CAPACITY DESIGN CRITERIA FOR CONNECTIONS IN PRECAST STRUCTURES

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ABSTRACT :

The seismic design of precast structures can be based on the standard capacity design criteria provided that a good seismic behaviour of the connections, without early brittle failures, is guaranteed. The knowledge of the seismic behaviour of connections, e.g. in terms of ductility resources, is so far lacking, and capacity design criteria for their proportioning are required. Based on this need, this paper presents, in an overall organized way, the capacity design criteria to be applied to the different types of connections which may be present in a typical precast concrete structure for industrial buildings, when designed for earthquake resistance.

KEYWORDS: Precast Structures, Connections, Seismic Behaviour, Capacity Design.

1. INTRODUCTION

Experience of past earthquakes has shown a general good behaviour of precast structures even in regions of high seismicity. In particular precast frame systems, which are commonly used for one-storey industrial buildings, rely their high resistance capacity on the strong attenuation of the seismic action, due to their long natural vibration period, joined to a large ductile deformation capacity. Lessons learned from previous earthquakes shows that this good behaviour is conditioned by some discriminating design aspects, the most important of which is related to the supports of the beams.

First of all, it is not possible to entrust the seismic force transmission at the supports only to the friction due to gravity loads. Under the combination of horizontal and vertical shakes, the supported element can jolt out of its seating. This concerns the supports both between beams and columns and between floor elements and beams. The dry friction bearing, with interposed rubber pads without any other connector, can provide a support between beams and columns sufficient for static actions, but it must be excluded in seismic zones where mechanical connection devices are needed to transmit the horizontal actions also in the absence of vertical gravity forces.

The supports of the beams in the lateral direction are the other discriminating aspect. These supports shall prevent the possible lateral overturning of the beams which can be induced by the seismic actions. Strong transverse horizontal forces may act together with vertical shakes which decrease the stabilising effects of gravity loads. Therefore, dedicated lateral supports shall be provided to stabilize the beam also in the absence of gravity forces.

A wide dedicated experimental and theoretical investigation has been performed all along the last decade on the seismic behaviour of precast frame structures (Saisi and Toniolo, 1998; Biondini and Toniolo 2003, 2007, 2008; Biondini et al. 2004; Ferrara et al. 2007) and their overall behaviour is now well known and standard methods of analysis (lateral force and modal dynamic with the same force reducing factors as for cast-in-situ structures) can be applied with good reliability. However, this is conditioned by an as good behaviour of the connections, without early brittle failures. Therefore, since the knowledge on the ductility resources of connections is so far lacking, capacity design criteria for the design of connections are required. Based on this need, this paper presents, in an overall organized way, the capacity design criteria to be applied to the different types of connections which are present in a typical precast concrete structure for industrial buildings, when designed for earthquake resistance.

2. CONNECTIONS

In this paper only connections belonging to frame systems are considered, neglecting those employed in wall-panels and cell systems. This choice is representative of the large majority of precast structures. Moreover, only *typical joint systems* are considered, i.e. dry joints consisting of steel connectors, such as angles, plates, channel bars, anchors, fasteners, bolts, dowels and bars. Hence, *emulative joining systems* consisting of wet joints with bar splices and cast-in-situ concrete are not considered. Actually these are not so common in precast construction, and they are not needed for a good seismic behaviour.
On the basis of the location in the building and of the consequent different structural functions, the following six classes of joint systems are distinguished:
1 – *mutual joints between floor (or roof) elements* that, in the seismic behaviour of the structural system, concern the diaphragm action of the floor;
2 – *joints between floor elements and supporting beams* that provide the peripheral constraints to the floor diaphragm in its seismic behaviour;
3 – *joints between beams and columns* that shall ensure a hinged behaviour in the vertical plane of the beam and a full out-of-plane constraint;
4 – *joints between column segments,* consisting of protruding bars anchored in sleeves grouted in-situ (or other devices);
5 – *joints between column and foundation* with the column base fit in a pocket foundation or with protruding bars anchored by bond, as above;
6 – *fastenings of cladding panels to the structure* that shall ensure the stability of the panels but also allow the large drifts expected under seismic action.
Generally speaking, a connection consists of three parts: two “side” parts corresponding to the local regions of the adjacent elements close to the connector, and a central part, which is the properly said connector with its steel components. Usually the side parts have a non-ductile non-dissipative behaviour characterized by brittle failure, with small displacements, due to the tensile cracking of concrete. A ductile dissipative behaviour of the connection can be provided by the central steel connector, if correctly designed for a failure mode which involves flexural or tension-compression modes and not shear modes.
For a ductile connection, in addition to providing a ductile connector, its under-proportioning with respect to the lateral concrete parts is necessary, in the framework of capacity design.
Non ductile connections shall be adequately over-proportioned through capacity design with respect to the strength of the critical dissipative regions of the structure.

