods for the formulation of the calculation model is applied with the specific pertinent indications given in 5.3 for conventional systems, 5.2 for isostatic systems or in 7.1 for integrated systems of connections.

5.3.3.3 Conditions for strengthening interventions

Any intervention of upgrading or retrofitting of panel connections on existing buildings shall be made only when the adequacy of all the remaining parts of the structure has been verified to be compliant with the requirements of the chosen level of seismic resistance.

When the proposed analysis procedure is used for existing buildings the following specifics shall be considered:

— the analysis and design shall follow the general requirements for the design of the main structural system;

— in Step 1 the adequacy of the main structural system (bare frame) shall be checked first before proceeding to Step 2. The main structural system itself may need upgrading first.

5.3.3.4 Refined analysis model

Numerical models of the connections shall be able to describe all important features of their seismic response. For conventional fastening systems, the appropriate hysteretic models presented in 6.3 can be used. Such models can imply the nonlinear dynamic analysis. For regular structures, the analysis can be simplified using the nonlinear pushover-based analysis.

The quoted numerical models of connections can be further simplified. An equivalent linear model [see Figure 7 a)], considering an increased effective damping ξ_{eff} [see Formula (4)] (hysteretic damping is taken into account), can be used. The damping can be estimated as:

$$\xi_{eff} = \frac{1}{2\pi}\left[\frac{\Sigma E_{D,i}}{K_{eff} d_{cd}^2}\right] \tag{4}$$

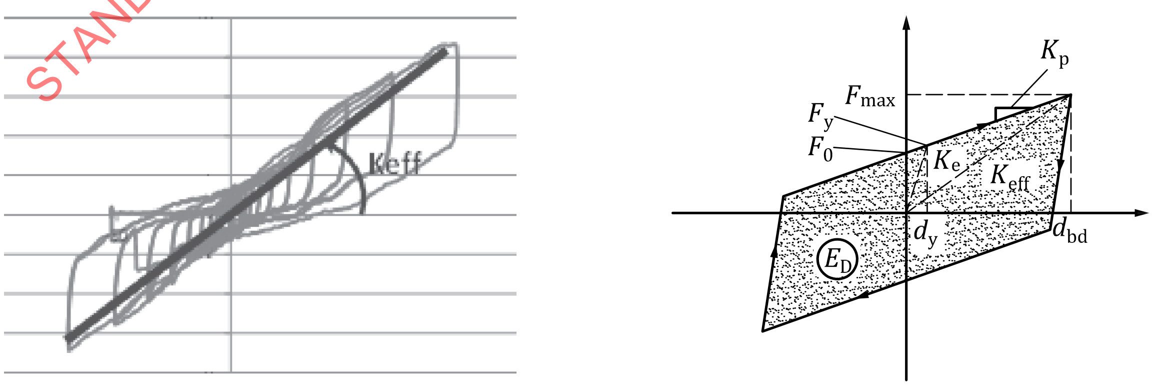
where

E_D is the dissipated energy;

K_{eff} is the effective stiffness;

d_{cd} is the maximum displacement.

The variables used in Formula (4) are illustrated in Figure 7.



a) Equivalent linear model **b) Parameters for calculation of effective damping**

Figure 7 — Numerical models of connections

5.4 Design of isostatic system connections

5.4.1 General

In order to ensure the pure frame behaviour of the resisting structure, the connection system of the cladding panels shall actually allow without reaction the large displacements of the frame structure under seismic action, except for possible minor unintended reaction effects due to friction or sealing. In this case, one shall adopt sliding connection devices with adequate capacities or pinned connectors free for rotations.

5.4.2 Structural arrangements

5.4.2.1 Vertical structural arrangements

Isostatic, vertical arrangement configurations, adequate for all seismic levels shall be easy to apply and shall follow one of the described solutions:

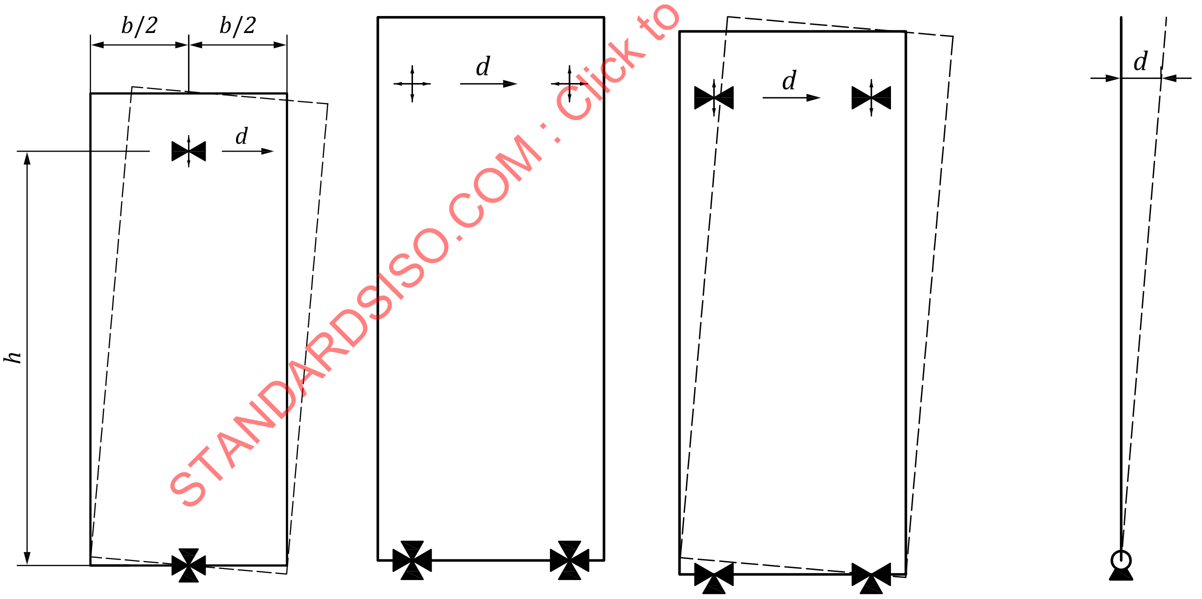
— pendulum solution presented in Figure 8 a) with arrangement matrix given in Table 3;

— cantilever solution presented in Figure 8 b) with arrangement matrix given in Table 4;

— rocking solution presented in Figure 8 c) with arrangement matrix given in Table 5.

For all three solutions, in the out-of-plane direction the panels are supported with an isostatic pendulum scheme Figure 8 d).

Considering that the vertical panels are placed over the bottom beam, to which they transmit their weight, and are supported horizontally by the upper beam with connections placed close to the top, the arrangements to obtain an isostatic connection system for these panels are those shown in Figure 8.



a) Pendulum behaviour   b) Cantilever behaviour   c) Rocking behaviour   d) Out-of-plane behaviour

Figure 8 — Vertical isostatic configurations

The first solution adopts hinged lower and upper supports so to have a pendulum behaviour for any single panel.

In the pendulum arrangement for a given top displacement d, the adjacent vertical sides of the panels display a relative slide of bd/h

where

h is the height of the upper support;

b is the width of the panels.

The two adjacent sides get closer by a minor quantity that requires in any case a sufficient free spacing between the panels, closed by the sealant. The sealing between the adjacent panels may give a minor reaction to the motion that can be neglected. The rotating base supports of the panels, in arrangements that allow it, may display a friction reaction to the motion that has very small (negligible) dissipative effects on the seismic response. Since only vertical compression forces are expected at the base supports, these can consist of simple seating able to provide only a unilateral restraint in −z direction. To allow for thermal expansion, the upper supports may be made of one central or two lateral vertical slide channel bars able to transmit only horizontal forces.

Table 3 — Arrangement matrix - pendulum behaviour

Direction	A	B	C	D	E	F
x	f	/	f	/	/	/
y	f	/	f	/	/	/
z	f	/	s	/	/	/

The second solution adopts fixed (pinned) supports at the base of the panels and one or two sliding connections to the structure at the upper position so to have a cantilever behaviour uncoupled from the structure.

The cantilever arrangement [see Figure 8 b)] keeps the panels still during the motion of the structure because of the horizontal slide channel bars placed at the upper position. To allow for thermal expansion, vertical channel bars may be coupled to the horizontal ones (one in the beam and one in the panel). Sensible friction effects may arise due to the contemporary orthogonal forces caused by the biaxial vibratory motion. The base support of the panels can be provided with reinforcing bars protruding from the bottom and anchored by bond within corrugated sleeves inserted in the foundation and filled with no-shrinking mortar. Other types of dry mechanical connections may be adopted for the base support of the panels.

Table 4 — Arrangement matrix – sliding frame (cantilever)

Direction	A	B	C	D	E	F
x	f	f	s	s	/	/
y	f	f	f	f	/	/
z	f	f	s	s	/	/

The third solution adopts a simple seating of the panels on bearings placed at the two edges of the base side together with a hinged connection to the structure at the upper position so as to have a rocking behaviour for large displacements.

The rocking arrangement consists of the seating of the panels on the foundation through unilateral bearings that work only in compression. The two edges of the base side of the panel alternatively rise up during the rocking motion, hence should be properly reinforced against spalling. Small horizontal actions applied to the upper connection of any single panel are equilibrated by its weight until Formula (5) is satisfied.

$$H \leq Gb/(2h) \tag{5}$$

where

G	is the weight of the panel;
b	is its width;
h	is the height of the applied horizontal action.

In this condition, the panels behave as integrated in the structure that becomes a dual wall-frame system with a much higher global stiffness. Under seismic action, therefore, the reacting system receives an initial high horizontal impulsive force that decreases when that limit is overcome and the panels begin to rock, behaving as an isostatic system. In the rotated position the panel, seated in its edge active bearing, provides a stabilizing constant horizontal force expressed in Formula (6)

$$H = Gb/(2h) \tag{6}$$

At the reverse motion, the panel seats back again on the two base lateral bearings restoring its initial stable equilibrium. The analysis of such vibration motion requires calculation codes with refined algorithms for the solution of the nonlinear equations. A simplified approach for the structural analysis is given in 5.2.3.

Table 5 — Arrangement matrix – rocking behaviour

Direction	A	B	C	D	E	F
x	f[a]	f*	f	f	/	/
y	f	f	f	f	/	/
z	f-	f-	s	s	/	/

[a] Working alternatively.

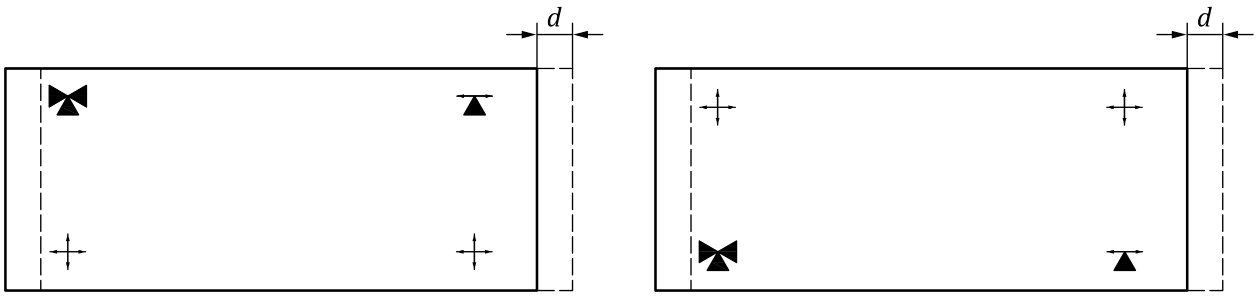
5.4.2.2 Horizontal structural arrangements

The horizontal panels are connected externally to the adjacent columns to which they transmit their weight, being restrained horizontally by the same connections. The lower panels can be seated directly with their weight on the foundation elements.

Isostatic horizontal arrangement configurations, adapted for all seismic levels, shall be easy to apply and shall follow some of the described solutions.

— Hanging solution presented in Figure 9 a) with arrangement matrix given in Table 6.

— Seated solution presented in Figure 9 b) with arrangement matrix given in Table 7. Following an alternative solution, the horizontal panels can be seated one over the other, taking all their weights directly to the foundation and transmitting to the adjacent columns only the horizontal orthogonal actions due to their mass. In this case, there is a lower reliability of the model because of the higher uncertainties of the friction longitudinal behaviour of the mutual joints under seismic conditions. For these reasons, this alternative solution is not recommended.



a) Hanging   b) Seated

Figure 9 — Horizontal isostatic configurations

The panels shall have a free spacing at the joint between the adjacent sides to allow the relative slide motion without friction. This joint is sealed with proper material that may introduce a minor reaction effect.

Any single panel is provided by two upper or lower vertical supports placed at the ends and fixed to the columns. One of them provides also the horizontal restraint in the plane of the panel. To allow for thermal expansion, the opposite one gives no horizontal reaction in the plane of the panel. At the opposite lower or upper side, two couples of sliding connections are placed allowing the free horizontal and vertical displacements. All four corner connections provide a fixed horizontal support orthogonal to the panel.

Table 6 — Arrangement matrix - hanging horizontal panel

Direction	A	B	C	D	E	F
x	s	s	f	s	/	/
y	f	f	f	f	/	/
z	s	s	f-	f-	/	/

Table 7 — Arrangement matrix - seated horizontal panel

Direction	A	B	C	D	E	F
x	f	s	s	s	/	/
y	f	f	f	f	/	/
z	f-	f-	s	s	/	/

5.4.3 Sliding devices

The sliding devices, such as fixed channel bars with internal moving head slides, shall allow free longitudinal alternate displacements without reactions. Tangling effects may result in unexpected integrated behaviour: they shall be avoided in all cases. The slides shall be proportioned with the due clearance with respect to the channel without sharp corners and shaped in such a way to avoid seizure especially under impulsive actions. Steel-to-steel contact shall be avoided and special inner coating of the channel bar shall be provided with PTFE polymer or other slippery materials.

An initial type testing shall be performed submitting some devices to monotonic and cyclic longitudinal actions for different values of the transverse force (3D effects) consistently with the joint arrangement expected in the structure and with the geometrical allowances proportioned considering also the production and installation tolerances in the overall façade. The proper functioning shall be checked all over the range of the expected displacements.

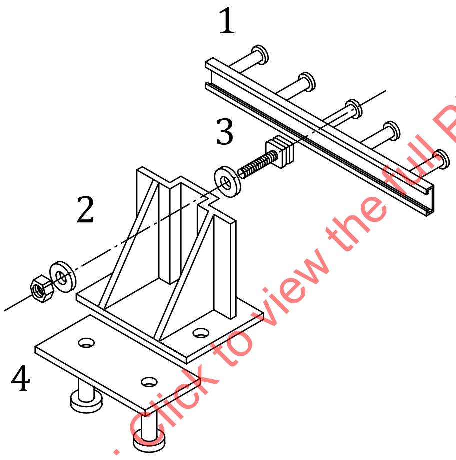
Particular care shall be given to the installation of the sliding devices in order to keep the positioning deviations within the admissible linear and angular tolerances of the joint.

The sliding devices of concern shall provide an effective connection between panel and structure in the direction orthogonal to the panel, without excessive out of plane plays. In the overall arrangement of the panels, all possible hammering phenomena between adjacent panels and panel to structure shall be prevented.

Specific checks of the geometrical compatibility of the displacements shall be addressed to the corners of the building where orthogonal panels are jointed, with possible interposed angle elements.

Figure 10 shows a type of sliding device for connecting a vertical panel to the upper beam. The channel bar short or long (1) is fixed to the beam. A fixing gear (2) is attached to the panel in a special pocket, bolted to a counter-plate (4). A fastener (3) is placed to connect the two parts, where its grooved head slide is inserted in the lips of the channel bar to provide a bilateral transverse support while it remains free to slide in the longitudinal direction. At the other end, the fastener is attached to the fixing gear with a full support that restrains displacements and rotations.

The sliding length of the channel bar is dimensioned with reference to the expected drift of the floor under seismic conditions amplified by a factor of 1,2 to cover the uncertainties of the calculation. The resistance design of the connection is made with reference to the transverse force based on the mass of the panel.



