SAFECAST Project: European research on seismic behaviour of the connections of precast structures

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SUMMARY:
SAFECAST Project is the last of a long series of co-normative researches that supported the standardisation of precast structures within Eurocode 8. The paper presents the activity performed by the 16 European partners. The seismic behaviour of four classes of connections is investigated: floor-to-floor, floor-to-beam, beam-to-column and column-to-foundation. The experimental qualification is made in terms of strength, ductility, dissipation, deformation, decay and damage. More than 100 cyclic and dynamic tests have been performed in the laboratories of Lisbon, Milan, Ljubljana, Athens and Istanbul. But the most relevant series of tests have been performed at ELSA Laboratory of the JRC of Ispra, where a full-scale prototype of a three storeys precast structure has been subjected to pseudodynamic and cyclic tests. Other authors are presenting the details of any specific testing activity. This paper presents the design criteria deduced from these activities, as collected in the final document of design rules.

Keywords: precast structures, connections, cyclic & dynamic tests.

1. INTRODUCTION

The first testing campaign has been carried out on 1994 when a first draft of Eurocede 8 has been prepared with a section dedicated to precast structures. To make up for the lack of experimental data, a number of cyclic and pseudodynamic tests has been performed on precast columns in pocket foundations at ELSA Laboratory of Ispra (see Figure 1). These tests gave the required demonstration that precast columns behave very well, like the corresponding cast-in-situ ones, but also better because there are no bar splices and no danger of stirrup packaging due to their horizontal casting position (see Saisi & Toniolo 1998).

In the typical arrangement of one storey industrial buildings (see Figure 2), the role of the hinged connections had to be investigated in comparison to the monolithic joints proper of cast-in-situ construction. This has been done first with non linear dynamic numerical comparative simulations made on two similar prototypes of one storey structures, one precast and the other cast-in-situ, designed with the same base shear resistance (see Biondini & Toniolo 2002).

The experimental verification of the numerical results came from the pseudodynamic tests performed on 2002/2003 again at ELSA Laboratory of Ispra within an European Ecoleader Programme. Two full scale prototypes of one storey structures (see Figures 3a-b) with the same base shear resistance have been submitted to the same accelerogram. The results gave the expected demonstration that the two arrangements have the same seismic capacity: the cast-in-situ structure in its more numerous critical regions dissipates the same amount of energy dissipated by the precast structure in its fewer and larger critical regions dimensioned as they are for higher moments. It is the global volume involved in dissipation and not the number of plastic hinges that gives the measure of the energy dissipation (see Biondini & Toniolo 2004).

These results have been taken to the competent CEN Technical committee that, on 2004 in Vienna under the presidency of prof. Fardis, approved the final version of Eurocode 8 with the full equalisation between precast and cast-in-situ frame systems

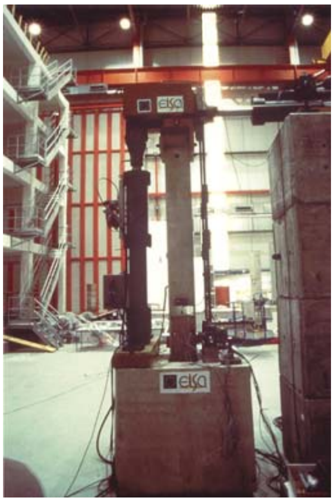


Figure 1: Assobeton tests 2004/06
20 cyclic and 8 PsD tests

Figure 2: precast structure of industrial building
in erection stage

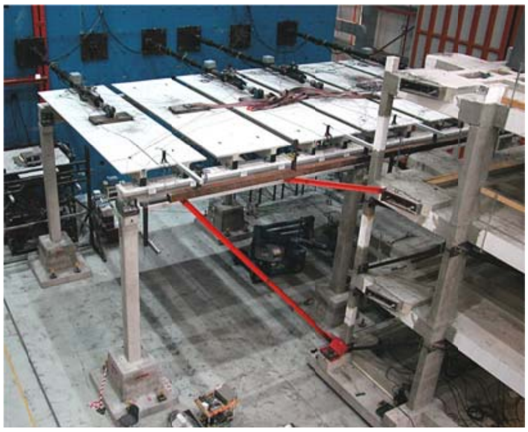
(a) (b)

Figure 3: Ecoleader PsD tests 2002/03 on full scale prototypes: precast (a) and cast-in-situ (b)