3. SUBORDINATE ELEMENTS

In a structural assembly, subordinate elements are those in which the maximum values of the actions are limited by the strength of the critical zones chosen elsewhere to control the failure mechanism of the structure. The ductility of the critical regions is transferred into an as high ductility of the structure provided the subordinate elements are over-proportioned.
Among subordinate elements, the connections shall be considered with special attention. The following clauses present a discussion on specific criteria for capacity design of the different classes of connections, starting from the joints of the main frames (classes 3 and 4 of the above list), following with the joints of the floors and roofs (classes 1 and 2 of the above list) and ending with the joints at the foundations (class 5 of the above list).

3.1. Frame Joints
Figure 1.a shows the typical arrangement of a *column-to-beam connection* of an one-storey precast structure. It consists of a couple of bars protruding from the column, passing through the beam in special holes which are then grouted with mortar. The bars are finally fixed at the top with screwed caps. The beam is set on a pad (steel or rubber) which has to be thick enough in order to preserve the corners of the elements from possible spalling under the large relative beam to column rotation induced by earthquake motion.

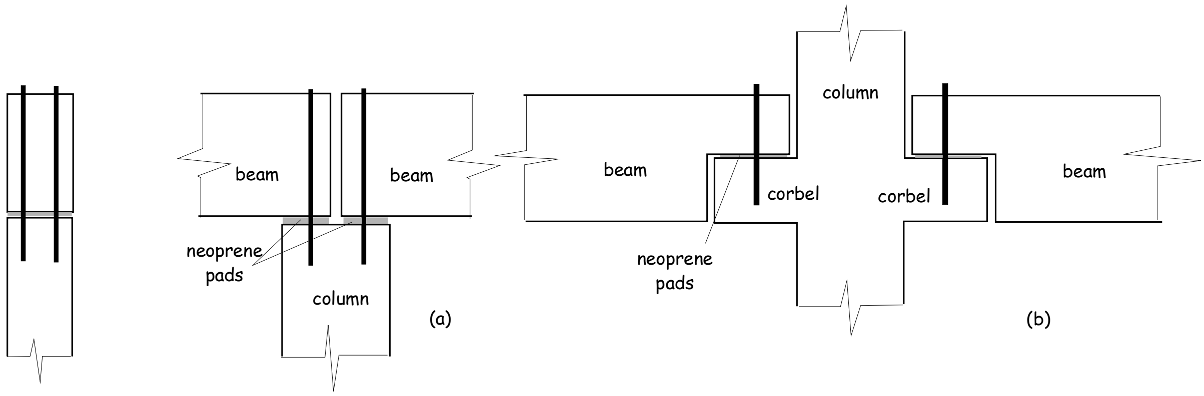


Figure 1: Beam-column connection for roofs (a) and for floors (b).

Together with the full restraint of the three traslatory components of the displacement, the connection should allow an almost free rotation in the vertical plane of the beam, as well as a full restraint of the orthogonal vertical rotation. The couple of bars should also provides a certain degree of restraint to the horizontal rotation.
Vertical forces are transmitted by means of pressure through the pad, up-wards tensions being not expected. Neglecting in seismic condition the gravity force and the consequent horizontal friction contribution, the longitudinal force is transmitted by the shear of the bars. In the orthogonal direction the horizontal force is applied at the top of the beam and gets down to the bearing level together with a bending moment which compresses one side of the pad and tensions the opposite bar. The corresponding shear force can be subdivided between the bars and the compressed part of the pad. Finally the possible horizontal moment can be decomposed into a couple of opposite shear forces in the bars. Secondary stresses, neglected in the design of the connection, can be caused by unintended small longitudinal moment.