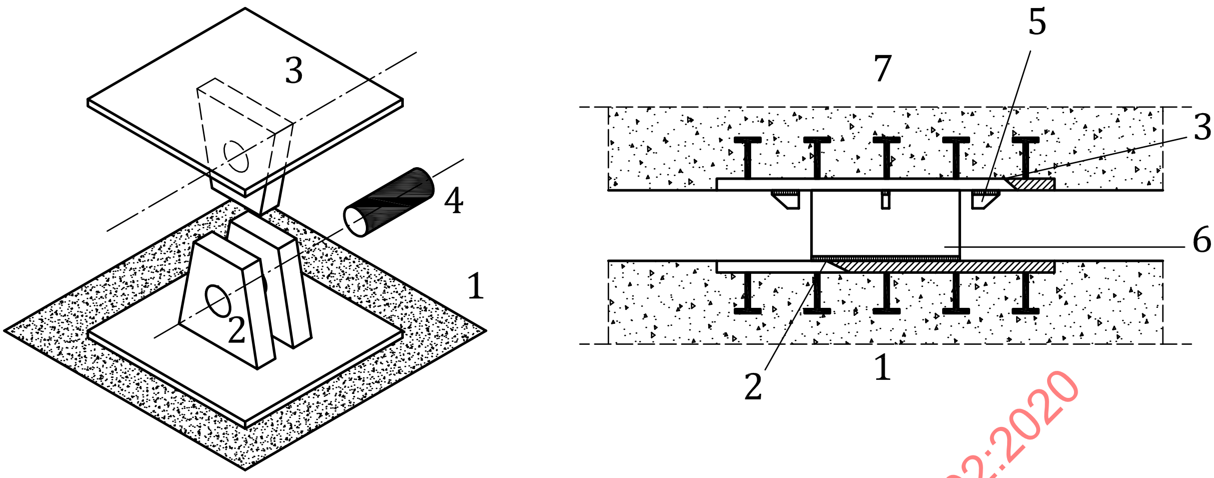
Key

1 channel bar
2 fixing gear
3 fastener
4 counter plate

Figure 10 — Sliding device connection

5.4.4 Hinge connections

Rotating connections can be used at the base of vertical panels in pendulum arrangement. Figure 11 a) shows the mechanical device that can be used when a resultant vertical upward (lifting) action is expected on the panel so to require a downward reaction from the connection. It is the case of the end panels of a cladding, subjected to a longitudinal horizontal alternate force, where the panels are connected to each other by lateral joint connectors (possibly dissipative). The quoted one is an ordinary hinge, common in steel construction, which relays its resistance on the shear strength of the pin and on the bearing action of the lateral plates. If no upward actions are expected on the panel, a simple support on a steel pad that concentrates the compressions in a small print can be used [see Figure 11 b)]. The plastic settlements of the pad allows the small relative rotations of the bearing contact surfaces. Proper lateral restraints (stoppers) shall be provided for the shear action.



a) Hinge connection **b) Simple steel pad support**

Key

1	foundation beam	3	upper part	5	brackets	7	panel
2	bottom part	4	pin	6	steel/laminated rubber pad		

Figure 11 — Vertical supports

5.4.5 Supports with steel brackets

Strong steel brackets are conceived to support the horizontal suspended panels, connecting them to the columns. They are made with traditional steel profiles that may be filled with concrete to increase their stiffness. Bracket connections are conceived to carry the gravity load of the panel, and are usually placed in two positions in a row. A recess in the panel or/and an inclined contact surface may prevent the panel out-of-plane displacement. Even if they do not usually allow the panel in-plane displacement, special features may be adopted to make them slide horizontally for thermal expansion. The efficiency of the sliding mechanism shall be experimentally verified through monotonic and cyclic tests.

Figure 12 shows a type of bearing bracket device provided with an adjustable steel bolt for the support of horizontal panels, connected to the column. The bracket is inserted in a recess left in the column and the panel, which is provided with a hosting groove.

For the bearing capacity of the device, reference should be made to the technical specifications of the producer that are usually based on experimental testing. The out-of-plane resistance design of the connection is made with reference to the transverse force calculated according to normal rules for non-structural elements based on the mass of the panel.

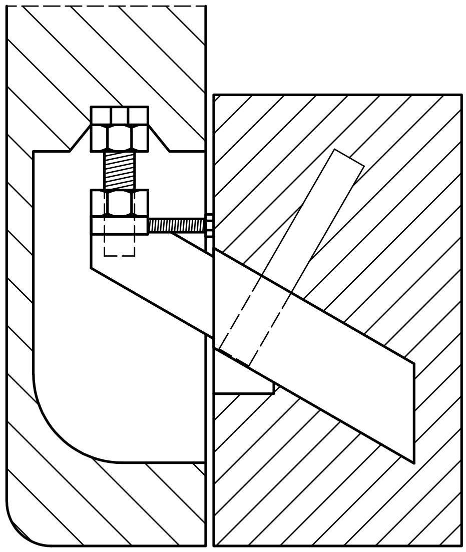


Figure 12 — Bearing bracket device

6 Design of conventional connections

6.1 General

For the seismic analysis, the old design practice of the reinforced concrete structures of concern was often based on a bare frame model where the peripheral cladding panels enter only as masses without any stiffness. The panels are then connected to the structure with fixed (pinned) fastenings dimensioned with a local calculation on the base of their single mass for anchorage forces orthogonal to the plane of the panels.

For the many existing buildings designed following this old practice, proper upgrading or retrofitting interventions shall be made. These interventions are needed also for the many existing buildings originally not designed for seismic actions and placed in areas that now, following the updated knowledge, are classified as seismic zones.

6.2 Structural arrangements

6.2.1 Vertical structural arrangements

Figure 13 shows the graphic scheme of a very common connection system used during many years for vertical panels. The unit is simply supported on the foundation beam without mechanical connectors. It is connected to the top beam through vertical channel bars and headed fasteners giving a bilateral out-of-plane y restraint and an unintended "weak" bilateral restraint with limited movement allowance in the horizontal x direction. Table 8 – gives the corresponding arrangement matrix.

Table 8 — Arrangement matrix – conventional vertical panels

Direction	A	B	C	D	E	F
x	f[a]	f[a]	f	f	/	/
y	f	f	f	f	/	/
z	f-	f-	s	s	/	/

[a] Restrain due to friction.

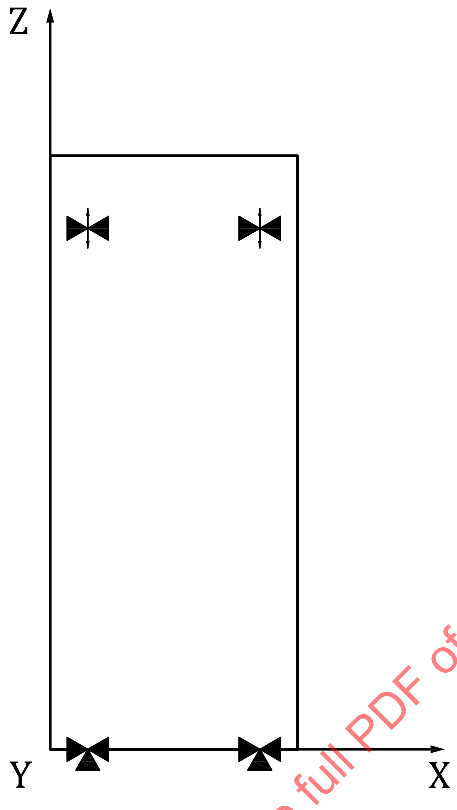


Figure 13 — Vertical structural arrangement

6.2.2 Horizontal structural arrangements

Figure 14 shows the graphic scheme of one of the connection systems used during many years for horizontal panels. The unit is connected to the two lateral columns. The two lower connections consist of steel corbels protruding from the columns proportioned to give the vertical -z support to the panel; the two upper connections consist of shear dowels that ensure the bilateral horizontal x restraint; all the four connections provide the bilateral horizontal y restraint. Table 9 gives the arrangement matrix.

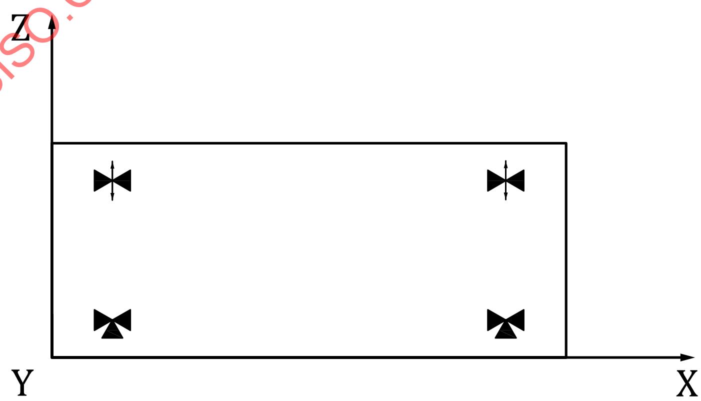


Figure 14 — Horizontal structural arrangement

Table 9 — Arrangement matrix – conventional horizontal panels

Direction	A	B	C	D	E	F
x	f[a]	f[a]	f	f	/	/
y	f	f	f	f	/	/
z	f-	f-	s	s	/	/
[a] Weak unintended restraint.						

6.3 Conventional fastening systems

6.3.1 General

The deformation capacity of the system and the interaction of all components in the structure shall be carefully verified.

The use of any other existing fastening types or the above connections with different characteristics than those described in 6.3.2 to 6.3.4 is not allowed unless adequate experimental and analytical studies do provide these data and verify the design methodology for the particular type.

The data in 6.3.2 to 6.3.4 are provided for the two levels of analyses foreseen in 5.3. For the simplified check of elastic displacements (Steps 1 and 2 in the procedure outlined in 5.3.3), only deformation capacity of the connection, based mainly on the geometric characteristics, is needed.

For the more refined analysis, (Step 4 in the procedure outlined in 5.3.3) a set of data is provided based on the experimental results. It includes strength and deformation capacity based on the cyclic envelope as well as hysteretic rules for inelastic response analysis.

6.3.2 Hammerhead strap connection

6.3.2.1 General

Hammerhead strap connection consists of two steel channels that are mounted in a beam (or column) and a panel and a hammerhead strap that is fastened to the channel mounted in the beam (or column) by means of a single bolt (see Figure 15). Hammerhead strap connections are most often used for the fastening of vertical panels to the beams.

Hammerhead strap connections can be provided with channels of different strength. Principally, two different types of channels should be recognized:

a) strong channel – when loaded in shear, the connection fails due to the failure of the strap (see Figure 16);

b) weak channel – when loaded in shear, the connection fails due to the failure of the channel (see Figure 17).

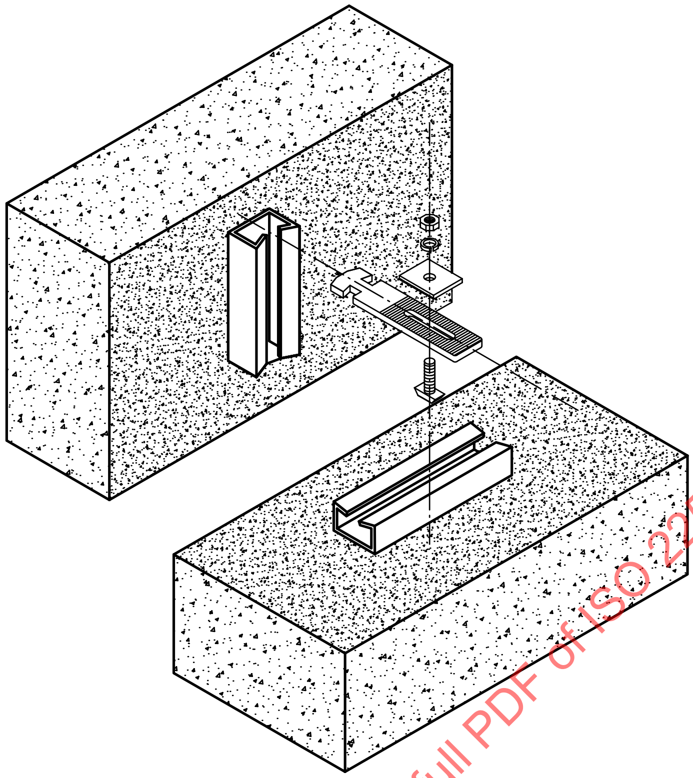


Figure 15 — Schematic presentation of the hammer-head strap connection assembly

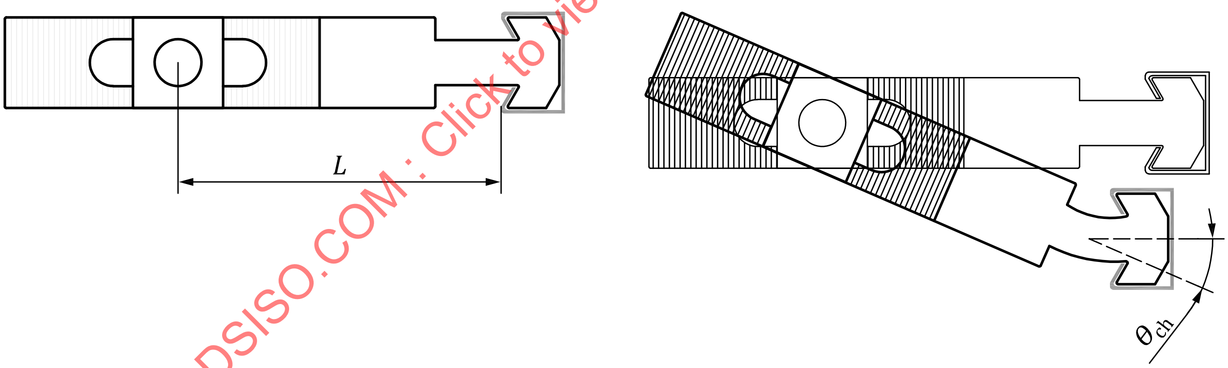


Figure 16 — Failure mechanism of the hammer-head strap connection with strong channels

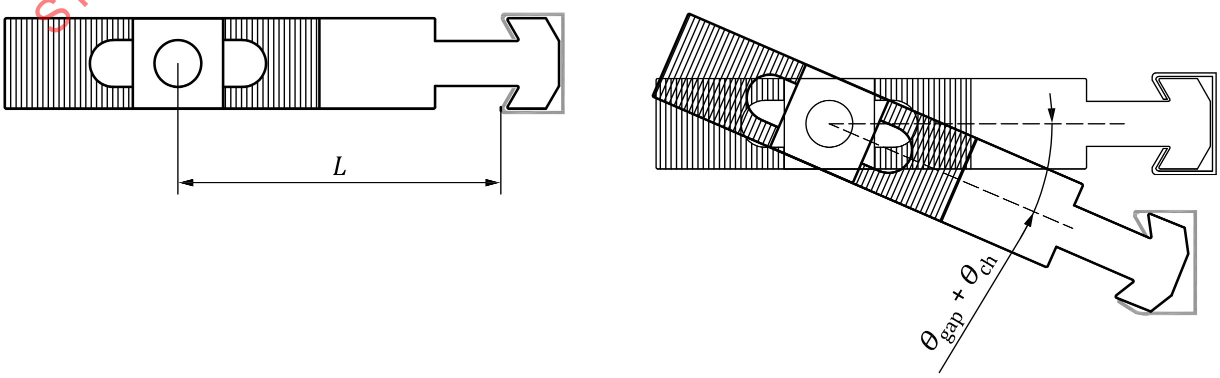


Figure 17 — Failure mechanism of the hammer-head strap connection with weak channels

For seismic loading, deformation capacity in shear and out of plane resistance of the hammerhead strap connections shall be checked.

In the direction perpendicular to the panel plane, the design forces in the panel-to-structure connections shall be calculated according to normal rules for non-structural elements. Out of plane resistance as well as shear deformation capacity of hammerhead strap connection shall be proved by experimental testing.

In the direction parallel to the plane of the panels, weak interaction between the panel and the bare frame may be expected until certain deformation threshold is exhausted. Until this deformation limit is reached, the system behaves essentially as isostatic and relatively simple structural models can be used, neglecting the structure-to-cladding interaction. The relevant design parameters for hammerhead connections are given in 6.3.2.2.

After the deformation limit has been reached, either a more complex model shall be used, considering the interaction between the panels and the bare structure through the fastening system, or a different cladding system shall be chosen. The relevant design parameters for hammerhead connections are given in 6.3.2.2.2.

6.3.2.2 Design based on the simplified model, neglecting panel-to-structure interaction

6.3.2.2.1 General

Two checks shall be done:

a) deformation capacity of the connection when loaded in shear shall be adequate;

b) closing of the gap between a beam and a panel shall be controlled.

6.3.2.2.2 Deformation capacity control

Deformation capacity of the hammerhead strap connection when loaded in shear depends on the geometry of the main components of connection and deformability of the strap or the channel, depending on the type of the failure mechanism (see Figure 15 and Figure 16). It shall be estimated in the following way.

a) If strong channels are used, the ultimate displacement of the connection when loaded in shear can be estimated by means of Formula (7):

$$d_u = \left(\theta_{gap} + \theta_{st}\right) L \tag{7}$$

where

θ_{gap} is the rotation of the strap due to allowances within the channel taking also into account the production and erection tolerances of the panel;

θ_{st} is the rotation of the strap due to the flexural deformations of the strap at the narrowing just under the head, which can be estimated as ultimate curvature multiplied by the length of the narrowing;

L is the distance between the bolt and the channel mounted in the panel (see Figure 16).

b) If weak channels are used, the ultimate displacement of the connection when loaded in shear can be estimated by means of Formula (8):

$$d_u = \left(\theta_{gap} + \theta_{ch}\right) L \tag{8}$$

where

θ_{gap} is the rotation of the strap due to the tolerances within the channel;

θ_{ch} is the rotation of the strap due to the deformations of the channel mounted in the panel (which shall be evaluated by Finite Element analysis or experimentally verified, in both cases under cyclic action);

L is the distance between the bolt and the channel mounted in the panel (see Figure 17).