In the meantime BIBM, the international association of prefabrication industry, had expressed its interest on the research, so that an other wider testing campaign has been launched within the European Growth programme. “Precast structures EC8” concerned in general the seismic behaviour of precast concrete structures for industrial buildings with respect to Eurocode 8 and was supported by ten partners from Italy, Portugal, Slovenia and Greece. Also the Tongji University of Shanghai participated:

Figures 4a-b show the images of the full scale prototypes tested at ELSA Laboratory of Ispra with the pseudodynamic procedure. But also in Lisbon, Athens and Shanghai important tests have been performed. The role of cladding panels has been investigated together with the effectiveness of the diaphragm action. The quoted PsD tests are reported in Biondini, Toniolo & Zaho 2008. A more exhaustive interpretation of results is presented in Biondini & Toniolo 2009.

(a)



(b)

Figure 4: Precast structures EC8 2003/06 – full scale prototypes ready for PsD tests
(a) with cladding panels (b) turned roof arrangement

2. TESTS ON CONNECTIONS

Through their very large crop of results, the previous researches pointed out the importance of the connections on the overall behaviour of the structures. This was a problem still pending with a general lack of knowledge. So a new project has been launched within the scope of the European FP7-SME-2007-2 Programme.

Safecast project is at present the last of the research campaigns. It started on 2009 with the participation of 16 partners coming from Portugal, Spain, Italy, Slovenia, Germany, Greece and Turkey:

Associaions	*Research providers*	*Users (producers)*
1. Assobeton (I – coordinator)	6. EC DG JRC – Elsa Lab.	14. DLC (I)
2. Andece (E)	7. Politecnico di Milano	15. Rphorsa (E)
3. Anipb (P)	8. Nat. Techn. Univ. of Athens	16. Halfen (D)
4. Sevips (GR)	9. Istanbul Techn. University	
5. Tpca (TR)	10. LNEC of Lisbon	
	11. Univ. of Ljubljana	
	12. Labor (I)	
	13. Lugea (I)	

It has been closed on March 2012. More than 4 million Euro has been the cost, partly supported by the European Commission.

A large number of cyclic and dynamic tests have been performed on sub-assemblies of structural elements connected at their joints. Figure 5 shows a beam-to-column connection placed on the shaking table of the Technical University of Athens. Figure 6 shows a different type of beam-to-column connection placed in an overturned position in the laboratory of the Istanbul Technical University. Figure 7 shows the prototype of a column-to-foundation connection installed in the testing plant of the Politecnico di Milano. Figure 8 shows a frame modulus placed in the National laboratory of civil engineering of Lisbon and finally Figure 9 shows the laboratory of the University of Ljubljana where additional tests on beam-to-column connections have been made.

Figure 5: Beam-to-column joint
NTUA Lab. – Athens

Figure 6: Beam-to-column joint
ITU Lab – Istanbul

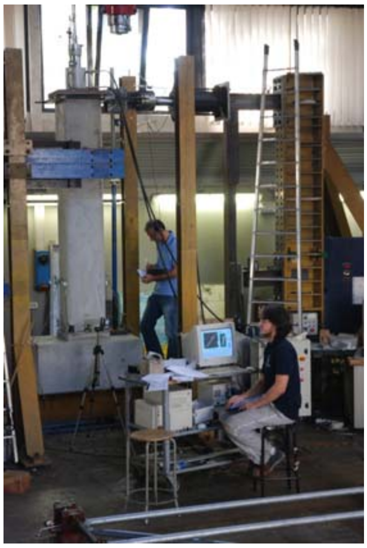


Figure 7: Column-to-found. joint
POLIMI Lab. – Milan

Figure 8: Frame modulus
LNEC Lab. – Lisbon

Fig.ure 9: Beam-to-column joint
UL Lab. – Ljubljana

More than one hundred of such tests have been performed in all in the quoted laboratories providing a large data-base that is now the basis for any possible improvement of the related technology and design. Following the due interpretations a manual containing the Design rules for connections of precast structures has been drafted as presented in Chapter 3.

But the most important series of tests has been performed between June and August 2011 at ELSA Laboratory of Ispra. Figure 10 shows the full scale prototype of three storeys structure installed against the reaction wall of that laboratory at an advanced stage of erection, while Figure 11 shows the same prototype completed with all its components. The dimensions are about 16 by 16 m in plan and more than 10 m in elevation. It is the bigger prototype ever tested in that laboratory and one of the biggest ever tested in the world. From the experimentation a complete information has been obtained about the seismic behaviour of this type of structures in terms of reliability of the analysis, displacement control and effectiveness of connections system, as reported by Negro, Bournas & Molina 2012 and Bournas & Negro 2012.