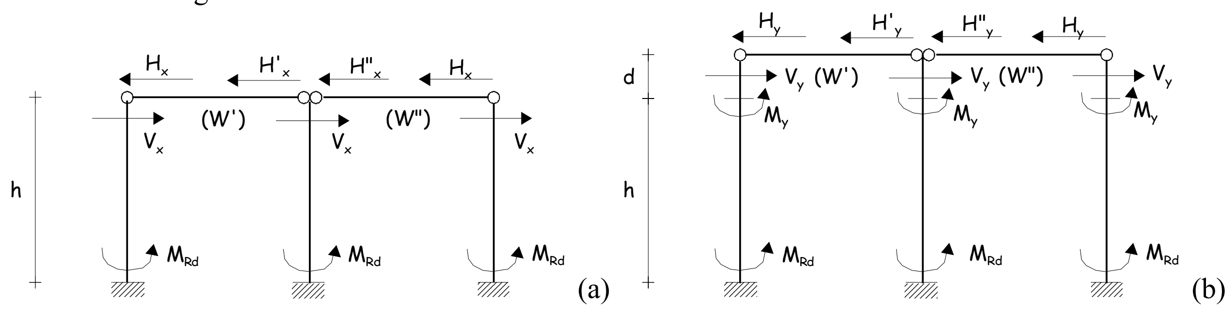


Figure 2: (a) Model for longitudinal shear force. (b) Model for transverse shear force.

Figure 2.a shows the simple model to calculate the longitudinal force in terms of resisting moment M_{Rd} of the critical section at the base of the column:

$$H_x = \gamma_R V_x = \gamma_R M_{Rd} / h$$

where γ_R is a proper confidence factor. For the internal column the shear force $V_x = M_{Rd}/h$ can be subdivided between the two beams proportionally to the respective masses:

$$H'_x = \gamma_R V_x W'/(W'+W'') \qquad H''_x = \gamma_R V_x W''/(W'+W'')$$

In the orthogonal direction a similar model can be adopted (Figure 2.b):

$$H_y = \gamma_R V_y = \gamma_R M_{Rd} / (h+d) \qquad M_y = H_y d$$

where, in case of internal columns with two beams, the shear force $V_y = M_{Rd}/(h+d)$ can be subdivided in the same way proportionally to the respective masses.

Figure 1.b shows the typical *beam-to-column connection* of a *multi-storey* precast structure where the floors are constituted by precast elements without cast-in-situ topping. Often the topping is present and in this case the connection is differently arranged, being possible to rely also on reinforcing ties included in the topping. In the case of Figure 1.b the arrangement is similar to the connection of Figure 1.a, with the necessary adaptations due to the different geometry.
Figure 3 shows the model to calculate the forces at the different floor levels in terms of resisting moment M_{Rd} of the critical section at the base of the column. For the three floors of the quoted Figure, the equilibrium around the base support gives:

$$H_1 z_1 + H_2 z_2 + H_3 z_3 = \gamma_R M_{Rd}$$

With the simplified assumption of a linear increase of the floor forces with the height:

$$H_2 = H_1 z_2/z_1 \qquad H_3 = H_1 z_3/z_1$$

the following forces are obtained:

$$H_1 = \gamma_R M_{Rd} z_1/(z_1^2+z_2^2+z_3^2)$$
$$H_2 = \gamma_R M_{Rd} z_2/(z_1^2+z_2^2+z_3^2)$$
$$H_3 = \gamma_R M_{Rd} z_3/(z_1^2+z_2^2+z_3^2)$$

The linear approximation of the horizontal forces seems to be safe enough for structures with a natural vibration period not greater than 2 sec, provided a model factor γ'_R=1,25 is added (Biondini et al. 2004). Based on the calculation of these forces, the design of the corresponding connections can be done in the same way as indicated above.

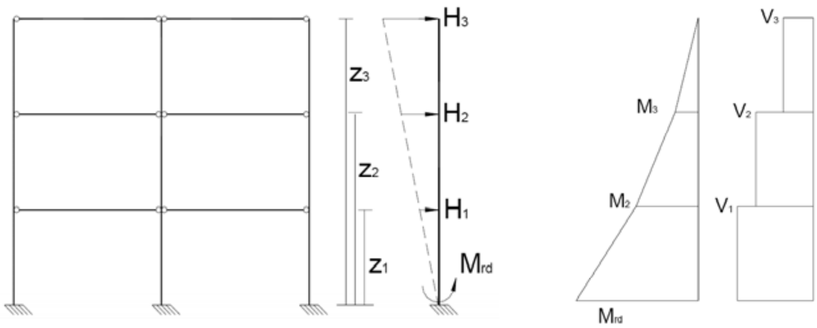


Figure 3: Model for the floor forces.