6.3.2.2.3 Gap closure control

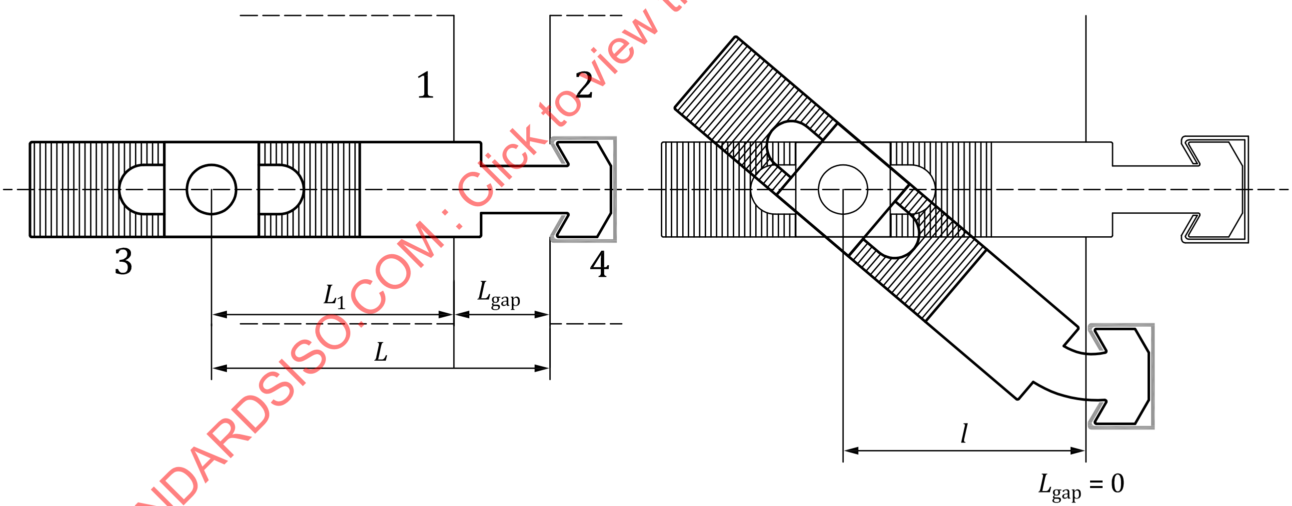
The minimal gap shall be equal to L_{gap}, calculated with Formula (9):

$$L_{gap} \geq L\left(1-\sqrt{1-\left(\frac{d_u}{L}\right)^2}\right), \tag{9}$$

where

L and L_{gap} are the distances marked in Figure 18;

d_u is defined in 6.3.2.2.2.



Key
1 beam
2 panel
3 hammerhead stripe
4 channel

Figure 18 — Closing of the gap between a panel and a beam

6.3.2.3 Design based on the refined model considering panel-to-structure interaction

6.3.2.3.1 General

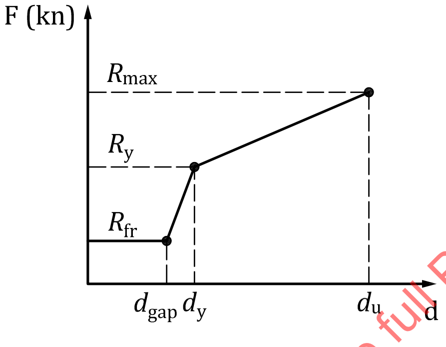
The design procedures and parameters shall be based on the experimental data for cyclic tests:

a) hysteretic response envelope given to define stiffness, deformation and strength properties;

b) hysteretic behaviour model.

6.3.2.3.2 Hysteretic response envelope

Hysteretic response envelope of the shear behaviour of hammerhead strap connections can be idealized as suggested in Figure 19. Three characteristic points are recognized.



Key

R_{fr} friction force
d_{gap} displacement in which hammerhead strap is stuck within the channel in panel
R_y force at yielding (of the hammerhead strap or the channel in plane)
d_y displacement at yielding
R_{max} strength
d_u displacement at failure

Figure 19 — Calibrated hysteretic model for hammerhead strap connections

The characteristic points of the force-displacement response envelope of a hammerhead strap connection with strong channels can be evaluated using Formulae (10) to (15):

$$R_{fr} = \frac{M_{fr}}{L} \tag{10}$$

$$R_y = \frac{\left[M_{y,N} + d_y\, P + M_{fr}\right]}{\sqrt{L^2 - d_y^2}} \tag{11}$$

$$R_{max} = \frac{\left[M_{pl,N} + d_u\, P + M_{fr}\right]}{\sqrt{L^2 - d_u^2}} \tag{12}$$

$$d_{gap} = \theta_{gap}\, L \tag{13}$$

$$d_y \approx d_{gap} \tag{14}$$

$$d_u = (\theta_{gap} + \theta_{st}) L, \tag{15}$$

where

M_{fr} is the moment in the bolt (it can be estimated as the tightening torque T_b);

L is the distance between the bolt and the channel mounted in the panel (see Figure 16);

$M_{y,N}$ is the flexural resistance of the hammerhead strap at the narrowing just under the head taking into the account axial force N;

$M_{pl,N}$ is the flexural resistance of the hammerhead strap at the narrowing just under the head taking into the account axial force N;

P is the force in the direction perpendicular to the panel plane;

θ_{gap} is the rotation of the strap due to the tolerances within the channel;

θ_{st} is the rotation of the strap due to the flexural deformations of the strap at the narrowing just under the head (see Figure 16), which can be estimated as ultimate curvature multiplied by the length of the narrowing.

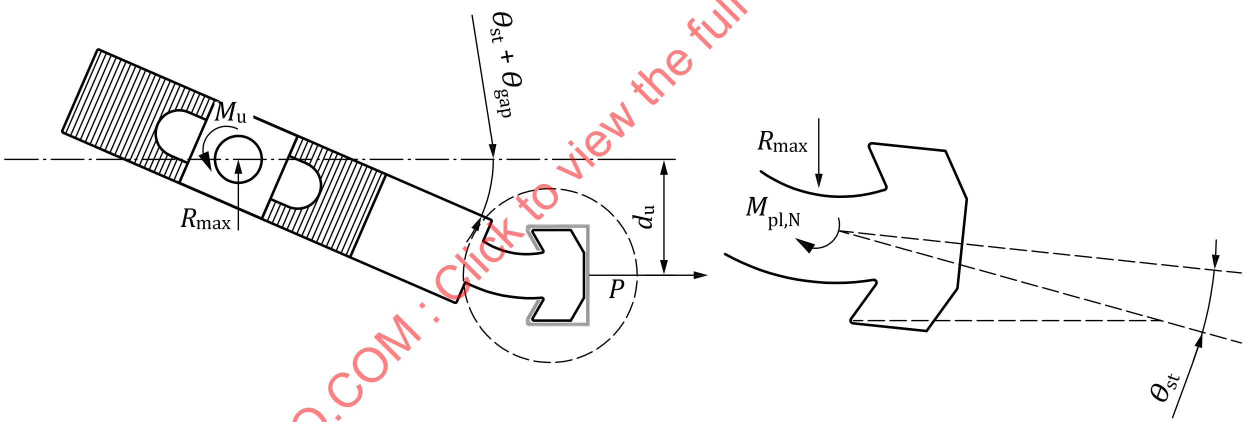


Figure 20 — Failure mechanism of hammerhead strap connections with strong channels

The characteristic points of the force-displacement response envelope of a hammerhead strap connection with weak channels can be evaluated using Formulae (16) to (21):

$$R_{fr} = \frac{M_{fr}}{L} \tag{16}$$

$$R_y = \frac{\left[\frac{1}{2} R \sqrt{1 - \left(\frac{d_y}{L}\right)^2} \left(R_{ch,y} - P\right) + d_y P + M_{fr}\right]}{\sqrt{L^2 - d_y^2}} \tag{17}$$

$$R_{max} = \frac{\left[\frac{1}{2} R \sqrt{1 - \left(\frac{d_u}{L}\right)^2} \left(R_{ch,u} - P\right) + d_u P + M_{fr}\right]}{\sqrt{L^2 - d_u^2}} \tag{18}$$

$$d_{gap} = \theta_{gap} L \tag{19}$$

$$d_y \approx d_{gap} \tag{20}$$

$$d_u = \left(\theta_{gap} + \theta_{ch}\right) L , \tag{21}$$

where

M_{fr}	is the moment in the bolt (can be estimated as the tightening torque T_b);
L	is the distance between the bolt and the channel mounted in the panel (see Figure 17);
$R_{ch,y}$	is the out of plane yield resistance of the channel (normally specified by the producer);
$R_{ch,u}$	is the out of plane resistance of the channel (normally provided by the producer);
R	is the distance marked in Figure 21;
P	is the force in the direction perpendicular to the panel plane;
θ_{gap}	is the rotation of the strap due to the tolerances within the channel;
θ_{ch}	is the rotation of the strap due to the deformations of the channel mounted in the panel (to be evaluated by FE analysis or experimentally verified).

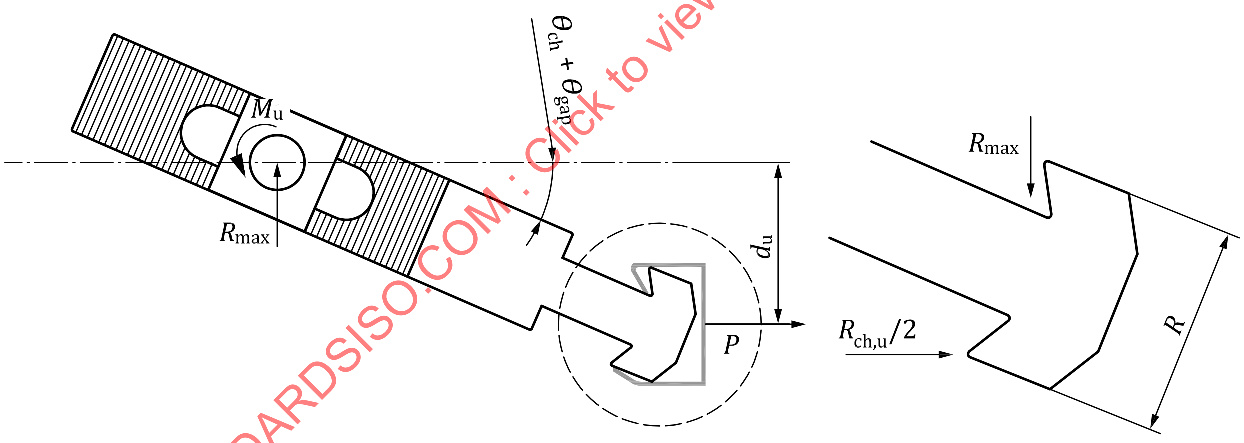
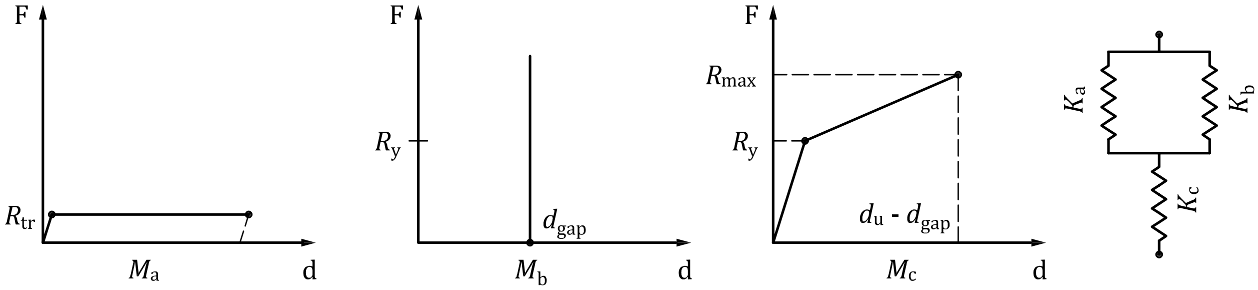


Figure 21 — Failure mechanism of hammerhead strap connections with weak channels

6.3.2.3.3 Hysteretic behaviour model

Hysteretic response of a hammerhead strap connection can be modelled by using a combination of three different responses: elasto-plastic, gap and hysteretic (see Figure 22).



Key

M_a elastic PP

M_b elastic PP gap

M_c hysteretic

Figure 22 — Combination of three different force-displacement responses

6.3.2.4 Other aspects

If panels are not anchored but only seated on the foundation beam, the relative displacements in the panel-structure connections are more difficult to model. Nonlinear dynamic analysis with appropriate models allowing for rocking behaviour should be performed.

6.3.3 Cantilever box connection

6.3.3.1 General

Cantilever box connections consist of a vertical channel, which is mounted in the columns and a special steel element that is mounted in the panel. The two components are then connected by means of a single bolt (see Figure 23).

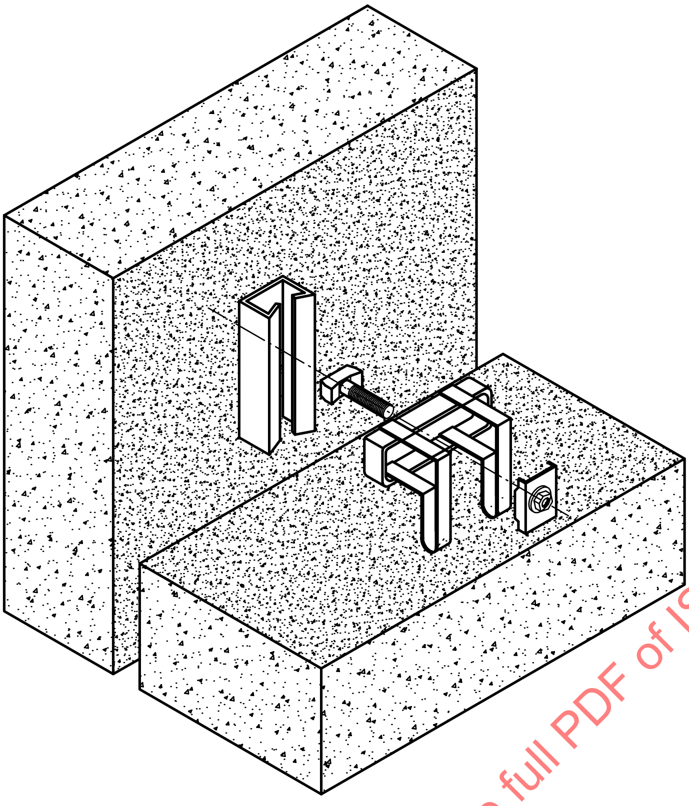


Figure 23 — Schematic presentation of the cantilever connection assembly

The behaviour of the cantilever connections, when loaded in shear, is schematically presented in Figure 24. At first the bolt slides along the profile. The resistance of the connection is equal to the friction force between the components. At some displacement level, the bolt reaches the end of the profile. At that point, the stiffness of the connection increases. Finally, the channel fails and the bolt is pulled out of the channel.

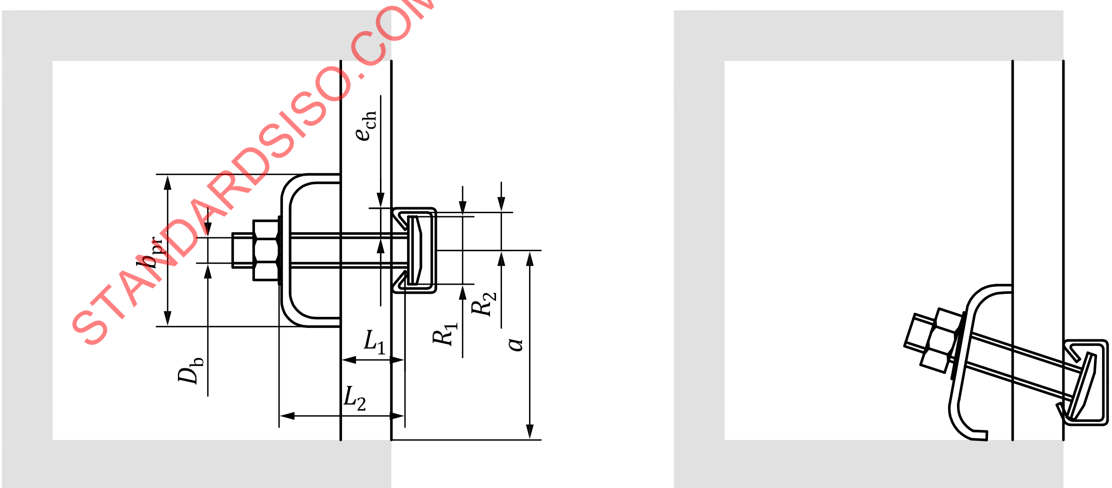


Figure 24 — Failure mechanism of the cantilever connection

6.3.3.2 Design

For seismic loading, the deformation capacity in shear and the out of plane resistance of the cantilever box connection shall be checked.

In the direction perpendicular to the panel plane, the design forces in the panel-to-structure connections shall be calculated according to normal rules for non-structural elements. The out of plane resistance as well as the shear deformation capacity of cantilever box connection shall be proved by experimental testing.

In the direction parallel to the plane of the panels, weak interaction between the panel and the bare frame may be expected until certain deformation threshold is exhausted. Until this deformation limit is reached, the system behaves essentially as isostatic and relatively simple structural models can be used, neglecting the structure-to-cladding interaction. The relevant design parameters for cantilever box connections are given in 6.3.3.3.

After the deformation limit is reached, either a more complex model shall be used, considering the interaction between the panels and the bare structure through the fastening system, or different cladding shall be chosen. The relevant design parameters for cantilever box connections are given in 6.3.3.4.

6.3.3.3 Design based on the simplified model, neglecting panel-to-structure interaction

The deformation capacity of the connection when loaded in shear shall be adequate. The deformation capacity of the cantilever box connection when loaded in shear depends on the geometry of the main components of the connection (Figure 24). It should be estimated with Formula (22):

$$d_{\text{gap}} = a - \frac{D_{\text{b}}}{2} \qquad (22)$$

where

D_{b} is the bolt diameter

A is the distance marked in Figure 24

6.3.3.4 Design based on the refined model considering panel-to-structure interaction

6.3.3.4.1 General

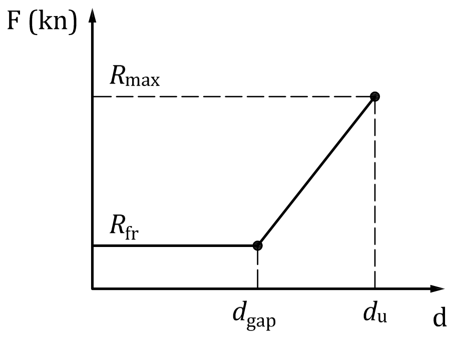
The design procedures and parameters shall be based on the experimental data for cyclic tests:

a) hysteretic response envelope given to define stiffness, deformation and strength properties;

b) hysteretic behaviour model.