Figure 10: Three storey prototype in erection
ELSA Lab. – Ispra

Figure 11: Three storey prototype completed
ELSA Lab. – Ispra

3. DESIGN RULES FOR CONNECTIONS

The main supply delivered to Safecast purchasers (that is the five national associations of small and medium enterprises) is the manual containing the "Design rules for connections of precast structures". The purpose of this document is to give simple design rules for practical applications, drafted in the format of a manual for designers. The rules of this manual have a theoretical derivation supported by the experimental results of the testing campaign and by the numerical simulations performed within the research. General know-how on production practice and international literature on the matter have been also considered.

The rules refer to the connections in precast frame systems of one-storey and multi-storey buildings. Four orders of connections are considered: floor-to-floor, floor-to-beam, beam-to-column and column to foundation. Different types of connections are treated for any order belonging to the three main systems of typical joints, that is dry joints with mechanical connectors, emulative joints, that is wet joints with rebar splices and cast-in-situ concrete, and mechanical joints, that is joints with bolted flanges as used in steel construction. Simple bearings working by gravity load friction are not considered. Sliding and elastic deformable supporting devices neither, being all these types of connections not suitable for the transmission of seismic actions.

Some fundamental requirements give the basic design criteria to which the document is shaped:
"Any type of connection shall be experimented with an initial type testing in order to quantify its strength and possibly the other properties that affect its seismic behaviour. From this experimentation a design model may be deducted, by means of which a verification by calculation can be applied on the different connections of the same type. For a specific application one can refer to the available results of previous experimentations like those provided in this document or in other reliable documents such as official regulations (Eurocodes, CEN product standards and CEN Technical specifications, …). "
"Non ductile connections can be used provided they are opportunely over-proportioned by capacity design with respect to the resistance of the critical dissipative regions of the structure."

The main parameters that characterize the seismic behaviour of the connections, as measured through monotonic and cyclic tests, refer to the six properties of:
-*strength*, that is the maximum value of the force which can be transferred between the parts;
-*ductility*, that is the ultimate plastic deformation compared to the yielding limit, where the plastic deformation can be replaced by other physical phenomena like friction;
-*dissipation*, that is the specific energy dissipated through the load cycles related to the correspondent perfect elastic-plastic cycle;

-*deformation*, that is the ultimate deformation at failure or functional limit;
-*decay*, that is the strength loss through the load cycles compared to the force level;
-*damage*, that is the residual deformation at unloading compared to the maximum displacement or the details of rupture.

For any single type of connection strength is specified with the definition of:
-the *behaviour models* related to the resisting mechanisms of the connecion;
-the *failure modes* of the resistant mechanisms;
-the *calculation formulae* of the ultimate strength for any failure mode.

For what concerns ductility the connections are classified on the basis of the force-displacement diagrams obtained from the experimentation:
-*brittle connections* for which failure is reached without relevant plastic deformations;
-*over-resisting connections* for which, at the functional limit, failure has not been reached:
-*ductile connections* for which a relevant plastic deformation has been measured.
Ductile connections are again classified in:
-*high ductility* with a displacement ductility ratio of at least 4,5;
-*medium ductility* with a displacement ductility ratio of at least 3,0;
-*low ductility* with a displacement ductility ratio of at least 1,5.

This classification of ductility refers to the connection itself. This ductility has not direct reference to the global ductility of the structure. Ductile connections may give or not a relevant contribution to the energy dissipation at the no-collapse limit state of the structure depending on their location within the structural assembly and on their relative stiffness. In general all the connections, ductile or not ductile, shall be over-proportioned by capacity design with respect to the critical regions of the structure.

For what concerns dissipation the connections are classified on the basis of the specific histograms obtained from the experimentation:
-*non dissipative* with specific values of dissipated energy lower than 0,10;
-*low dissipation* with specific values between 0,10 and 0,30;
-*medium dissipation* with specific values between 0,30 and 0,50;
-*high dissipation* with specific values of dissipated energy over 0,50.
Medium dissipation corresponds to well confined reinforced concrete sections under alternate flexure and high dissipation can be achieved with the use of special dissipative devices.