The same linear model can be used for the calculation of *column-to-column connections* when, for high buildings, a segmental construction is adopted. Figure 4 shows a typical joint made with bars protruding from the upper segment of the column, anchored by bond in sleeves inserted at the top of the lower segment and grouted in-situ.
For the design of the connection the axial, shear and bending components of the internal force shall be computed at the joint level. This can be done on the base of the horizontal forces H_i defined above, through the following equations:

$$V_3 = H_3 = \gamma_R M_{Rd} z_3/(z_1^2+z_2^2+z_3^2)$$
$$V_2 = H_3 + H_2 = \gamma_R M_{Rd} (z_3+z_2)/(z_1^2+z_2^2+z_3^2)$$
$$V_1 = H_3 + H_2 + H_1 = \gamma_R M_{Rd} (z_3+z_2+z_1)/(z_1^2+z_2^2+z_3^2)$$

$$M_3 = H_3 (z_3-z_2) = \gamma_R M_{Rd} z_3(z_3-z_2)/(z_1^2+z_2^2+z_3^2)$$
$$M_2 = H_3 (z_3-z_1) + H_2 (z_2-z_1) = \gamma_R M_{Rd} [z_3(z_3-z_1)+z_2(z_2-z_1)]/(z_1^2+z_2^2+z_3^2)$$
$$(M_1 = M_{Rd})$$

and considering the vertical gravity loads G_i applied at any floor level:

$$N_3 = G_3 \qquad N_2 = G_3 + G_2 \qquad N_1 = G_3 + G_2 + G_1$$

The strength of the connection can be verified with the ordinary equations for the analysis of r.c. sections subjected to eccentric axial force, with special care for the full bond anchorage of the bars in tension. They result again overdimensioned with respect to the flexural strength of the critical sections of the column base.

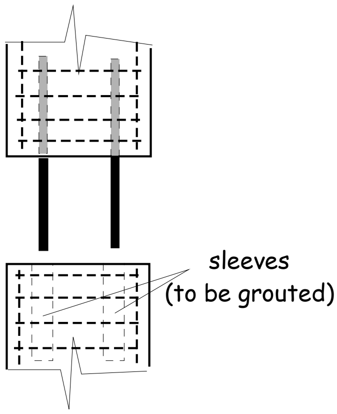


Figure 4: Column-to-column connection.

3.2. Roof Joints

Figure 5 shows the schematic plan of a floor with two bays and three lines of beams. Denoting by n the number of bays, the total seismic force F_h of the floor will be shared by the $n+1$ frames depending on the effectiveness of diaphragm action. In case of the absence of diaphragm action, assuming that the mass associated to each bay is subdivided approximately into two equal parts, the internal frames would have a force $2F=F_h/n$ and the edgel frames would have a force $F=F_h/(2n)$ (Figure 5.a). In case of a rigid diaphragm, assuming the same stiffness for all the frames, the total seismic force is equally subdivided, so that each force turns out equal to $F_h/(n+1)$ (Figure 5.b). Therefore, the diaphragm in-plane shear action transferred from the lateral to the internal frames can be calculated as the difference between the two extreme values (Ferrara and Toniolo 2008):

$$\Delta F = [F_h/(n+1)] - [F_h/(2n)] = F_h\,(n-1)/[2n(n+1)]$$

The higher diaphragm action occurs in the case of two bays (n=2):

$$\Delta F = F_h / 12$$

and, with a safe-side approximation, this could be the design value for all situations.

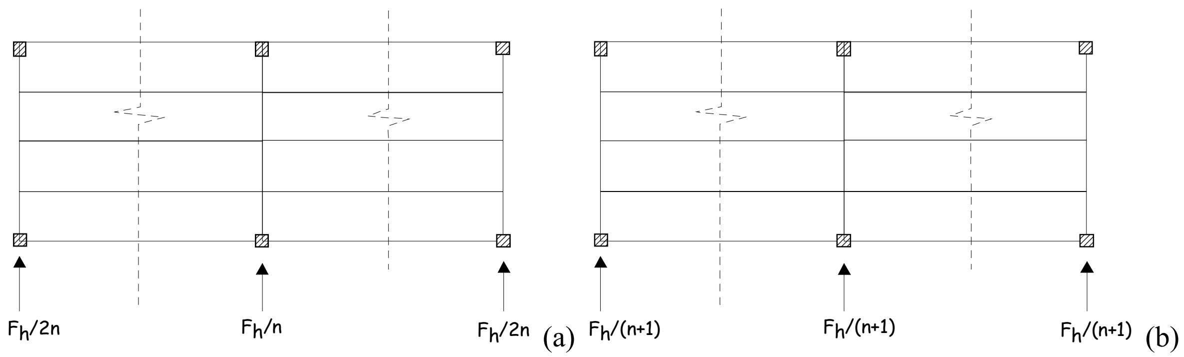


Figure 5: Scheme of a precast floor.