6.3.3.4.2 Hysteretic response envelope

The hysteretic response envelope of the shear behaviour of cantilever connections can be idealized as suggested in Figure 25. Two characteristic points are recognized.



Key

R_{fr} friction force

d_{gap} displacement in which the bolt reaches the end of the steel box

R_{max} strength

d_u displacement at failure

Figure 25 — Hysteretic response envelope definition for cantilever connections

The characteristic points of the force-displacement response envelope of a cantilever connection with strong channels can be evaluated using Formulae (23) to (29):

$$R_{fr} = P_v k_{fr} \tag{23}$$

$$R_{max} = \frac{R_{ch,u}(r_1 + r_2)}{2l_1} \tag{24}$$

$$d_{gap} = a - \frac{D_b}{2} \tag{25}$$

$$d_u = a - \frac{D_b}{2} + e_{ch} \tag{26}$$

where

P_v is the axial force in the bolt due to the tightening torque;

k_{fr} is the friction coefficient between panel and beam;

$R_{ch,u}$ is the out of plane resistance of the channel;

D_b is the bolt diameter;

A is the distance marked in Figure 24;

e_{ch} is the distance marked in Figure 24.

$$r_1 = R_1\sqrt{1 - \frac{L_1(b_{pr} - D_b)}{2L_2}} \tag{27}$$

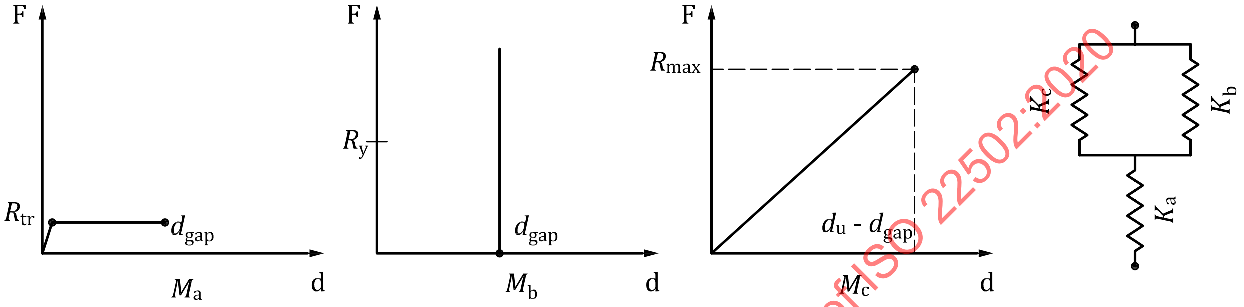
$$r_2 = \frac{L_1(b_{pr} - D_b)}{2L_2} + R_2 \tag{28}$$

$$l_1 = L_1\sqrt{1-\left(\frac{b_{pr}-D_b}{2L_2}\right)^2} \tag{29}$$

where R_1, R_2, b_{pr}, L_1 L_2 are marked in Figure 24

6.3.3.4.3 Hysteretic behaviour model

The hysteretic response of a cantilever box connection can be modelled by using a combination of three different responses: elasto-plastic, gap and elastic (see Figure 26).



Key

M_a elastic PP

M_b elastic PP gap

M_c elastic

Figure 26 — Combination of three different force-displacement responses

6.3.4 Steel angle connections

6.3.4.1 General

The steel angle connection consists of two steel channels that are mounted in a beam (or column) and a panel and a steel angle that is fastened to the channels by means of bolts (see Figure 27).

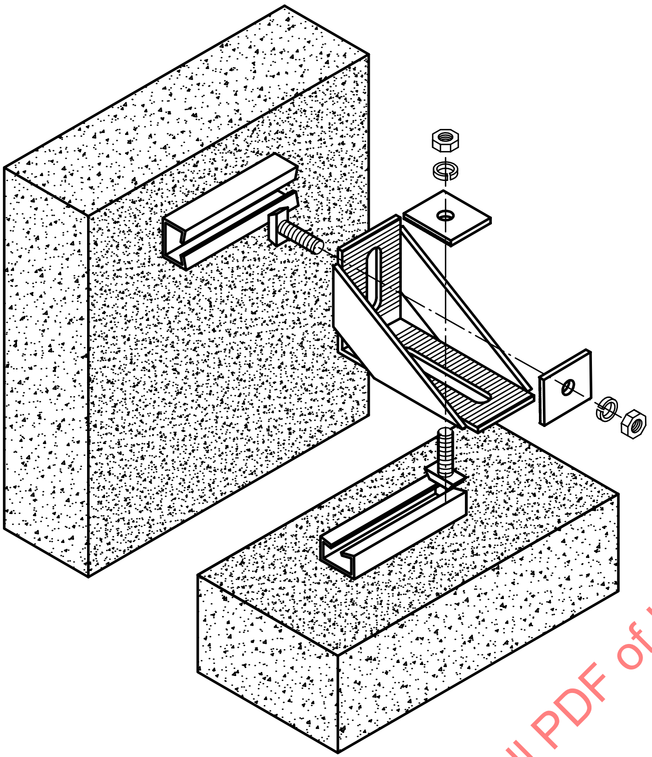


Figure 27 — Schematic presentation of the steel angle connection assembly

The behaviour of the steel angle connections, when loaded in shear, is schematically presented in Figure 28. Compression forces are induced at one edge of the angle and tension forces are induced in the bolt. The connection fails due to the failure of the channel mounted in the panel. The bolt is then pulled out of the channel.

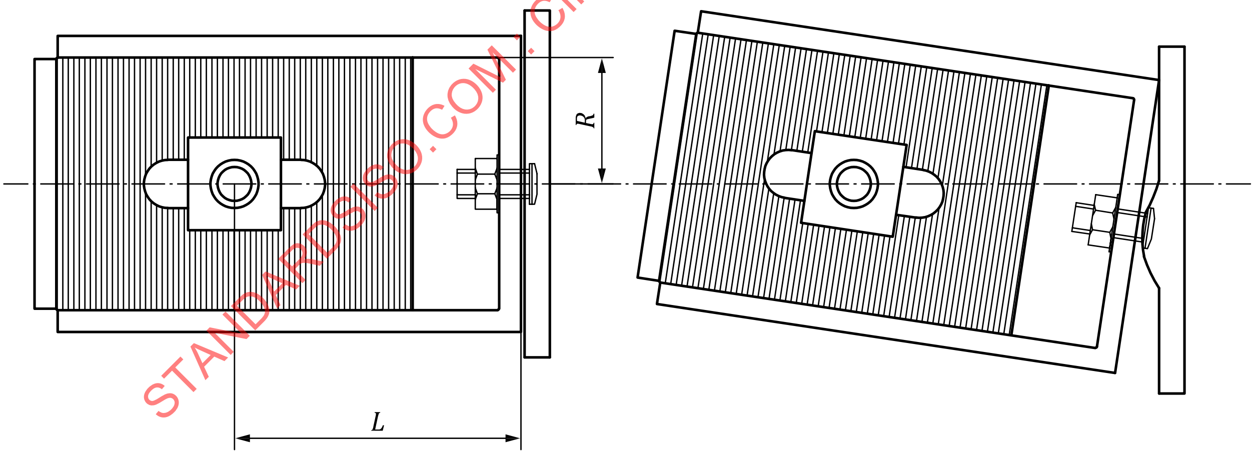


Figure 28 — Failure mechanism of steel angle connections

6.3.4.2 Design

For seismic loading, the shear and out of plane resistance of the connection shall be checked. The stiffness of the steel angle connection is not negligible and it should be taken into account in the global analysis of the structure. In the most simplified situation, the steel angle connection can be considered as a pinned joint.

In the direction perpendicular to the panel plane, the design forces in the panel-to-structure connections shall be calculated according to normal rules for non-structural elements. The out-of-plane resistance as well as the shear deformation capacity of cantilever box connection should be proved by experimental testing.

6.3.4.3 Design based on the refined model considering panel-to-structure interaction

6.3.4.3.1 General

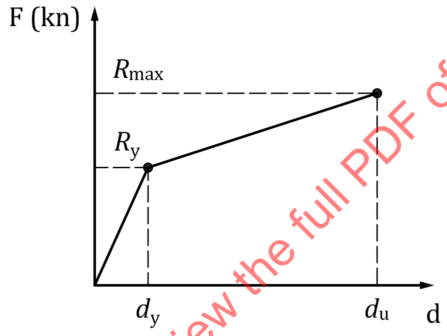
The design procedures and parameters shall be based on the experimental data from cyclic tests:

a) hysteretic response envelope given to define stiffness, deformation and strength properties;

b) hysteretic behaviour model.

6.3.4.3.2 Hysteretic response envelope

The hysteretic response envelope of the shear behaviour of steel angle connections can be idealized as suggested in Figure 29. Two characteristic points are recognized.



Key

R_y force at yielding (channel in the panel yields)

d_y displacement at yielding

R_{max} strength

d_u displacement at failure

Figure 29 — Hysteretic response envelope definition for steel angle connections

The characteristic points of the force-displacement response envelope of a steel angle connection with strong channels can be evaluated using Formulae (30) to (33):

$$R_y = \frac{\left[\frac{1}{2} R \sqrt{1-\left(\frac{d_y}{L}\right)^2} \left(R_{ch,y} - P\right) + d_y P + M_{fr}\right]}{\sqrt{L^2 - d_y^2}} \quad (30)$$

$$R_{max} = \frac{\left[R \sqrt{1-\left(\frac{d_u}{L}\right)^2} \left(R_{ch,u} - P\right) + d_u P + M_{fr}\right]}{\sqrt{L^2 - d_y^2}} \quad (31)$$

$$d_y = \sqrt{e_{ch,y}\left(2L - e_{ch,y}\right)} \quad (32)$$

$$d_u = \sqrt{e_{ch,u}\left(2L - e_{ch,u}\right)} \tag{33}$$

where

M_{fr} is the moment in the bolt (it can be estimated as the tightening torque T_b);

R is the distanced marked in Figure 28;

L is the distance between the bolt and the channel mounted in the panel (Figure 28);

P is the force in the direction perpendicular to the panel plane;

$R_{ch,y}$ is the out of plane yield resistance of the channel (normally specified by the producer);

$e_{ch,y}$ is the out of plane deformation of the channel at yielding (evaluated by FE analysis or experimental testing);

$e_{ch,u}$ is the out of plane deformation of the channel at failure (evaluated by FE analysis or experimental testing).

6.3.4.3.3 Hysteretic behaviour model

Hysteretic response of a steel angle connection can be modelled by using an appropriate hysteretic model.

6.4 Conventional strengthening and fastening systems

6.4.1 Second line back up devices

A not very invasive technique of intervention on existing buildings, to prevent the fall of cladding panels under earthquake conditions, consists in the insertion of short slack cables connecting the panel to the main structural element as shown for vertical panels in Figure 30. solution can be used only for a quick upgrading of existing buildings in order to ensure their operativeness for a (short) transitory period.

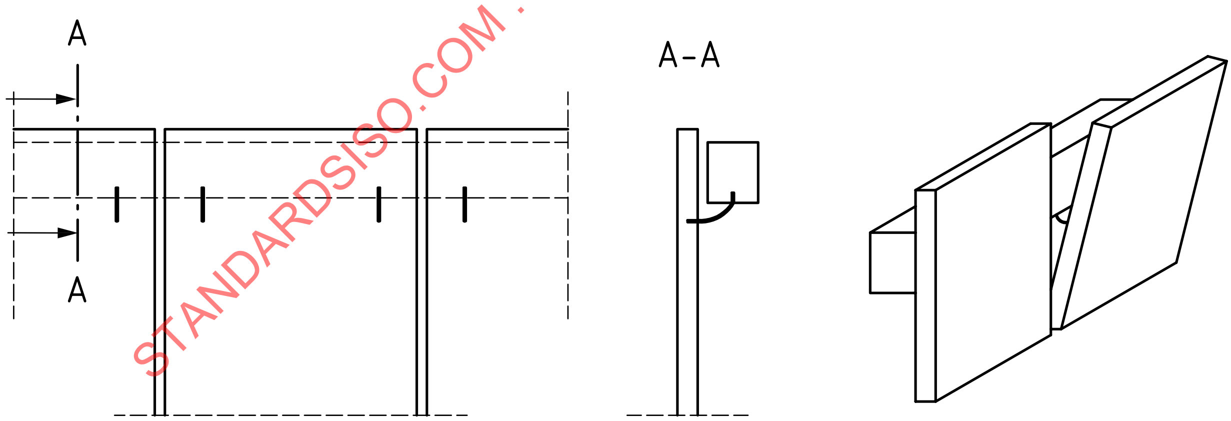


Figure 30 — Second line back up fasteners configuration 1

The seismic force demand in short restrainers can be estimated as the larger of the forces listed in Formulae (34) and (35):

$$f = \sqrt{\left(k\frac{v_0}{\omega}\right)^2 + \left(2mS_a\right)^2} \tag{34}$$

$$f = \sqrt{2}k\frac{v_0}{\omega} \tag{35}$$

where

Ω is the frequency of the restrainer;

K is the stiffness of the restrainer;

m is the tributary mass of the panel, supported by the restrainer;

S_a is the maximum acceleration of the bare frame structure;

v_0 is the maximum velocity of the bare-frame structure.

$$v_o = S_a / \Omega \tag{36}$$

where Ω is the frequency of the bare-frame structure.

Formulae (34) and (35) can be used for panel supported at their base, only if second order $P\Delta$ effects are negligible. The ratio s/H, where s is the length of the restrainer and H is the distance between the bottom of the panel and the restrainer, should be less than 0,1. For panels that can fall from higher positions, again for short restrainers, the conventional force should be referred to the double of their total weight.

For the proportioning of the end fastenings of the cable, a capacity design criterion should be applied, assuming a force given by Formula (37)

$$R_f = \gamma_R A_s f_{ym} \tag{37}$$

where

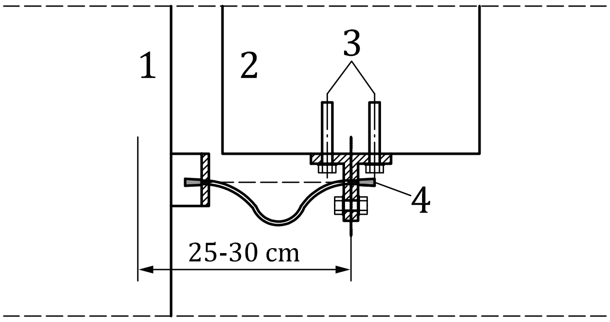
A_s is the sectional area of the cable;

f_{ym}=1,08f$_{yk}$ is its mean yield strength.

This force should not be greater than the pull-out resistance of the panel fastener at one end, and not greater than the shear resistance of the beam fastener at the other end. The values γ_R = 1,2 and γ_R = 1,35 can be used for medium or high ductility respectively.

The resistance design of the entire restraining device (cable plus end fastenings) can be performed also by testing a set up reproducing the local arrangement of the connection.

A possible alternative solution is illustrated in Figure 31. The system consists of wire or synthetic fibre rope and steel anchoring elements. It can be constructed with a wire or synthetic fibre rope terminated with a steel socket and anchoring elements. In the case of the failure of the existing panel-to-structure connections, the panel falls in the direction perpendicular to its plane, the rope is activated and the tension force is transmitted through the sockets into the steel anchoring elements and then into panel and beam.



Key

1 panel

2 beam

3 anchors

4 swaged or resign-potted end termination

Figure 31 — Second line back up fasteners configuration 2

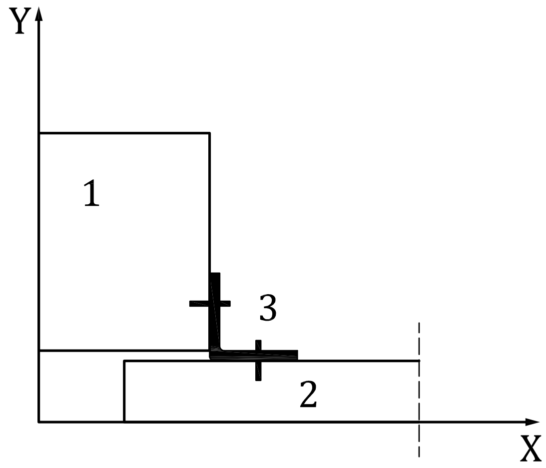
The stiffness, strength and deformation capacity of this restraining system should be determined experimentally.

6.4.2 Strengthening folded steel plates

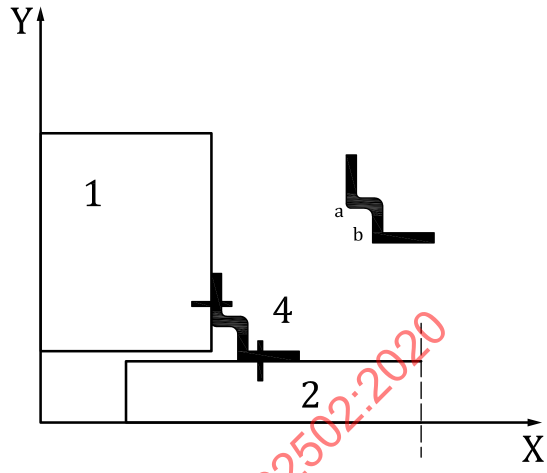
In addition to the original connectors, in order to strengthen the connections of horizontal panels to the supporting columns, steel angles can be used [see Figure 32 a)]. With reference to the longitudinal –x direction, this provision can strongly improve the resistance but without any ductile capacity, and can be justified only when the brittle failure is prevented by a proportioning of the connection verified with respect to the forces determined from the structural analysis of the dual wall-frame assembly.