For a direct comparability of the results, the quantification of the properties quoted above has been carried out by means of tests performed following the procedures described in a special Protocol for connection testing. From monotonic (push-over) tests the first information about the yielding limit, the maximum force, the ultimate deformation and the ductility ratio is obtained. From cyclic tests, performed following a standard loading history, information about the strength variation, the force decay and the energy dissipation is obtained.

3.1 Beam-to-column dowel connection

Among the different types of beam-to-column connections the qualification of a traditional wide spread one is presented. Figure 12 shows this type with the details of a beam placed on a supporting column. In the case (a) two dowels protrude from the top of the column and enter into the sleeves inserted in the beam. The sleeves are filled with no-shrinkage mortar of adequate strength to ensure by bond the anchorage of the dowels. The anchorage can also be ensured providing the dowels with a cap fixed at the top with a screwed nut. In any case the sleeve shall be filled in with mortar to avoid hammering under earthquake conditions. The case (b) refers to the same technology but with only one dowel. In the transverse direction the use of two dowels improve the resistance against overturning moments. Due to the much lower stability against overturning moments the use of one only dowel is not recommended especially with reference to the uneven load conditions during the construction stages. The beam usually is placed over a pad to localise the load out of the peripheral edges of the connected elements. The rules given in the following clauses are based on tests made only on connections with flexible rubber pads and adherent dowels (see Psycharis & Mouzakis 2012).

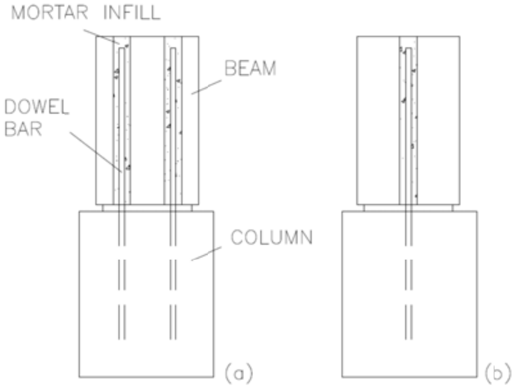


Figure 12: beam-to-column connection with two dowels (a) or one dowel (b)

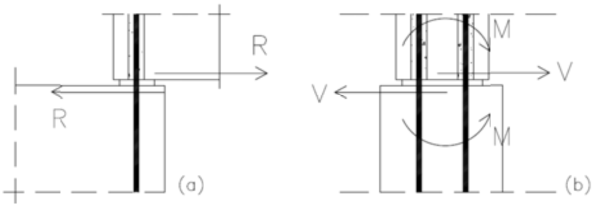


Figure 13: behaviour models for longitudinal (a) and transverse (b) action

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

This type of connection provide an hinged support in the vertical plane of the beam and a full support in the orthogonal vertical plane. In the longitudinal direction of the beam the horizontal force *R* is transmitted through the shear resistance of the connection (see Figure 13a), which is given by the shear resistance of the dowels and their local flexure between the elements in correspondence of the bearing pad. In the transverse direction, omitting the vertical gravity loads, the connection transmits a shear force *V* together with the corresponding moment *M* (see Figure 13b).

The principal failure modes are listed below together with the corresponding verification formulae. With reference to Figure 13a for the longitudinal action of a given force *R* evaluated by capacity design, as deduced from tests performed on connection with flexible pads, the following verifications shall be made:
a – breaking of the dowel connection due to combined shear, tension and flexure on steel bar and bearing stresses on concrete;

$$R_{Rd} = 0{,}90\ n\ \phi^2 \sqrt{[\ f_{yd}\ f_{cd}\ (\ 1 - \alpha^2\ )\ ]} \geq R$$

with n number of dowels, ϕ diameter of dowels, f_{cd} design strength of concrete, f_{yd} design strength of steel, $\alpha = \sigma / f_{yk}$ with σ normal tensile stress due to other possible contemporary effects on the dowel;
b/c – spalling of the concrete edge of the beam or of the column due to tensile stresses;

$$R_{Rd} = \kappa\ 1{,}6\ \phi^{\alpha}\ h^{\beta} \sqrt{(f_{ck,cube}\ c^3)} / \gamma_C \geq R \qquad \alpha = 0{,}1\ (h / c)^{0,5} \qquad \beta = 0{,}1\ (\phi / c)^{0,2}$$

where $f_{ck,cube}$ is the characteristic cubic strength of concrete, ϕ is the diameter of the dowel, c is the edge distance of the dowel axis, $h=8\phi$ is the effective length of the dowel and $f_{ck,cube}$ is expressed in N/mm^2, R and R_{Rk} in N and d, h, c, ϕ in mm.