In order to relate the diaphragm actions computed in this way to the flexural strength of the critical zones of the structure, the total seismic force can be calculated with the capacity design criterion as:

$$F_h = \Sigma M_{Rd} / h$$

where the summation is extended to all the columns of the structure, assuming them contemporarily yielded at ultimate condition and applying the linear distribution along the height for multi-storey buildings.

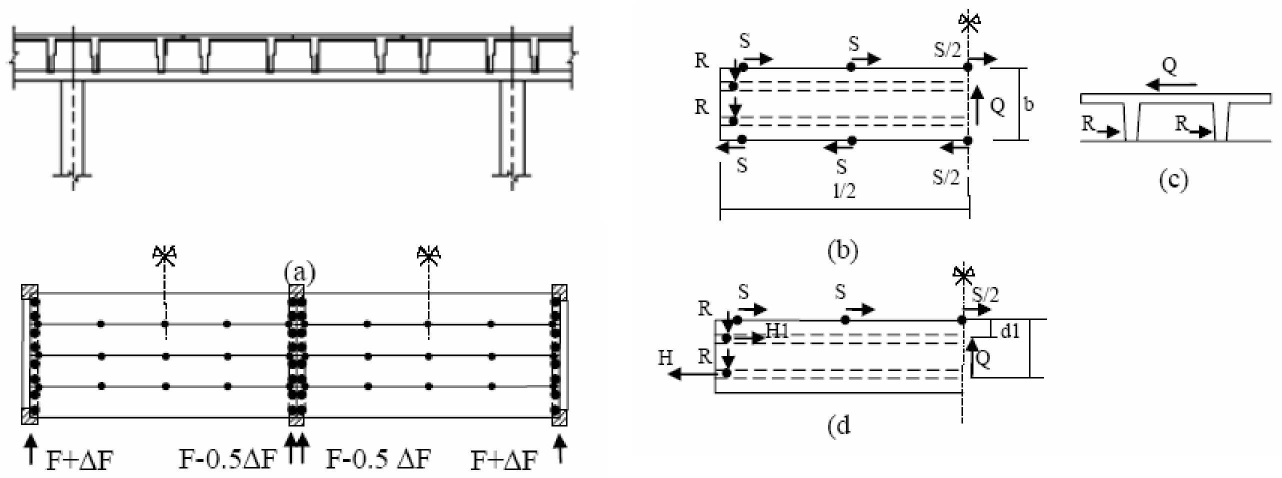


Figure 6: Scheme of a continuous floor.

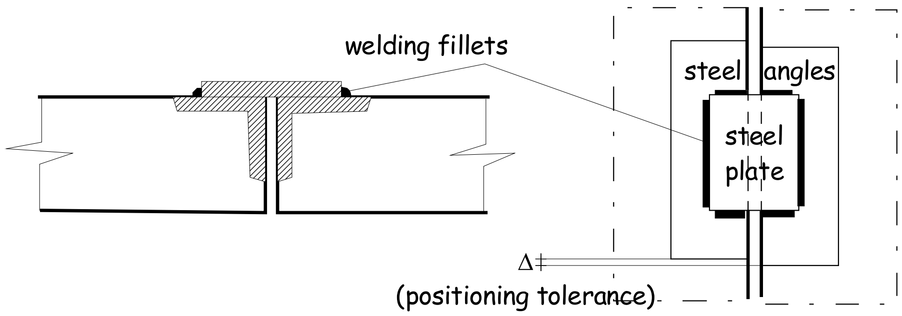


Figure 7: Details of a typical floor-to-floor connection.