Folded steel plates can be used, instead of steel angles, in existing buildings to retrofit or upgrade the connections of horizontal panels to the structure ensuring a good dissipative capacity through plastic cyclic deformations [see Figure 32 b)].

As for the common steel angles, the installation of the steel folded plates does not require any special care for tolerances since they can be shaped on site in the existing situation and attached to column and panel with post-installed drilled fasteners. Folded steel plates can also be effectively used in new constructions. For their design, see 8.3.6.



a) Regular steel angle plate



b) Patterned dissipative steel angle plate

Key

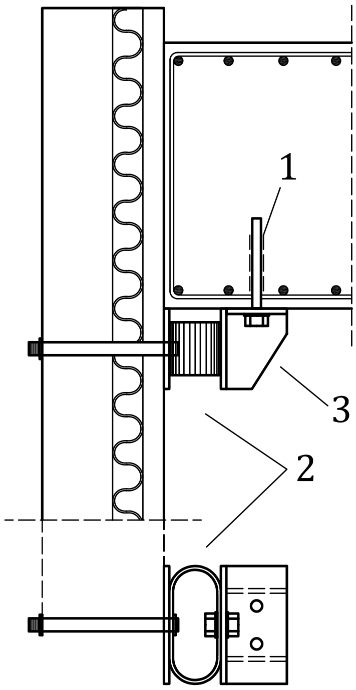
1 panel
2 column
3 steel angle plate
4 dissipative steel angle plate

Figure 32 — Folded steel plates

6.4.3 Strengthening with steel cushions

Steel cushions can be used in existing buildings to retrofit or upgrade the connections of vertical panels to the structure, ensuring a good dissipative capacity through plastic cyclic deformations (see Figure 33). The installation of the steel cushions does not require any special care for tolerances since they can be shaped on site in the existing situation and attached to beam and panel with post-installed drilled fasteners.

Steel cushions shall be proportioned to maintain an elastic behaviour up to the serviceability limit state (with small deformations) and possibly to suffer plastic deformations beyond this limit, with residual deformations after the earthquake and need for their replacement and for the adjustment the panel arrangement.



Key

1 epoxy

2 steel cushions

3 steel angle

Figure 33 — Steel cushions

For the behaviour, properties of steel cushions and the calculation models, see 8.3.5.

7 Integrated systems

7.1 General

For buildings with integrated arrangements of cladding panel connections, the structural analysis under seismic action shall refer to the dual wall-frame system that includes in the resisting structure columns, beams, floor elements and cladding panels with their connections.

7.2 Analysis of the buildings

7.2.1 General

In addition to the ordinary output data used for the design of member resistance at ultimate limit state, the forces in the panel-beam joints shall be provided for the design of the pertinent capacities of panel connection devices.

Clause 7 concerns rules for the design of panels with integrated connections. It is noted, however, that other types of panel connections can be used together with integrated ones, which shall be designed following the rules reported in the corresponding sub clauses. In such cases, the most unfavourable value shall be assigned to the parameters referring to the overall response of the structure (e.g. the behaviour factor q).

A simplified design flowchart for the required steps to design a cladding to concrete structure connection is given in Annex A. Specific suggestions regarding the integrated systems structural model analysis are given in 7.2.2 to 7.2.5.

7.2.2 Behaviour factor

In buildings with integrated arrangements of panel connections, the panel claddings participate in the lateral load resisting system. Typically, the lateral resistance of the panel claddings is more than 50 % of the total lateral resistance of the building. Therefore, such buildings are classified as wall systems or wall-equivalent dual systems.

In addition, reinforced concrete buildings with cladding panels shall be designed for medium ductility. Therefore, the maximum allowed basic value of the behaviour factor is $q_0 = 3{,}0$.

Based on the above, the overall behaviour factor that is used in the seismic design shall be calculated with Formula (38):

$$q = q_0 \, k_w \geq 1{,}5 \quad (38)$$

where

q_0 is 3,0;

α_0 is the prevailing aspect ratio of the panel claddings, where $\alpha_0 = \frac{h_{wi}}{l_{wi}}$;

$k_w = \frac{(1+\alpha_0)}{3} \leq 1{,}0$ but not less than 0,5.

7.2.3 Design aspects

For structures with integrated arrangements of cladding panel connections, special attention shall be addressed to the analysis of floor and roof diaphragms, verifying their elements and relative internal and peripheral connections for the transfer of the inertia forces to the lateral resisting claddings. If a null diaphragm action is offered by the roof arrangement, the verification of the compatibility of the joint distortions shall be made.

It is noted that the large stiffness of the panel claddings can cause the development of large forces not only in the panel connections but also in all other connections of reinforced concrete members (roof-to-roof, roof-to-beam, beam-to-column), which shall be verified.

The connections shall not be designed for contributing to the ductile response of the system but shall be overdesigned. Hence, there shall not be any plastic deformations in the connections.

The design action-effects of the connections shall be derived based on the capacity design rules. For the dual wall-frame systems, over-proportioning of the structural cladding connections shall be made referring to the forces coming from a structural analysis performed with a behaviour factor $q = 1{,}5$.

7.2.4 Structural modelling

7.2.4.1 General issues

In general, due to the large stiffness of the panel claddings, the model of the structure that is used in the analysis must reflect the real stiffness distribution within the structure in order to be able to capture accurately the distribution of the internal forces that develop during the seismic excitation, especially the forces induced to the connections among the precast members.

For the numerical model of the structure, beam/column elements in combination with plate elements can be used, positioned along the axis or in the mid-plane of the corresponding structural element. It is recommended to reproduce the different eccentricities between the members, using rigid link elements at their joints. The connections between the elements shall be faithfully represented with their degrees of freedom in the different planes, especially for the cladding connections.

If the connections are modelled with no deformability (e.g. fixed "built-in" full support or hinged support) the results of the analysis can lead to an unrealistic distribution of the joint forces. Thus, the actual deformability of the connections is deemed necessary in order to obtain reliable results.

To avoid an excessive number of modes to be considered in the modal analysis, it is recommended to neglect all the local vibrations of the elements by considering the masses, mainly the masses of cladding panels, concentrated at the joints of the frame structure.

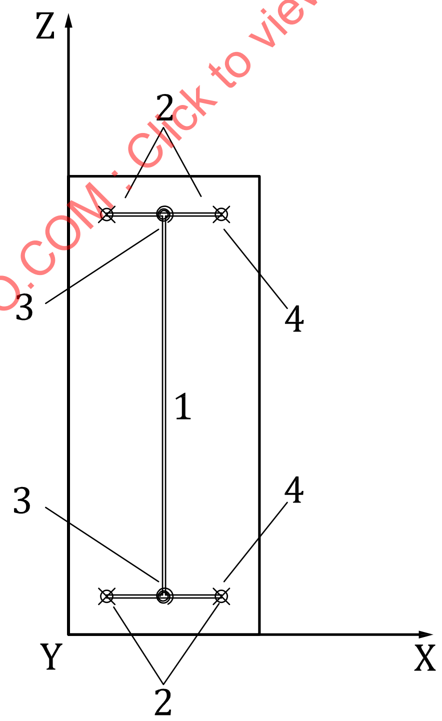
7.2.4.2 Modelling of the cladding panels

Besides using plate elements for modelling the behaviour of the panels, beam-column elements can also be selected. Each panel can be modelled with five elastic elements (see Figure 34). The main element is placed at the centreline of the panel while the remaining four elements are used to reach the connections with the beams.

If the connections are modelled with no deformability (e.g. fixed "built in" full support or hinged support), the results of the analysis can lead to very high joint forces. The actual even small deformability of the connections can lower sensibly these forces. More reliable results can be obtained if also the actual deformability of the connections is reproduced in the model.

In order to account for the deformability of the connections, zero-length rotational springs should be placed at the ends of the panel element (see Figure 34) which capture the overall rotational response at the panel-beam joint.

In the case that the two upper fastenings are replaced by vertically sliding connections to allow thermal expansion of the panel [see Figure 37 b)], the rotation at the top side of the panel is released. Therefore, the stiffness of the top rotational spring is set to zero (pinned connection between the panel element and the top connecting elements).



Key

1 panel element

2 connecting element

3 zero length element

4 connection

Figure 34 — Cladding panel numerical model

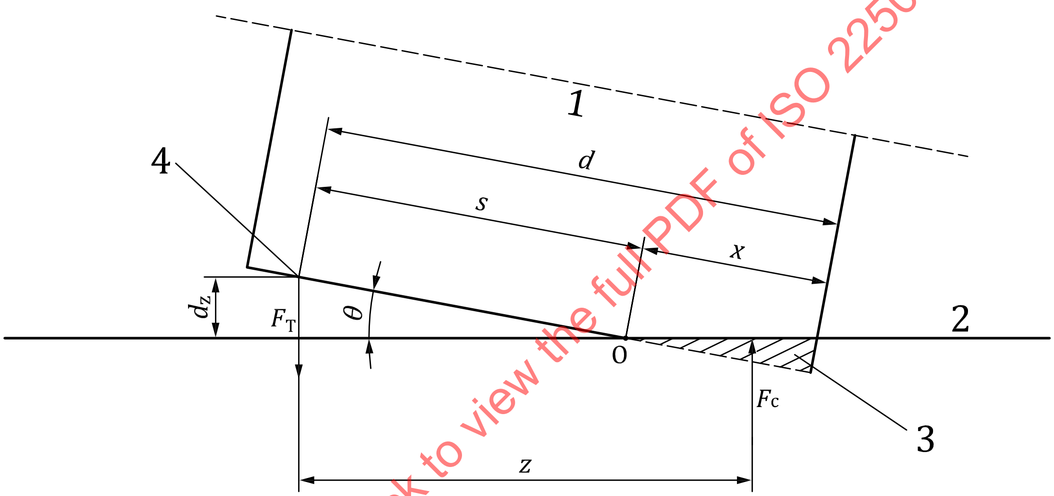
The stiffness, K_θ, of the zero-length rotational spring can be calculated assuming that the connection under tension can be simulated by a vertical linear spring of stiffness K_z (which refers to the stiffness up to the theoretical point of yielding). M is the bending moment at the base of the panel, which produces rotation, θ (see Figure 35). Then, the vertical displacement at the connection (elongation of the equivalent connection spring) is evaluated with Formula (39)

$$d_z = s\theta \tag{39}$$

where

S is the distance of the centreline of the connection from the neutral axis O

θ is the rotation angle



Key

1 pane
2 beam
3 compression zone
4 tension zone

Figure 35 — Rotating panel base

The tensile force induced to the connection is expressed with Formula (40):

$$F_T = K_z d_z = K_z s\theta \tag{40}$$

On the other hand, one can write Formula 41:

$$F_T = \frac{M}{z}, \tag{41}$$

where z is the inner lever arm of the tensile and the compressive forces developed at the base of the panel.

Combining Formulae (40) and (41) and setting $K_\theta = \frac{M}{\theta}$, Formula (42) can be derived:

$$K_{,} = K_z z s \tag{42}$$

where

z is set equal to z=0,9d, where d is the distance of the centreline of the connection from the opposite edge (effective depth);

s can be calculated as s=d–x, where x is the length of the compression zone that can be approximated by x=0,25d. Thus, s=0,75d.

For what concerns the value of K_z, for connections with protruding bars and wall shoes (see 7.3.4 and 7.3.5), one can write Formula (43):

$$K_z = \frac{EA}{L_{eff}} \tag{43}$$

where

E is the modulus of elasticity of the steel of the bars/bolts;

A is the stressed area of the bars/bolts;

L_{eff} is the equivalent length of the spring, denoting the effective length in which the elongation of the bars/bolts takes place.

L_{eff} can be estimated by Formula (44):

$$L_{eff} \cong 15d_b \tag{44}$$

where d_b is the diameter of the bar or the bolt.

If Formula (44) is used, it is suggested that two analyses are performed: one with double the value of K_θ, from which the maximum forces in the connections are determined, and one with half the value of K_θ, from which the maximum displacements is determined.

For connections with bolted plates, the calculation of the proper value of K_z is uncertain, since it is affected by a number of factors that cannot be easily modelled, as the shear deformation of the bolts, the elongation of the steel plate, the distortion of the holes and the plate itself, etc.

7.2.4.3 Pre-dimensioning of panel cladding connections

7.2.4.3.1 General

In order to calculate the rotational stiffness, K_θ, of the panel models by applying Formula (42), the cross-section of the bars/bolts is needed. Therefore, an initial evaluation of the forces expected to develop in the connections is necessary. This pre-dimensioning of the connections can be based on simplified assumptions concerning the distribution of the lateral forces, as the ones reported in 7.2.4.3.2 and 7.2.4.3.3.

This analysis can only be used for the pre-dimensioning of the connections, while the final design of the connections shall be based on the actual forces derived from the dynamic modal analysis. In case that this design shows that some connections need to be modified, the analysis shall be repeated.

7.2.4.3.2 Panels with four connections

Consider n panels at each side, pinned to the top and the bottom beam by two connectors at each edge. Each panel has dimensions $L_p \times H_p$, while L and H are the horizontal and the vertical distance between the connections of the panel (see Figure 36). Then, one can define the coefficient C_1 with Formula (45):

$$C_1 = L/L_p \tag{45}$$

In general, the total length, L_{tot}, of the building sides is not fully covered with panels, thus a coefficient C_2 can be defined as in Formula (46):

$$C_2 = n \cdot L_p / L_{tot} \tag{46}$$

Figure 36 — Connection distances

NOTE $C_2 = 1$ for sides fully covered with panels, but it can be significantly smaller than unity for sides with long openings (partially covered).

For a symmetrical building and cladding wall panels placed at the two external sides, and accounting for the large stiffness of the panels compared to the stiffness of the reinforced concrete frame, it can be assumed, as a first approximation, that the base shear due to the earthquake load, P_{base}, is taken only by the panels. Then it can be proved that the horizontal force, $P_{i,h}$ [see Formula (47)], and the vertical force, $P_{i,v}$ [see Formula (48)] that are induced to each panel connection are (see Figure 36):

$$P_{i,h} \cong \frac{P_{base}/2}{2n} \tag{47}$$

$$P_{\mathrm{i,v}} = P_{\mathrm{i,h}} \frac{H}{L} \tag{48}$$

The total force induced to each connection is expressed by Formula (49)

$$P_{\mathrm{i}} = \sqrt{P_{\mathrm{i,h}}^2 + P_{\mathrm{i,v}}^2} \tag{49}$$

and using the above, one can write [see Formula (50)]:

$$\frac{P_{\mathrm{i}}}{P_{\mathrm{base}}} = \frac{1}{4}\sqrt{\frac{1}{n^2} + \left(\frac{H}{C_1 C_2 L_{\mathrm{tot}}}\right)^2} \tag{50}$$

In general, for values of n larger than about 4, the term $1/n^2$ is much smaller than the term $\left(\frac{H}{C_1 C_2 L_{\mathrm{tot}}}\right)^2$ and can be neglected in Formula (50).

Denoting with P^* the base shear per unit length [see Formula (51)]:

$$P^* = \frac{P_{\mathrm{base}}}{L_{\mathrm{tot}}} = \frac{C_1 C_2 P_{\mathrm{base}}}{nL} \tag{51}$$

one obtains Formula (52):

$$P_{\mathrm{i}} = \frac{P^*}{C_1 C_2} \cdot \frac{H}{4} \tag{52}$$

7.2.4.3.3 Panels with vertically sliding top connections

For the same assumptions of the above analysis, but considering panels free to rotate at their top, Formula (53) can easily be proved

$$P_{\mathrm{i}} = \frac{P^*}{C_1 C_2} \cdot \frac{H}{2} \tag{53}$$

showing that the forces induced to the bottom connectors are double than the ones for panels with four connections. Formulae (52) and (53) imply that the force induced to each connection is independent of the width of the panels. This practically means that the forces at the cladding connections cannot be reduced by using more panels of smaller length or less panels of larger length. However, the connection forces greatly depend on the “coverage” of the external sides by panels (coefficient C_2) and increase significantly in case of sides with long openings (partially covered by panels). The force induced to each connection is linearly increasing with the vertical distance H of the connections, i.e. with the height of the storey. The major component of P_{i} is in the vertical direction.

7.2.5 Cladding panels detailing

Specific requirements for integrated cladding panels used in the dual wall-frame systems are presented henceforth. These requirements are formulated through proper adaptation of the rules for the cast-in-situ shear walls and aim at ensuring strong fastening of the connectors, adequate in plane shear resistance and sufficient ductility. Specifically:

a) panels shall have a solid bearing layer of at least 150 mm of thickness;

b) a double reinforcing mesh of ductile steel shall be provided at the two faces;

c) in both directions, the sides of the mesh shall be not larger than 200 mm and the bars should have diameter at least 8 mm;

d) a perimeter reinforcement shall be added with at least 2 longitudinal bars of diameter at least 12 mm and edge links of diameter at least 8 mm;

e) proper anchoring reinforcement of the inserts shall be located at the connection points.

Panels with openings shall be properly designed for the transmission of the expected in plane actions through the lateral posts of the openings. Proper reinforcement, specifically continuous steel ties, horizontal or vertical, shall be provided around the openings, similar to the reinforcement placed around openings in ductile shear walls.