With reference to Figure 13b, for the transverse action of a force V and a moment M evaluated by capacity design, the following verifications shall be made:
d – flexural failure of the bearing section due to the action of M:

$$M_{Rd} = A_s f_{yd} z \geq M$$

with *z* lever arm of the internal forces and A_s sectional area of the dowels;
e – pull-out of the tensioned dowel under the action due to *M*:

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

where f_{bd}=0,45f_{cd} is the ultimate bond strength, f_{ym}=1,08f_{yk} is the mean yielding stress of the steel, l_b is the anchorage length of the dowels, A_s is the sectional area of a dowel and *u* is its perimeter;
f – sliding shear failure under the action of *V*:

$$V_{Rd} = 1{,}3 A_s \sqrt{(f_{cd} f_{yd})} + 0{,}25 b x f_{cd} \geq V$$

where A_s is the sectional area of the dowels not yielded by flexure and *b* and *x* are the width and the depth of the compressed part of the concrete section.

In testing, failure mode *a* of dowel connection displayed a local shear ductility, due to the flexural and tensional deformation of the dowels within the joint gap, evaluated in μ=4,0 to 6,0. For $c/\phi<6$, with a failure mode *b* or *c*, the ductility decreased to μ=2,5 to 3,5. Cyclic tests performed in the longitudinal direction of the beam showed a *medium dissipation* capacity due to the alternate deformations of the dowels within the joint gap. In any case, due to their location in the structural assembly and to their high stiffness in comparison to the column flexibility, no contribution of ductility and dissipation are expected from this type of connections to the global ductility of the structure. They shall be over-proportioned by capacity design with respect to the critical sections of the column base.

3.2 Foundations with bolted sockets

Figure 14 shows a new type of connection of a column to the foundation obtained with steel sockets inserted in the column base and bolted to the foundation. The sockets are anchored to the column by means of couples of bars welded to them and spliced to the current longitudinal reinforcement by overlapping. Other transverse links can be welded to the sockets to avoid their lateral detaching. At the lower part of the connections, stud-bolts are protruding from the foundation, one for any socket. They consist of headed fasteners of adequate length previously inserted in the foundation element. A mortar embedding fills the lower joint in.

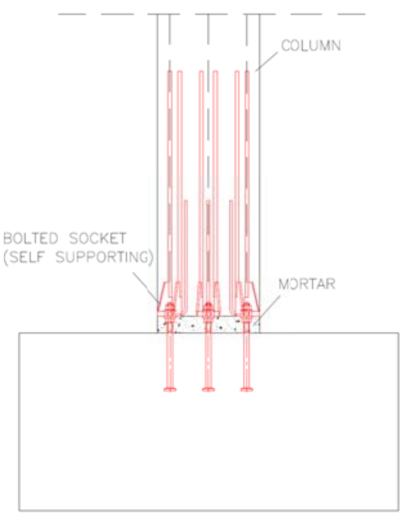


Figure 14: column-to-foundation with bolted sockets

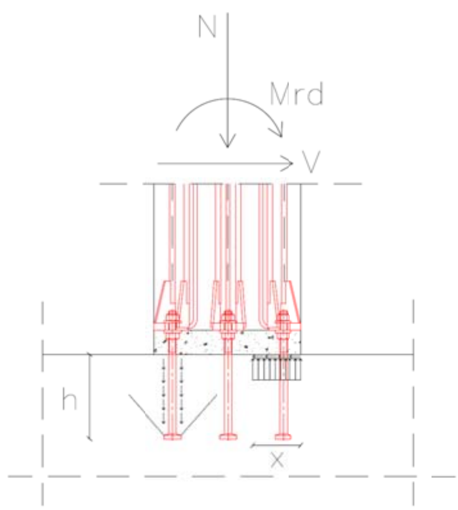


Figure 15: design model of the resisting mechanism

In expectation, under seismic conditions, of a plastic hinge at the base of the column, the length of this plastic hinge finds some difficulties to be determined because of the uncertain effectiveness of the longitudinal reinforcement in the lap zone of the bars. In any case the formation of the plastic hinge in a raised position over the lap length shall be avoided since for this position the displacement ductility of the column would be reduced. More reliable results and possibly a higher displacement ductility can be obtained moving upwards the lap zone so to leave a sufficient length of single (non overlapped) reinforcement at the base of the column, provided these lower bars are weaker and connected to the sockets with proper provisions that don't endanger their ductility.