For a continuous floor, consisting of precast elements connected with welded point-connections (Figure 6), denoting by m the number of floor elements of one bay, on each single element a diaphragm action force $Q=\Delta F/m$ would act, in addition to its own shore of the inertia force $F_0=F_h/m$. Equilibrated behaviour schemes are indicated in Figure 6.b respectively for an internal and an edge element of the bay. From the first of these schemes the following forces can be computed both for the lateral connections with the adjacent element and for the two end connections with the supporting beam:

$$R = F_0/2 + Q/2$$
$$S = Ql / (kb)$$

where l is the length of the element, b is the its width and k is the number of connections of one edge. For the second scheme, which refers to the end element with one free edge, the following forces can be computed:

$$H_1 = (Q/2)\,(1\text{-}d_2/b) / (b_0)$$
$$H_2 = (Q/2)\,(1\text{-}d_1/b) / b_0$$

where d_1 and d_2 indicate the distance of the two end supports from the internal edge and $b_0=d_2-d_1$ is their distance. These schemes save the force equilibrium and not the deformation compatibility and would require, to compensate the inaccuracy of this calculation, an adequate ductility of the connections, which may be difficult to obtain. A parametric investigation has been undertaken to assess the level of reliability of such a model (Ferrara and Toniolo 2008). Figure 7 shows the details of a floor-to-floor typical connection.

For a discontinuous roof, consisting of precast elements spaced to allow the positioning of skylights (Figure 8), assuming a double support on the beam able to restrain the horizontal relative rotation, the equilibrated scheme is represented in Figure 8.b. The two components of the reaction can be calculated with:

$$R = F_0/2 + Q/2$$
$$H = Q\,l\,/\,(2b_0)$$

The floor-to-beam connections can be designed to the above said forces in order to ensure, also for this type of discontinuous roofs, a diaphragm behaviour which in general is sufficient for a controlled response of the structure. Figure 9 shows the details of a roof-to-beam typical connection.

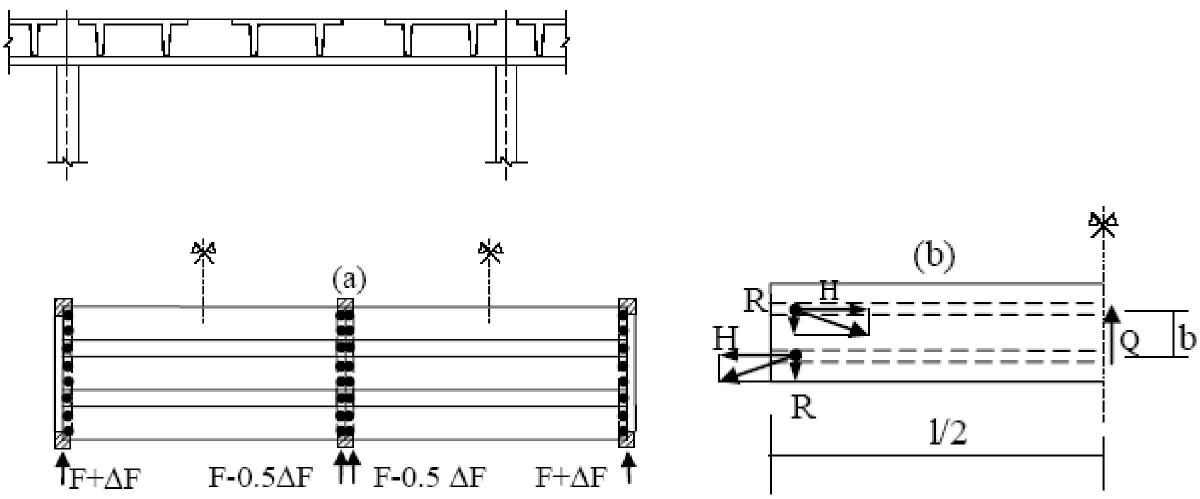


Figure 8: Scheme of a discontinuous floor.