7.3 Design of integrated systems connections

7.3.1 General

In the integrated system, the connections of each panel are arranged with a hyper-static set of fixed supports. The term “fixed” is here used interchangeably with the term “pinned” and denotes connections restrained in displacements only, while rotations are allowed. With this arrangement of connections, the panels participate to the seismic response of the structure within a dual wall-frame system that has a much higher stiffness, a lower energy dissipation capacity compared to a pure frame, and this leads to a structural seismic response with higher forces and lower displacements.

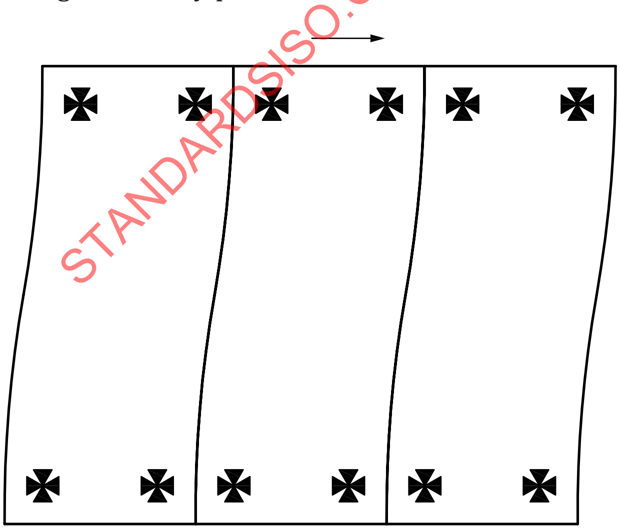
The panel connections shall be proportioned, by consequence, not with a local calculation based on the mass of the single panel, but from the analysis of the overall structural assembly with its global mass.

The adoption of an integrated system has also some side effects, such as those of a strong engagement of the floor diaphragm action necessary to transfer the inertia forces of the floors to the lateral resisting walls.

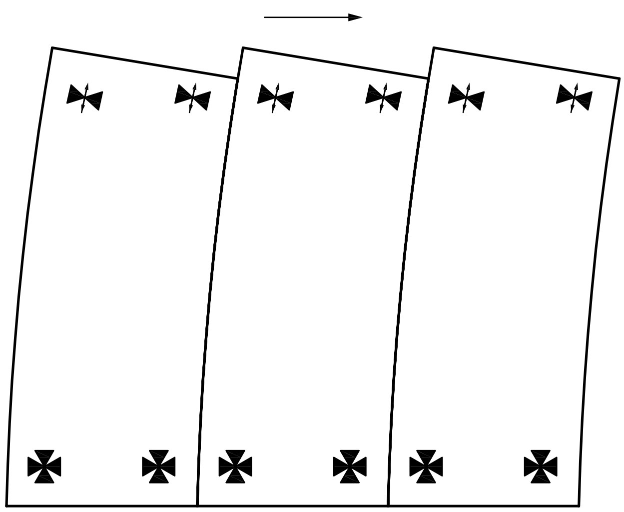
7.3.2 Structural arrangements

7.3.2.1 Vertical structural arrangements

A typical hyper-static arrangement of connections is shown in Figure 37 a) for vertical panels. Four fixed fastenings are used, one for each corner, the lower two attached to the bottom beam, the upper two attached to the top beam. The corresponding arrangement matrix is given in Table 10. With this arrangement, any panel acts as a vertical beam clamped at both ends.



a) Hyper-static



b) Sliding top connections

Figure 37 — Vertical structural arrangements

For large size panels, thermal fluctuations can induce significant axial forces to the panels if their expansion is prevented. These forces can lead to local damages and widespread cracking and to out-of-plane buckling. In such cases, thermal forces shall be considered in the design of the panels.

In order to allow the free vertical thermal expansion of the panel, the two upper fixed fastenings can be replaced by as many vertically sliding connections. Figure 37 b) shows this arrangement, in which each panel acts as a vertical cantilever beam clamped at its bottom and pinned at its top. The corresponding arrangement matrix is given in Table 11. For the same horizontal top action, this arrangement leads to double the vertical reactions of the lower connections compared to the arrangement of Figure 37 a). It is noted that the two upper corner connections can be replaced by only one central connection.

Table 10 — Arrangement matrix – vertical integrated

Direction	A	B	C	D	E	F
x	f	f	f	f	/	/
y	f	f	f	f	/	/
z	f	f	f	f	/	/

Table 11 — Arrangement matrix – vertical sliding top connections

Direction	A	B	C	D	E	F
x	f	f	f	f	/	/
y	f	f	f	f	/	/
z	f	f	s	s	/	/

7.3.2.2 Horizontal structural arrangements

Figure 38 shows a hyper-static arrangement of connections for horizontal panels. Four fixed fastenings are used, one for each corner, attached to the contiguous columns. The corresponding arrangement matrix is the same as the one given in Table 10. With this arrangement, any panel acts as a horizontal beam clamped at its ends.

In case of horizontal panels, the fixed connections applied to the columns affect significantly their deformation during earthquakes, inducing the development of high local forces. Additionally, the insertion of adequate fastening devices in the reduced dimensions of the columns, without affecting their resistance, can be a difficult construction problem. For these reasons, horizontal panel arrangement in integrated systems should not be used, especially in zones with moderate to high seismicity.

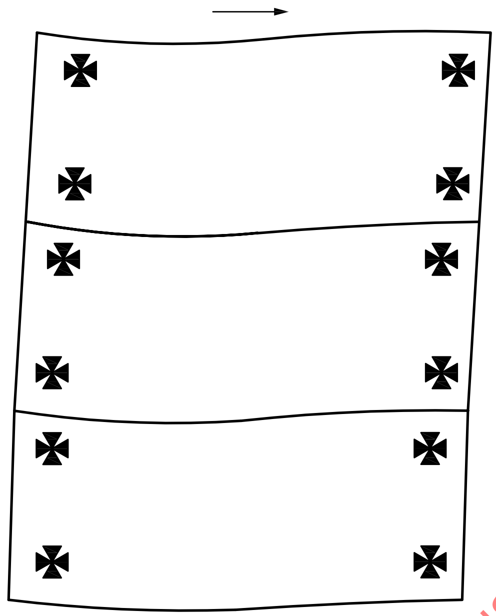


Figure 38 — Horizontal hyper-static structural arrangement

7.3.3 Base supports

Three types of base fixed connecting mechanisms are treated below:

a) connections with protruding bars;

b) connections with bolted shoes;

c) connections with bolted plates.

Independently from the type of connection, a gap is typically left, during construction, between the panel and the supporting beam, which is filled with high strength, non-shrinking grout after mounting the panels. The purpose of this bed of mortar is to form a uniform contact between the panel and the supporting beam, necessary to ensure friction and prevent sliding. Special holding provisions are required to ensure the proper positioning of the panels and to ensure their stability during erection.

The thickness of this bed of mortar should not exceed 50 mm. Use of fibre-reinforced grout leads to better behaviour.

Panels with integrated connections behave as clamped at their ends. For this reason, and taking under consideration their large in-plane stiffness, significant internal forces can develop during strong earthquakes. Openings in the panels reduce their local strength and, therefore, appropriate designs should be performed to ensure the ability of the reduced cross-sections of the panel at the place of the openings to transmit the large shear forces and moments. In addition, proper reinforcement, specifically continuous steel ties, horizontal or vertical, should be provided around the openings, similar to the reinforcement placed around openings in ductile shear walls.

7.3.4 Connections with protruding bars

7.3.4.1 General

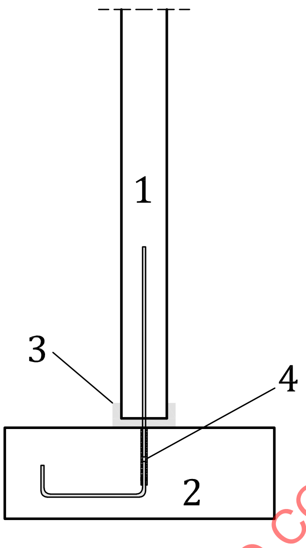
This type of connections can be executed using steel bars (e.g. reinforcement rebars) protruding from the panels into the beams or vice versa. In both cases, waiting corrugated sleeves are provided in the opposite element for the insertion of the bars. These sleeves are filled with high strength, non-shrinking fluid mortar after erection.

In Figure 39 a), the typical connection in case of bars protruding from the panel into the supporting beam is shown. In case that the connecting bars are protruding from the supporting beam into the panel, the corrugated sleeves are placed in the panels and must have holes for the injection of the grout. Special holding provisions are required to ensure the proper positioning of the panels and to ensure their stability until hardening and sufficient aging of the mortar.

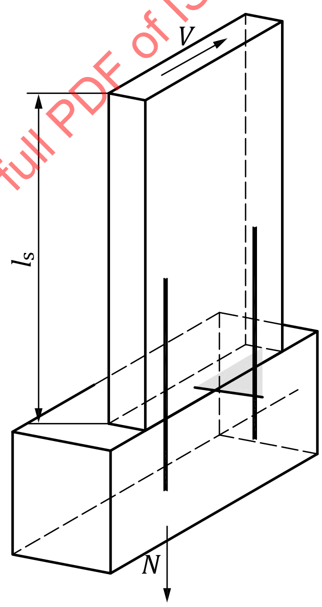
No special requirements apply to the embedded part of the protruding steel bars. However, proper reinforcement shall be placed around the sleeves to confine the concrete around them and anchor them against pull-out. The use of smooth sleeves instead of corrugated ones is not allowed, as pull-out slip can occur at the surface of the duct.

An alternative solution can be the use of mechanical couplers in order to attach the protruding part of the bars in site. Special technological provisions, depending on the adopted coupler, shall be applied to ensure the required strength of the connections. If bolted bushes are used, proper provisions should be adopted to avoid that the weakening of bars due to threading jeopardizes the strength of the connection leading to an early brittle failure.

In any case, when using mechanical couplers, the coupling device shall have been experimentally qualified for its effectiveness in terms of over-resistance with respect to the connected bars.



a) Setup



b) Resisting mechanism

Key

1 panel

2 beam

3 Non-shrinking high-strength grout

4 corrugated anchor sleeve

Figure 39 — Connections with protruding

The same type of connection can be used at the upper side of the panel to fix it to a superimposed element of the structure.

7.3.4.2 Strength

Figure 39 b) shows the resisting mechanism at the panel-to-beam joint under a seismic action that applies a shear force V to the panel. The figure shows only the height of the panel up to the zero-moment point (shear length, l_s), which is equal to one half of the total height in the case of Figure 34 and equal to the full height in the case of Figure 35.

The connecting bars provide the tensile strength in the tension zone, while the concrete provides the resistance in the compression zone [see Figure 39 b)]. No special measures are usually required to prevent sliding in the horizontal direction, since the horizontal resistance provided, mainly by the friction between the panel and the beam and secondarily by the dowel action of the bars, is adequate to sustain the horizontal forces.

The possible failure modes associated with the resisting mechanism shown in Figure 39 b) are:

a) pull-out of the tension bars;

b) break of the tension bars;

c) permanent elongation of the tension bars due to large strains developed;

d) failure of the concrete in the compression zone (not probable, except in case of very high forces);

e) sliding shear failure of the panel (not probable, except in case of yielded connecting bars).

Permanent elongation of the connecting bars occurs after their yielding with several unfavourable side effects, such as:

a) the panels stop being in contact with the beams. As a result, the friction resistance is lost and sliding shear failure of the panel can occur;

b) the nonlinear behaviour of the connections under cyclic loading is characterized by considerable pinching.

To avoid such undesired situations, the connections shall be overdesigned. Thus, the design action-effects of the connections shall be derived based on the capacity design rules.

Since the application of capacity design criteria to dual wall-frame integrated systems is, in general, difficult, it is suggested that the over-proportioning of the connections is made referring to the forces obtained from a structural analysis performed with behaviour factor q = 1,5.

Rules for the calculation of the forces that develop in the connections are given in 7.2.

7.3.4.3 Designs

7.3.4.3.1 Design against axial yielding

Formula (54) shall hold:

$$N_d \leq N_{Rd} \tag{54}$$

where

N_d is the axial force induced to the connecting bar, calculated from a structural analysis performed with behaviour factor q = 1,5 as mentioned in 7.3.4.2;

N_{Rd} is the axial resistance of the connecting bar calculated with Formula (55).

$$N_{Rd} = A_s f_{yd} \tag{55}$$

where

A_s is the cross-section area of the bar;

$f_{yd} = \frac{f_{yk}}{\gamma_s}$ is the design yield stress of the steel of the bar;

f_{yk} is the characteristic yield stress of the steel of the bar;

γ_S is the safety factor for steel (γ_S = 1,15).

7.3.4.3.2 Design against pull-out

To avoid a brittle bond failure, the anchorage length of the connecting bars shall be over-proportioned by capacity design with respect to the yielding force of the bars. Formula (56) shall hold:

$$l_b\, u\, f_{bd} \geq \gamma_R\, A_s\, f_{ym} \tag{56}$$

where

l_b is the anchor length of the bar;

$u = \pi d$ is the perimeter of the bar, d being its diameter;

$f_{bd} = 0{,}45 \cdot f_{md}$ is the bond strength of the mortar, f_{md} being the design cylinder compressive strength;

γ_R is the over-strength factor (γ_R =1,2 for medium ductility and γ_R =1,35 for high ductility);

A_s is the cross-section area of the bar;

$f_{ym} = 1{,}08 \cdot f_{yk}$ is the mean yielding stress of the steel of the bar.

7.3.4.4 Any other data

The deformation capacity of the connection is governed by the elongation capacity, d_z, of the connecting bars near the joint. The effective length, L_{eff}, in which the bar elongation takes place, extends in a distance of several bar diameters on both sides of the joint, where the bond capacity has reached its maximum value. L_{eff} should be about 15 times the bar diameter.

Concerning the contribution of the joint opening to the drift capacity of the panel, this depends on the rotation θ at the base of the panel. This rotation can be approximated by Formula (57) (see 7.2.4.2):

$$\theta = \frac{d_z}{0{,}75\, d} \tag{57}$$

where d is the distance of the connecting bars from the opposite edge of the panel.

7.3.5 Connections with wall shoes

7.3.5.1 General

Figure 40 a) shows devices, usually called “wall shoes”, used to connect cladding panels to beams or to other cladding panels. A typical wall shoe consists of:

a) a steel nest with a strong bottom plate which is cast into the bottom part of the upper cladding and fixed with anchor bars;

b) an anchor bolt which is cast into the upper part of the lower cladding or beam;

c) a washer and a nut, used to fasten the bolt to the bottom plate.

There are special requirements concerning:

a) the minimum concrete cover;

b) the minimum thickness of the cladding cross section;

c) the minimum distance from the cladding edges;

d) the principal and the supplementary reinforcement;

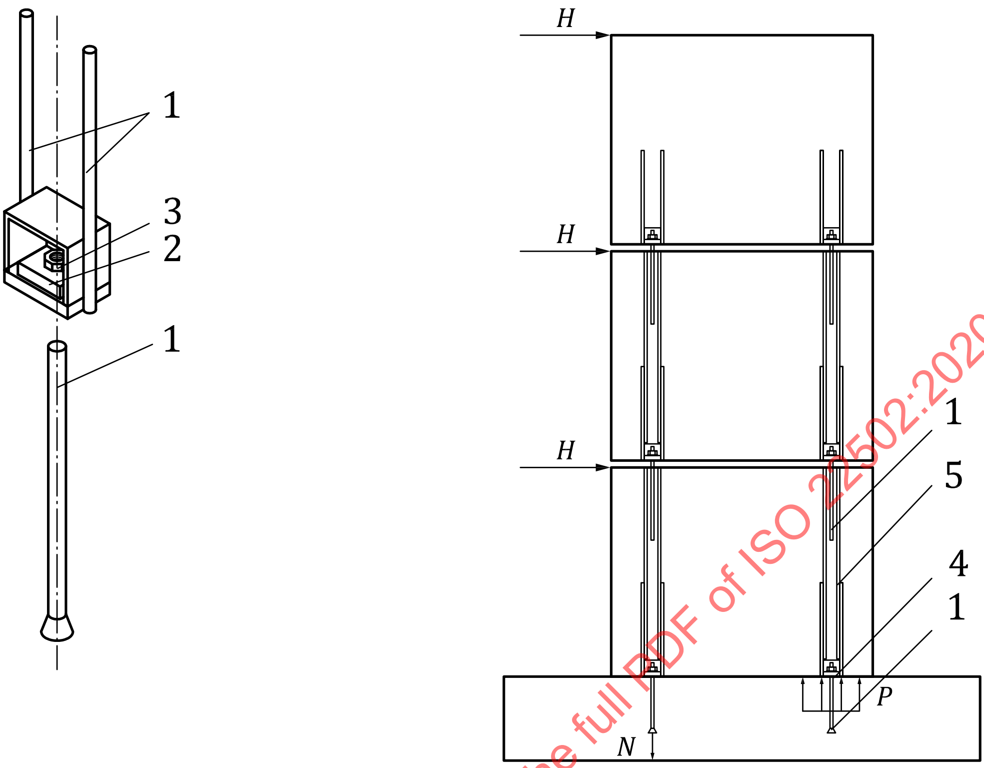
Which are usually specified by the producer. These requirements depend on the nominal force that each device can resist and vary from system to system.

Proper reinforcement shall be placed around the anchor bolts to confine the concrete around them and anchor them against pull-out.

7.3.5.2 Strength

Figure 40 shows the resisting mechanism at the panel-to-beam connection under a seismic action that applies horizontal forces, H, to the panels at any floor of the structure. The connections provide the tensile strength in the tension zone, while the concrete provides the resistance in the compression zone.