The connection shall be verified for the action of the (plastic) ultimate moment $M_{Rd}=M_{Rd}(N)$ at the base of the column with the correspondent contemporary axial force N and of the shear V. This calculation can be performed in the two main directions independently. The due overstrength factor γ_R shall be added as specified hereunder. The lap length of the lower bars with the upper bars of the column shall be overproportioned applying the same factor γ_R and this calculation is taken for granted in the following points. Due to their expected brittle failure modes, in general terms for a good ductile behaviour the local devices (sockets, bushes, bolts,…) with their coupling provisions (welding, threading, pressing,…) shall be over-dimensioned by γ_R with respect to the connected elements to which a ductile behaviour is required.

Figure 15 shows the detail of the resisting mechanism of the foot section of the column subjected to the combined bending moment $\gamma_R M_{Rd}$ and axial action N and to the shear $\gamma_R V$. Assuming that at this level of action the tensioned lower steel bars are at their maximum ultimate capacity F_u, the anchorage verification shall be referred to a correspondent pull-out force. The failure modes are listed hereunder together with the corresponding verification formulae:

For fasteners well spaced among them and from the foundation edges, with reference to the symbols described in Figure 15 and with γ_R overstrength factor, the following verifications shall be performed.
a – failure of the fastener subjected to the tensile force coming from the upper reinforcement:

$$F_{Rmin} \geq \gamma_R A_s f_{ym}$$

where F_{Rmin} is the minimum steel ultimate capacity of the fastener declared by the producer, A_s is the sectional area of the corresponding upper reinforcement, $f_{ym}=1{,}08f_{yk}$ is the mean yielding stress of the steel bars.
b – pull-out of the head-fastener subjected to the maximum upper force F_u with concrete cone-failure:

$$k \sqrt{(f_{ck,cube}\, h^3)} / \gamma_C \geq \gamma_R F_u$$

where $F_u=\min\{A_s f_{ym} , F_{Rmax}\}$, $F_{Rmax}=1{,}2F_{Rmin}$ and for current products k=7,0 may be assumed.
c – sliding shear failure at the foot section in the design situation corresponding to $\gamma_R M_{Rd}$, N and $\gamma_R V$:

$$1{,}5\, A_d \sqrt{(f_{cd}\, f_{yd})} + 0{,}25\, b\, x\, f_{cd} \geq V$$

where V is the shear corresponding to $\gamma_R M_{Rd}$, A_d is the sectional area of the dowel not yielded by the moment, f_{yd} is design strength and b and x are the width and the depth of the compressed part of the concrete section.

Tests have been performed on different prototypes with different arrangements of the connection showing different ductility capacities (see Dal Lago, Lamperti & Toniolo 2012). Some early failure occurred due to the rupture of defective welding of the socket, pointing out the importance of a correct coupling technology. When a correct coupling is made, the arrangement of weak bars under the lap zone moved in an upper position can save the full "medium" ductility and dissipation capacity of the column.

4. CONCLUSIONS

The work done in Safecast Project allowed to achieve a good knowledge on the behaviour of the connections of precast structures, enabling to have a reliable design under seismic action. All the rules given by the specific manual for the calculation of the resistance are based on the assumption to apply the capacity design criterion for the calculation of the action. In some cases the application of capacity design for the proportioning of the connections is simple and immediate: with reference to the hinged beam-to-column connections of one storey structure, the horizontal force at the top of the columns can be calculated from the resisting moment M_{rd} of the section at the critical region at the base of the columns with $H=\gamma_R M_{rd}/h$, where h is height of the column and γ_R is the due overstrength factor. For multi-storey structures the equilibrium around the base support gives $H_1z_1+H_2z_2+H_3z_3+\ldots=\gamma_R M_{rd}$ and the problem remains indeterminate, depending on the ratio between the storey forces H_i that are applied at the different levels z_i. Some approximate solutions are proposed in Biondini, Tsionis & Toniolo 2010 and Fischinger, Rejek. & Isakovic 2010. Also indeterminate remains the distribution of the diaphragm forces transmitted among the floor elements through their connections. A solution is suggested by Ferrara & Toniolo 2008 with reference to the roofs of one storey structures. But an inadequate approach is still now applied to the design of the fastening systems of cladding panels as pointed out by lesson of recent earthquakes (see Colombo & Toniolo 2012). This is a pending problem on which the research shall be addressed in the future years.

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