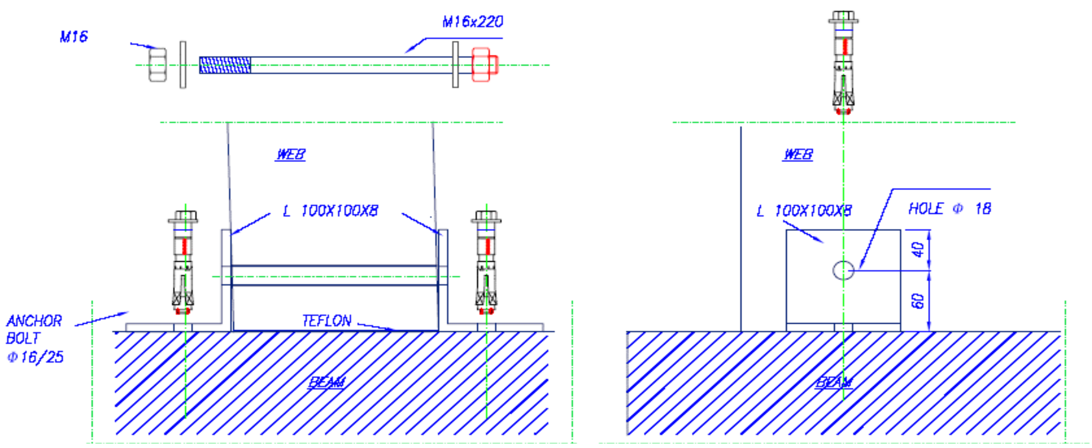


Figure 9: Details of a typical roof-to-beam connection

3.3. Foundation Joints

The typical precast socket foundation, which guarantees a perfect constraint of the column base, is shown in Figure 10.a. According to the capacity design, it has to be designed with reference to the actions shown in Figure 10.a, where:

$$M = \gamma_R M_{Rd} = \gamma_R M_{Rd} (N_{Ed})$$
$$V_{Ed} = \gamma_R M_{Rd}/h$$
$$N = N_{Ed}$$

where h is the column height and γ_R a suitable confidence factor.
When the dimensions of this type of foundation turn out too large, the alternative system shown in Figure 10.b can be used: in it the longitudinal bars of the column are extended outside for a suitable length to be inserted into sleeves into the foundation slab, which will be grouted after. Bars protruding from the foundation slab and inserted into ducts to lap with the main column reinforcement after grouting may also be used. Bolted connections are also employed: the column reinforcement is connected to a base plate, to which are screweb threaded bolts anchored into the foundation and, generally, accommodated into column corner recesses (Figure 10.c). Devices through which such a kind of connection can be detailed are available and experimental and numerical investigation are currently on going to assess the reliability of the flexural constraint they are able to provide (di Prisco et al., 2008).

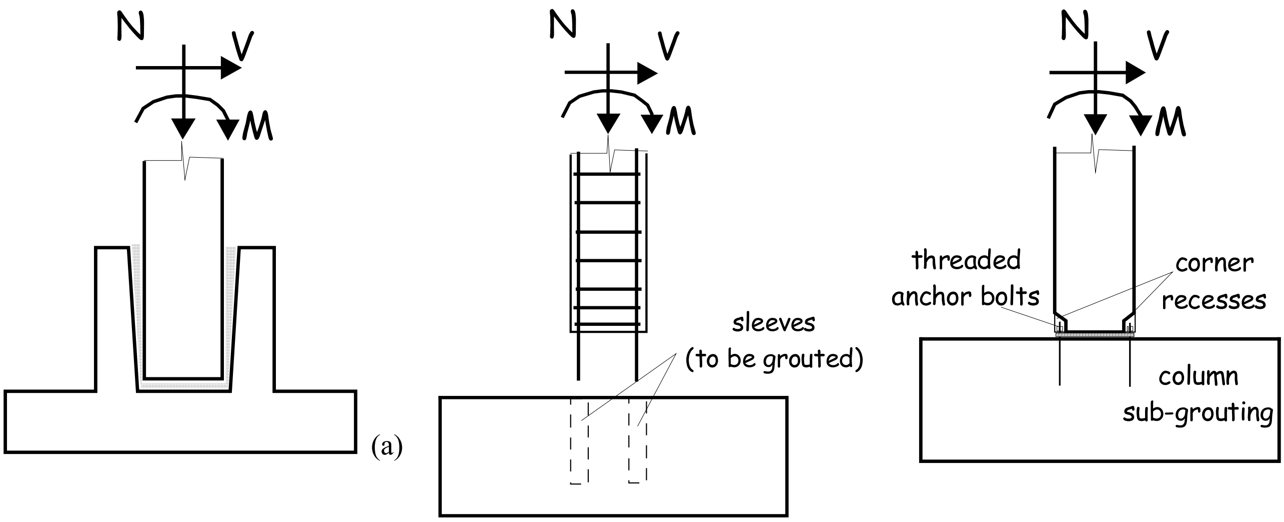


Figure 10. Details of typical column to foundation connections.