No special measures are usually required to prevent sliding in the horizontal direction, since the horizontal resistance provided, mainly by the friction between the panel and the beam and secondarily by the dowel action of the bolts, is adequate to sustain the horizontal forces.



a) **Wall shoe device** b) **Wall shoe connection resisting mechanism**

Key

1	anchor bolt	3	nut	5	continuity
2	washer	4	wall shoe		

Figure 40 — Wall shoe device and resisting mechanism

The possible failure modes associated with the resisting mechanism shown in Figure 40 are:

a) pull-out of the anchor bolts;

b) break of the anchor bolts;

c) permanent elongation of the anchor bolts due to large strains developed;

d) warping of the washer plate;

e) failure of the concrete in the tension zone (not probable except in case of very high forces);

f) sliding shear failure of the panel (not probable except in case of yielded anchor bolts).

Permanent elongation of the anchor bolts occurs after their yielding. Hence several unfavourable side effects arise, such as:

a) the nuts stop being tightly fastened to the washer, and as a result, the connections turn loose;

b) the nonlinear behaviour of the connections under cyclic loading is characterized by significant pinching.

To avoid such undesired situations, the connections shall be overdesigned. Thus, the anchor bolts shall be verified for the axial forces induced to the connections under the design earthquake assuming an elastic response (behaviour factor q = 1,5). Rules for the calculation of the forces that develop in the connections are given in 7.2.

In general, all parts of the wall shoes, except the anchor bolt, should be over-proportioned, since only the anchor bolt is allowed to yield.

Only wall shoe systems that have been experimentally qualified shall be used.

7.3.5.3 Designs

7.3.5.3.1 General

For industrially manufactured wall shoes, the design consists mainly on preventing the axial yielding of the anchor bolt and the failure of the concrete in the compression zone. Details about the connection capacity and the additional measures necessary to ensure its integrity can be found in the technical specifications of the product.

7.3.5.3.2 Design against axial yielding of the anchor bolt

Formula (58) shall hold:

$$N_{\mathrm{d}} \leq N_{\mathrm{Rd}} \tag{58}$$

where

N_{d} is the axial force induced to the anchor bolt, calculated from a structural analysis performed with behaviour factor q = 1,5;

N_{Rd} is the design axial resistance of the anchor bolt according to the specifications provided by the producer.

7.3.5.3.3 Design against failure of the concrete in the compression zone

The design against failure of the concrete in the compression zone shall be performed based on the moment that develops at the beam-panel joint under the design seismic loads. The joint moments are given as an output of the analysis if the panel model described in 7.2 is used.

7.3.5.3.4 Other failure modes

The design against breakage of the anchor bolts, permanent elongation of the anchor bolts, warping of the washer plate and sliding shear failure of the panel does not deem necessary, because it is improbable that these failure modes can occur, since the connections are overdesigned in accordance with the capacity design rule.

7.3.5.3.5 Any other data – Deformation

The deformation capacity of the connection is governed by the elongation capacity, d_{z}, of the anchor bolts. The effective length, L_{eff}, in which the bolt elongation takes place is equal to the sum of the thickness of the bottom plate of the shoe, the thickness of the washer and the length along the cast part of the bolt (beam side), where the bond capacity has reached its maximum value.

Concerning the contribution of the joint opening to the drift capacity of the panel, it depends on the rotation, θ, at the base of the panel. This rotation can be approximated by Formula (59):

$$\theta = \frac{d_{\mathrm{z}}}{0,75\,d} \tag{59}$$

where d is the distance of the anchoring bolt from the opposite edge of the panel.

7.3.6 Connections with bolted plates

7.3.6.1 General

This type of connections can be executed using steel plates that are connected to waiting nests fixed to the supporting beam and to the panel by an adequate number of bolts. The steel nests are embedded in the concrete and welded to reinforcing rebars, so that the connection forces are gradually transferred to the concrete. A typical configuration of this connection is shown in Figure 41.

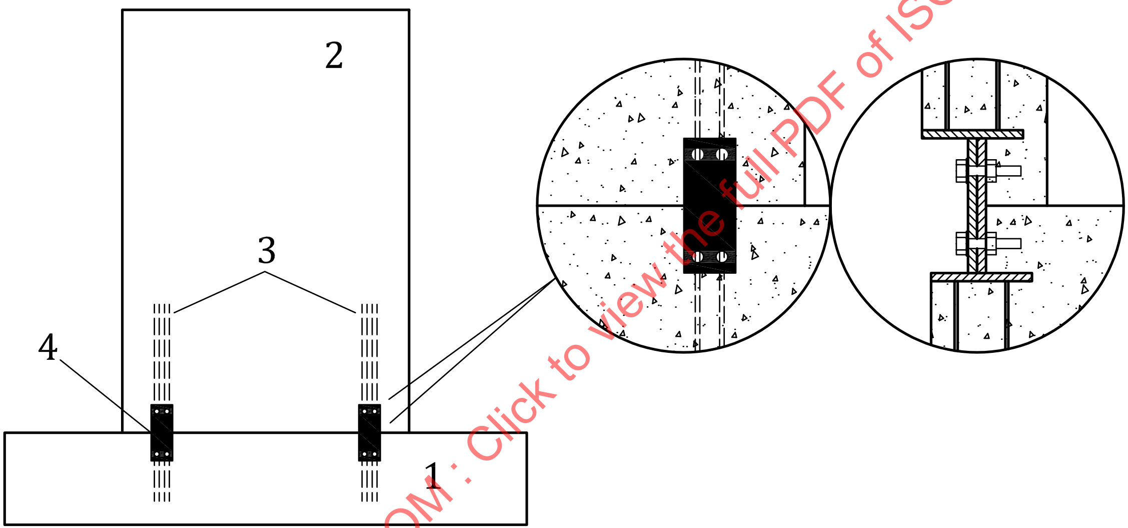
There are special requirements which depend on the nominal force that each device can resist and aim at preventing local damage to the concrete, concerning:

a) the minimum concrete cover;

b) the minimum thickness of the cladding's cross-section;

c) the minimum distance from the cladding edges;

d) the principal and the supplementary reinforcement.



Key

1 beam

2 panel

3 steel bars

4 bolted plate

Figure 41 — Bolted plates connection

Yielding of the steel plate is not expected and the critical failure mechanism is the shear failure of the bolts, which shall be avoided.

7.3.6.2 Strength

Figure 42 shows the resisting mechanism at the panel-to-beam joint under a seismic action that applies a shear force V to the panel. The figure shows only the height of the panel up to the zero-moment point (shear length, l_s), which is equal to one half of the total height in the case of Figure 34 and equal to the full height in the case of Figure 35.

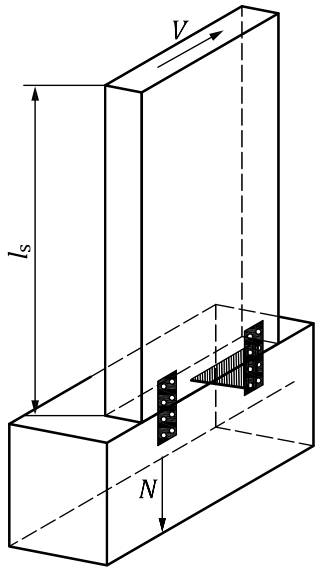


Figure 42 — Bolted plates connection resisting mechanism

The connecting mechanism provides the tensile strength in the tension zone, while the concrete provides the resistance in the compression zone. No special measures are required to prevent sliding in the horizontal direction, since the horizontal resistance provided by the friction between the panel and the beam and by the shear resistance of the connection is adequate to sustain the horizontal forces.

The possible failure modes associated with the resisting mechanism are:

a) shear failure of the connecting bolts;

b) permanent distortion of the bolts and/or the plate due to large strains developed;

c) loosening of the connection due to the permanent distortion of the bolt holes;

d) failure of the connecting plate;

e) failure of the concrete in the compression zone;

f) sliding shear failure of the panel.

Over-proportioning of the connections shall be made referring to the forces from a structural analysis performed with behaviour factor q = 1,5. Rules for the calculation of the forces that develop in the connections are given in 7.2.

7.3.6.3 Designs

7.3.6.3.1 Design against bolt shear failure

To avoid shear failure of the bolts, Formula (60) shall hold:

$$N_{\rm d} \le V_{\rm Rd} \tag{60}$$

where

N_d is the axial force induced to the connection, calculated from a structural analysis performed with behaviour factor q = 1,5;

$V_{\rm Rd}$ is the shear resistance of the connecting bolts calculated by Formula (61).

$$V_{\mathrm{Rd}} = n\, A_{\mathrm{s}}\, \alpha_{\mathrm{v}} \frac{f_{\mathrm{ub}} / \gamma_{\mathrm{M2}}}{\gamma_{\mathrm{Rd}}} \quad (61)$$

where

n is the number of bolts on each connected element;

A_{s} is the stress area of each bolt when the shear plane passes through the threaded portion of the bolt;

α_{v} is a constant depending on the bolt class and the location of the shear plane:

- for classes 4,6~ 5,6 and 8,8, when the shear plane passes through the threaded portion of the bolt: α_{v} = 0,6;
- for classes 4,8, 5,8, 6,8, and 10,9, when the shear plane passes through the threaded portion of the bolt: α_{v} = 0,5;
- when the shear plane passes through the un threaded portion of the bolt: α_{v} = 0,6;

f_{ub} is the nominal ultimate tensile strength of the bolts;

γ_{M2} is the material safety factor for the design of bolts (γ_{M2}=1,25);

γ_{Rd} is a general safety factor that also accounts for the reduction of the shear strength of the bolts due to the simultaneous development of tensile stresses caused by the out-of-plane distortion of the plate. Since these tensile stresses cannot be calculated within a typical design procedure, and it is desirable to design the connection to avoid the shear failure of the bolts, a value of γ_{Rd} = 1,20 is recommended.

7.3.6.3.2 Design against failure of the connecting plate

To avoid failure of the connecting plate, Formula (62) shall hold:

$$N_{\mathrm{d}} \le \min\left\{ N_{\mathrm{pl,Rd}}, N_{\mathrm{u,Rd}} \right\} \quad (62)$$

where

N_{d} is the axial force induced in the connection, calculated from a structural analysis performed with behaviour factor q = 1,5;

$N_{\mathrm{pl,Rd}}$ is the plastic resistance of the gross cross-section calculated by Formula (63);

$N_{\mathrm{u,Rd}}$ is the ultimate resistance of the net cross-section calculated by Formula (64).

$$N_{\mathrm{pl,Rd}} = A \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} \quad (63)$$

$$N_{\mathrm{u,Rd}} = 0{,}9\, A_{\mathrm{net}} \frac{f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}} \quad (64)$$

where

A is the gross area of the plate;

A_{net} is the net area of the plate at the position of the holes;

f_y is the nominal yield stress of the steel of the plate;

f_u is the nominal ultimate tensile strength of the plate;

γ_{M0} is the material safety factor (γ_{M0} = 1,00);

γ_{M2} is the material safety factor (γ_{M2} = 1,25).

7.3.6.3.3 Design against permanent distortion of the bolt holes

To avoid permanent distortion of the bolt holes, Formula (65) shall hold:

$$N_d \leq \frac{n k_1 \alpha_b f_u d t}{\gamma_{M2}} \tag{65}$$

where

N_d is the axial force induced in the connection, calculated from a structural analysis performed with behaviour factor q = 1,5;

n is the number of bolts on each connected element;

f_u is the nominal ultimate tensile strength of the plate;

f_{ub} is the nominal ultimate tensile strength of the bolt;

d is the nominal diameter of the bolt;

t is the thickness of the plate;

γ_{M2} is the material safety factor (γ_{M2} = 1,25);

α_b is the smallest of α_d, $\frac{f_{ub}}{f_u}$ or 1,0;

in the direction of load transfer:

— for end bolts: $a_d = \frac{e_1}{3d_0}$;

— for inner bolts: $a_d = \frac{p_1}{3d_0} - \frac{1}{4}$;

where

p_1 is the distance between the bolts in the direction of the load transfer

p_2 is the distance between the bolts in the orthogonal direction

e_1 is the distance of the bolt from the edge in the direction of the load transfer

e_2 is the distance of the bolt from the edge in the orthogonal direction

d_0 is the diameter of the hole

perpendicular to the direction of load transfer:

k_1 is the smallest of $2,8\frac{e_2}{d_0}-1,7$, $1,4\frac{p_2}{d_0}-1,7$ and 2,5 for edge bolts;

is the smallest of $1,4\frac{p_2}{d_0}-1,7$ and 2,5 for inner bolts.

7.3.6.3.4 Design against failure of the concrete in the compression zone

The design against failure of the concrete in the compression zone shall be performed based on the moment that develops at the beam-panel joint under the design seismic loads. The joint moments are given as an output of the analysis if the panel model described in 7.2.

7.3.6.4 Any other data

The deformation capacity of the connection is governed by the overall elongation capacity d_z, which is the cumulative sum of the elongation of the steel plate, the shear deformation of the bolts and the widening of the holes. The elongation d_z depends on the strength of the steel plate and the diameter and the strength of the bolts and it is not easy to calculate analytically.

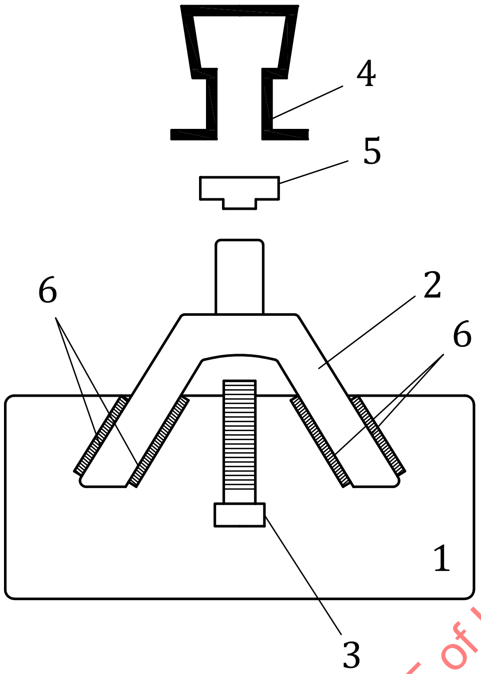
Concerning the contribution of the joint opening to the drift capacity of the panel, it depends on the rotation θ at the base of the panel. This rotation can be approximated by Formula (66):

$$\theta=\frac{d_z}{0,75\,d} \tag{66}$$

where d is the distance of the centreline of the connection from the opposite edge of the panel

7.3.7 Shear keys

Shear keys couple the horizontal in-plane displacements of panel and structure, providing a contemporary panel out-of-plane displacement restraint. For vertical panels, they should allow the vertical displacements related to thermal expansion and other geometrical effects. The efficiency of the sliding mechanism shall be experimentally verified through monotonic and cyclic tests, in accordance with 7.3.3.



Key

1 embedded steel plat
2 shear connector
3 bolt
4 slot
5 nut
6 weld

Figure 43 — Shear key connection

Figure 43 shows a type of shear key made by a steel element welded to a steel plate embedded in the beam or column, and connected with the anchor channel cast into the panel through a bolted connection.

For the bearing capacity of the device, reference shall be made to the technical specifications of the producer, that are usually based on experimental testing. The out-of-plane resistance design of the connection is made with reference to the transverse force calculated based on the mass of the panel.

The shear keys shall satisfy the vertical displacement and rotation demand and withstand the design load combination (comprising in-plane and out-of-plane actions) and additional forces related to friction.

An initial type testing shall be performed submitting the device to monotonic and cyclic longitudinal actions for different values of the transverse force (3D effects) consistently with the joint arrangement expected in the structure and with the geometrical allowances, considering also the production and installation tolerances. The proper functioning shall be checked all over the range of the expected vertical displacements and rotations.

8 Dissipative systems

8.1 General

Between the two extreme solutions of isostatic systems, with their large displacement demand, and integrated systems, with their high force demand, the dissipative systems of cladding connections offer an intermediate solution able to keep displacements and forces within lower predetermined limits.

The use of dissipative connections, placed between the panels as described in 8.3.2, introduces a source of friction or plastic hysteretic dissipation of energy in the dual wall-frame structural system. The contribution to energy dissipation of these connections in seismic behaviour of the overall structural system depends on the magnitude of their deformation and force capacities with respect to those of the overall structural system.

A simplified design flowchart for the required steps to design a cladding to concrete structure connection is given in Annex A. Specific suggestions regarding the dissipative systems structural model analysis are given in 8.2 and 8.3.

8.2 Analysis of the building

8.2.1 General

The friction dissipative devices considered in this paragraph refer to vertical panels only. They have a very small initial elastic flexibility and friction slide limited to few centimetres, with a constitutive law that can be represented by a rigid-friction diagram.

For the corresponding limited floor drifts, the columns that work in parallel remain usually within the elastic range and the dissipation of energy comes only from the dissipative devices. Therefore, with respect to the high seismic response of the initial stiff dual wall-frame system with fixed connections, the force reduction effects can result only from the set of dissipative devices interposed between the panels when the slip threshold is overcome.

To obtain a sensible force reduction, a suitable quantity of energy shall be dissipated. In the meantime, the stiffening effect that allows reducing sensibly the displacements with respect to those of the bare frame comes from the total contribution given by the friction devices in terms of resisting force. The same considerations are valid for the multi-slit devices.

For the panels used in the dissipative systems, the same detailing rules of 7.2.5 shall be applied.

The general approach for the design of reinforced concrete structures with dissipative systems of connections should be based on a nonlinear dynamic analysis applied to the spatial model of a dual wall-frame structure where the mutual panel connections are represented by their proper constitutive laws as presented in 8.3. For the details of the spatial model, reference can be made to 5.2.2 and 7.2.4.

A linear modal dynamic analysis of the dual wall-frame structure can be performed with the proper behaviour factor q representing the force reduction due to the dissipative connections. For the calibration of this behaviour factor, that is not presently regulated by the pertinent design codes, a preliminary parametric investigation is needed, comparing with a probabilistic approach the results obtained by the nonlinear and linear dynamic analysis for a significant set of structural situations.

Specific application rules are added in 8.3.3 for friction devices, while in 8.3.5 an alternative approach is presented for steel cushions.

In terms of roof drift, d_x, of a one-storey building, the relative slide clearance, $\pm s_z$, between two adjacent panels (see Figure 44) leads to Formula (67)

$$d_x = \pm \frac{s_z h}{b} \tag{67}$$

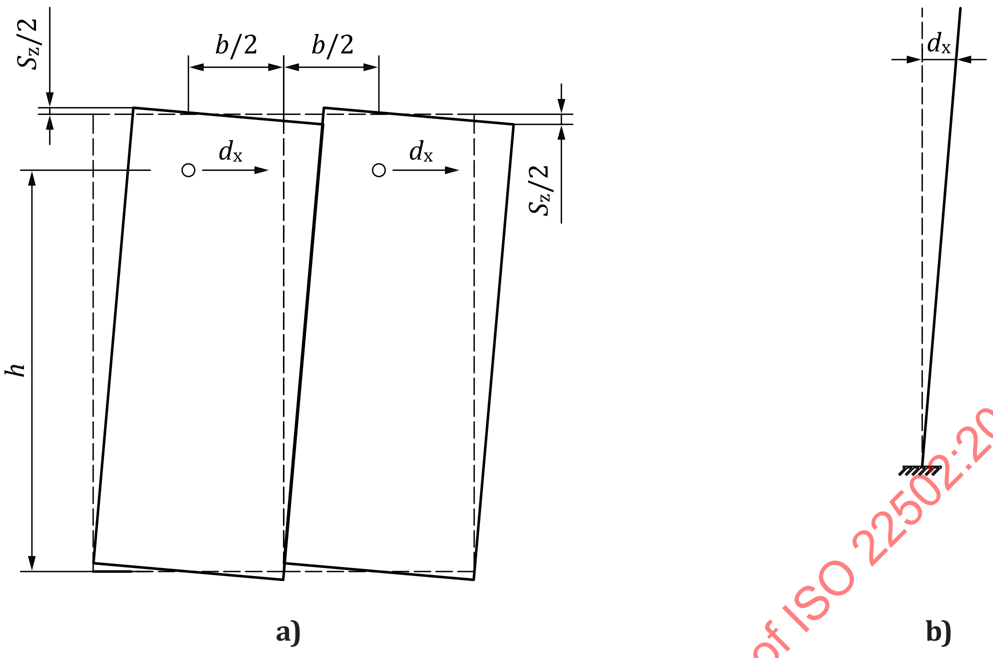


Figure 44 — Panels drift due to horizontal top displacement

Figure 45 shows the forces transmitted between the panels and to the frame structure, where:

a) the end panel is that transmits a relevant vertical force to the foundation;

b) an internal panel is that exchanges vertical shear forces at both sides;

c) a column of the frame connected at the top with the same roof diaphragm is.

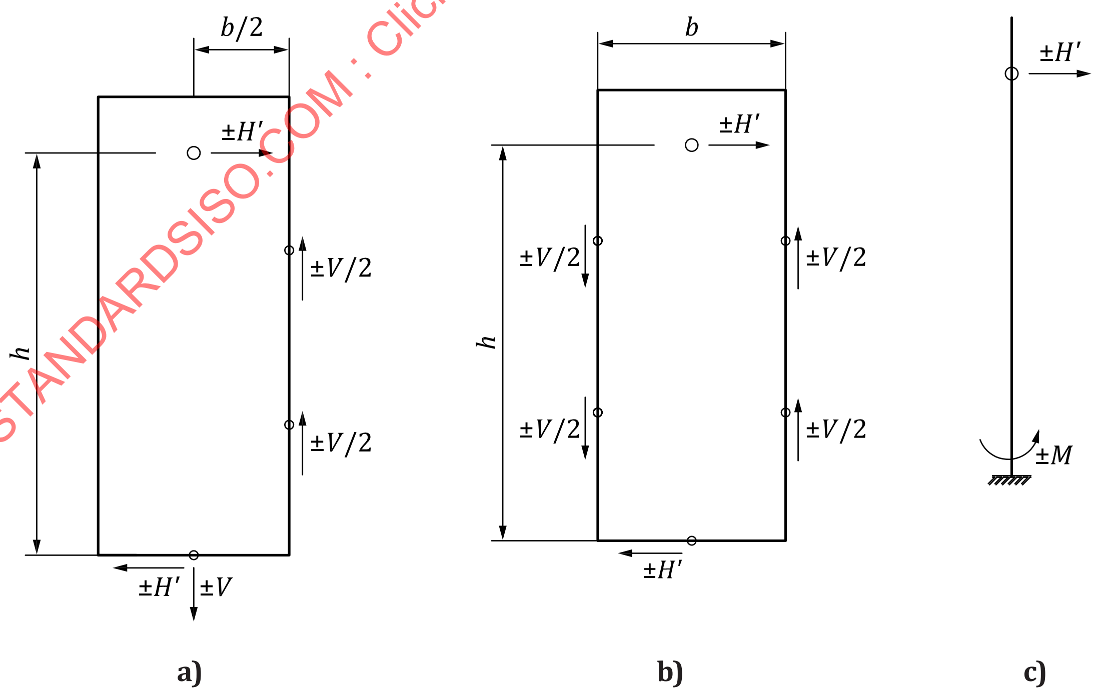


Figure 45 — Force transmission between vertical panels

The horizontal resistance contributions can be deduced from the rotation equilibrium [see Formulae (68) to (70)]:

$$H' = \frac{Vb}{2h} \quad (68)$$

$$H = \frac{Vb}{h} \quad (69)$$

$$V_o = \frac{M}{h} \quad (70)$$

where

V is the threshold force of an elastic-plastic model of a dissipative device;

M is the contemporary moment of the column base.

These horizontal forces correspond to the maximum response of the structure. Calling F_p the sum of the contributions H' and H of all the panels and F_c the sum of the contributions V_o of all the columns, this maximum response is expressed by Formula (71)

$$F = F_\mathrm{p} + F_\mathrm{c} \quad (71)$$

in which one can assume $F_\mathrm{c} = 0$ if the storey drift is kept within small values. With respect to the maximum response, F_{max}, of the integrated dual wall-frame system, in which the initial high stiffness is given by the large walls with fixed panel connections, this response leads to the required reduction (behaviour) factor [see Formula (72)]:

$$q = \frac{F_{\mathrm{max}}}{F} \quad (72)$$

The calculation of F_{max} can be referred to the total vibrating mass of the building and to the maximum spectral response of the site.

Formula (72) can be used to proportion the dissipative devices in number and strength by assuming a proper value of the q factor. For the panels used in the dissipative systems the same detailing rules of 7.2.5 shall be applied. This holds true as long as the total force contribution of the dissipative connections under the design seismic action is much higher than the total contribution of columns and the corresponding ultimate floor drift is compatible with the maximum displacement capacity of the dissipative devices.

8.2.2 Structures with friction devices

The seismic design of this kind of system should be based on proper nonlinear time-history dynamic analyses under prescribed ground motions. As an alternative, the classical linear methods of seismic analysis can be applied based on a conservative estimation of the behaviour factor.

Simplified seismic analysis can be carried out as follows:

1) definition of the elastic response spectrum of the site;

2) definition of the design response spectrum based on the conservative estimation of the reference value of the behaviour factor q;

3) static analysis or dynamic modal analysis of the 3D model of the dual wall-frame structure with integrated arrangement of the panels, attached to each other with fixed connections:

NOTE The first natural vibration period of the frame-panel earthquake resisting system is small and falls in the plateau region of the spectrum.

4) evaluation (from the above analysis) of the forces in the panel-to-panel connections and consequent design of the dissipative devices;

5) capacity design of the panel-to-frame connections according to Figure 45 and related formulae where $V = V_{max}$ of 8.3.3;

6) evaluation of the inelastic maximum top displacement of the dual wall-frame structure associated with the reference value of the behaviour factor [elastic displacement multiplied by $(1 + q)/2$] and computation of the corresponding inelastic relative displacements in the dissipative panel-to-panel devices, to be compared with their kinematic capacity, δ_{max}.

8.2.3 Structures with steel cushions

The steel cushions are elements with very large displacement capacity. Their energy dissipation capability is function of the top displacement or drifts of the structure. A combined energy dissipation of RC columns and steel cushions is the type of behaviour that is desired. At high levels of displacements, the cushions contribute largely to the overall energy dissipation of the entire structure. The key points here are:

a) target displacement of the system;

b) ductility demand on the cushions and on the columns;

c) amount of hysteretic energy and the overall damping.

These characteristics of the cushions suggest the use of a displacement-based design procedure rather than a forced-based.

In order to estimate the target displacement, the procedure below should be followed.

1) The fraction of the lateral load, β, to be carried by the cushions, is assigned by the designer. This β value should be in the range of 20 % to 60 %, depending on the stiffness of the rigid diaphragm.

2) Define the design displacement for the columns by calculating the yield and the ultimate displacements. The ultimate drift will be dictated either by the stability-related drifts of the columns, such as the maximum allowable rotation to prevent the toppling of the beams, or by the material strain limits of the column.

First, the yield displacement of the frame, $\Delta_{y,c}$, is calculated with Formula (73):

$$\Delta_{y,c} = \frac{\varphi_y \left(H + L_{sp}\right)^2}{3} \tag{73}$$

where φ_y is the yield curvature and is calculated as in Formula (74):

$$\varphi_y = \frac{2{,}10\,\varepsilon_y}{h_c} \tag{74}$$

Consequently, the overall design displacement of the frame, $\Delta_{d,f}$, is estimated as given in Formula (75) and compared to the displacement limit required for keeping the stability. The minimum of the two displacements is assumed as the design displacement of the frame.

$$\Delta_{d,f} = \Delta_{y,c} + \Delta_{p,c} = \frac{\varphi_y \left(H + L_{sp}\right)^2}{3} + \left(\varphi_{ls} - \varphi_y\right) L_p H \tag{75}$$

3) Assume a yield displacement for the cushions to be used. This yield displacement will be the first assumption since the steel cushions are not chosen yet, but the yield displacement of the cushions is highly correlated with their geometric properties. Thus, an accurate assumption for the yield displacement of the cushions can be made initially and can be revised with a single iteration at the end of the design process, before even starting the computer modelling.

4) Calculate the ductility demand of the RC columns and of the steel cushions.

5) Estimate the hysteretic overall damping for the structure from the contributions of the columns and of the steel cushions, by using Formulae (76) to (78).

NOTE The formula for the RC columns is based on Takeda-like hysteretic responses, while the formula for the cushions is based on a bilinear behaviour. The elastic damping for the reinforced concrete RC columns is assumed 5 %. If this level of elastic damping is deemed high and a lower value of, say, 2 % is to be adopted, then the coefficients in Formulae (76) to (79) need to be revised.

$$\xi_\mathrm{f} = 0{,}05 + \frac{0{,}565(\mu - 1)}{\mu\,\pi} \tag{76}$$

$$\xi_\mathrm{c} = 0{,}05 + \frac{0{,}519(\mu - 1)}{\mu\,\pi} \tag{77}$$

$$\xi_\mathrm{str} = \xi_\mathrm{f}\,(1 - \beta) + \xi_\mathrm{c}\,\beta \tag{78}$$

6) Reduce the design displacement spectral values by using Formula 79:

$$\eta = \sqrt{\frac{7}{2 + \xi}} \tag{79}$$

7) For the design displacement and by using the over-damped design displacement spectrum, calculate the effective period, T_e.

8) Calculate the effective stiffness, K_e with Formula (80):

$$K_\mathrm{e} = \frac{4\pi^2 m_\mathrm{e}}{T_\mathrm{e}^2} \tag{80}$$

9) Calculate the design base shear of the structure, V_B as in Formula (81):

$$V_\mathrm{B} = K_\mathrm{e}\Delta_\mathrm{d} \tag{81}$$

10) Distribute the design base shear to the cushions and to the frame by using the initially assumed β value.

11) Check the required design shear versus initially assumed yield displacement for the cushions and select a compatible cushion type. If not available, select the closest and conduct only one iteration between steps 3 to 10.

12) Conduct a standard reinforced concrete design procedure for the columns

8.3 Design of dissipative systems connections

8.3.1 General

Between the two extreme solutions of isostatic systems, with their large displacement demand, and integrated systems, with their high force demand, the dissipative systems of cladding connections solution keep displacements and forces within lower predetermined limits.

8.3.2 Structural arrangements

8.3.2.1 General

In this document three structural arrangements are considered, one for vertical panels and two for horizontal panels.

8.3.2.2 Vertical structural arrangements

For vertical panels, starting from the isostatic pendulum system of fastenings of Figure 8 a), a number of dissipative mutual connectors are added between the panels, opposing the relative slide at their vertical joints. Figure 46 shows this solution. The corresponding arrangement matrix is given in Table 12. An equivalent solution can be obtained starting from the isostatic rocking system of Figure 8 c).

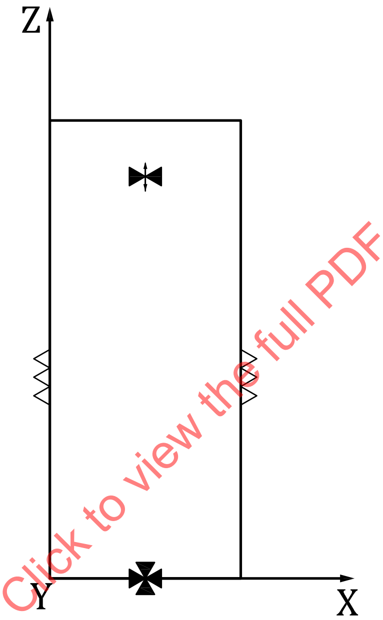


Figure 46 — Vertical dissipative mutual connections setup

Table 12 — Arrangement matrix - vertical dissipative mutual connections

Direction	A	B	C	D	E	F
x	f	/	f	/	d	d
y	f	/	f	/	f	f
z	f	/	s	/	d	d

8.3.2.3 Horizontal structural arrangements

For horizontal panels, starting from the isostatic hanging system of fastenings of Figure 9 a), a number of dissipative mutual connectors are added between the panels, opposing the relative slide at their joints. An equivalent solution can be obtained starting from the isostatic-seated system of Figure 9 b) that can be also used in combination with the former one for the different panels. Figure 47 shows the solution with dissipative connections added between the panels. The corresponding arrangement matrix is given in Table 13.

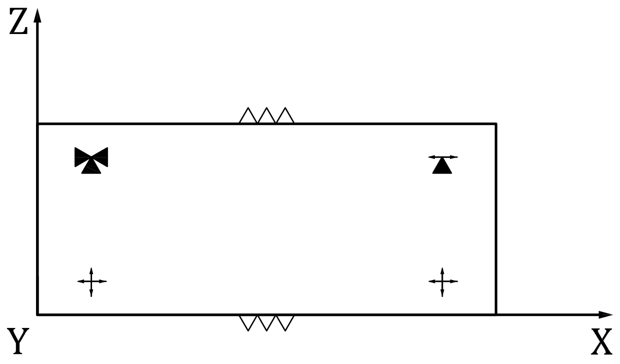


Figure 47 — Horizontal dissipative mutual connections setup

Table 13 — Arrangement matrix - horizontal dissipative mutual connections

Direction	A	B	C	D	E	F
X	s	s	f	s	d	d
Y	f	f	f	f	f	f
Z	s	s	f	f	d	d

The last solution consists of a hanging (or seated) system where two lower corner supports along x are added to the panel with dissipative folded plates attached to the columns. No mutual connectors are placed in this case in the joint between the adjacent panels as shown in Figure 48. The corresponding arrangement matrix is given in Table 14.

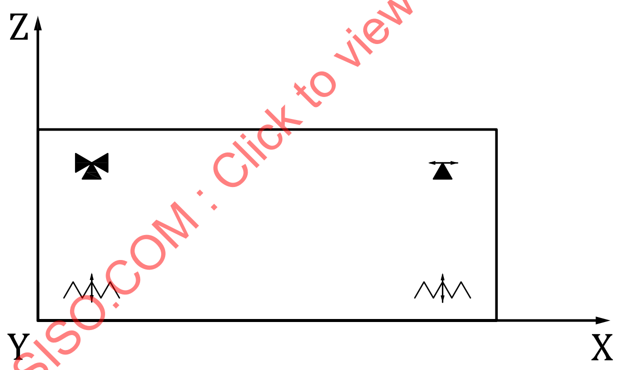


Figure 48 — Horizontal hanging panel setup

Table 14 — Arrangement matrix - horizontal hanging panel

Direction	A	B	C	D	E	F
x	d	d	f	s	/	/
y	f	f	f	f	/	/
z	s	s	f	f	/	/

8.3.3 Friction devices

8.3.3.1 General

The friction devices considered in this document are made of two steel T shape parts that are fixed with a symmetrical set of bolted fasteners to the adjacent panels in special recesses and coupled with two lateral bolted steel plates, as shown in Figure 49 b). The length of the slotted holes made in the web of the T shape profiles gives the limit to the reciprocal slide between the parts. The tightening