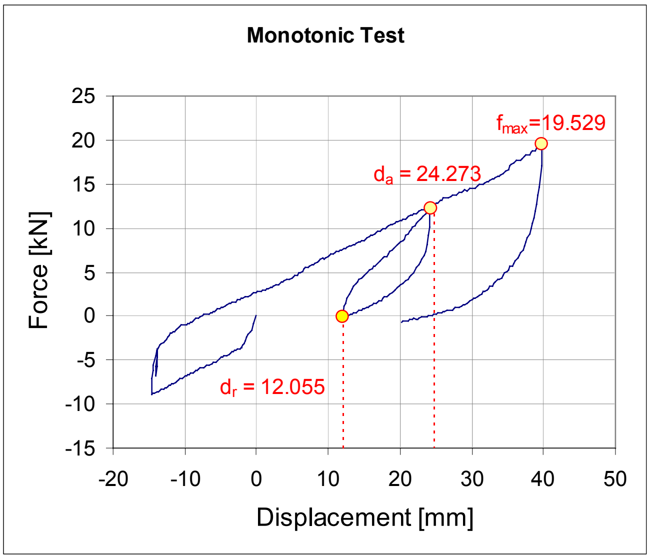
4. EXPERIMENTAL PARAMETERS

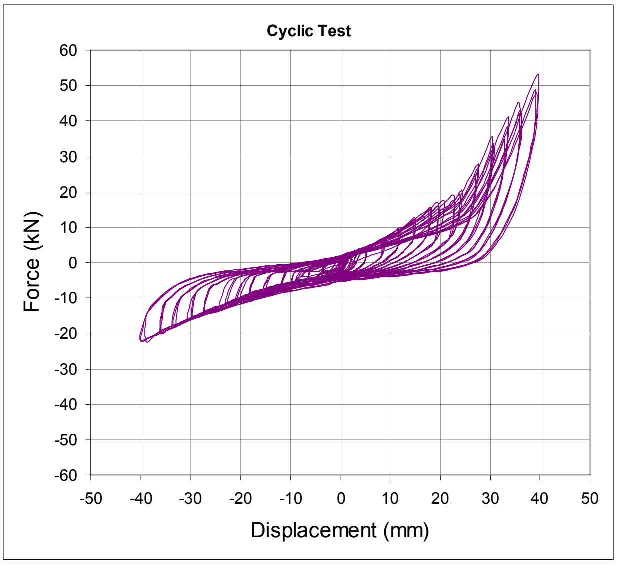
For the seismic design of a precast structure, with the application of capacity design to subordinate connections, their pertinent behaviour parameters shall be available. These parameters shall be determined by testing on the base of a proper standard protocol (Felicetti et al. 2008). For this, a wide European research has started joining the associations of SME (Small and Medium Entreprises) of the principal seismic European countries.
The principal parameters which characterize the seismic behaviour of the connections refer to the six properties of:

- *resistance* as maximum value of the force, f_{max}, which can be transmitted between the parts;
- *ductility* as the ratio between the ultimate and the yield deformation;
- *dissipation* as specific energy, U_i/U_{0i}, dissipated through the load cycles;
- *deformation* as ultimate deformation d_u at failure limit;
- *decay* as strength fall through the load cycles;
- *damage* as residual deformation d_r at unloading.

An example of experimental determination of these parameters is reported in Figure 11 with reference to the roof-to-beam connection shown in Figure 9 (Felicetti et al. 2008), for both monotonic and cyclic tests: an exceprtum of a table showing the most relevant parameters which can be identified from experimental tests is also shown where:

- d_i and f_i represent the maximum displacement and the corresponding value of the force for each half-cycle;
- $u_i = U_i/U_{0i}$ is the specific energy dissipation along each half-cycle;
- D_{sp} represents the specific decay, calculated with reference to the maximum value of the forces measured for each series of three nominally identical half-cycles.





	d_i	f_i	d_{ei}	d_{pi}	U_{oi}	U_i	u_i	D_{sp}
- 24 - 1	-23.8	-12.1	1.8	22.0	2662	1750	0.66	
- 24 - 2	-22.9	-11.2	1.9	21.0	2352	1372	0.58	
- 24 - 3	-22.2	-10.9	1.7	20.5	2235	1460	0.65	0.10
+ 24 - 1	24.5	20.1	3.5	21.0	4212	2060	0.49	
+ 24 - 2	24.0	19.1	3.0	21.0	4020	1724	0.43	
+ 24 - 3	22.9	17.5	0.9	22.0	3844	1581	0.41	0.13

Figure 11: Identification of the parameters characterizing the behavior of a connection from monotonic and cyclic tests.

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