RESEARCH INVESTIGATIONS ON THE SEISMIC PERFORMANCE OF PRECAST CONCRETE INDUSTRIAL BUILDINGS IN EUROPE

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ABSTRACT

Precast prestressed concrete systems had successful application in earthquake resisting buildings in many parts of the world. However, poorly designed details of connections between the structural members can drastically compromise the overall performance of these structures; in fact, during recent severe South-European earthquakes, low-rise multi-storey precast industrial buildings designed according to the current codes have been extensively damaged.

These industrial / commercial buildings are generally limited to two-three storeys and most of them adopt as structural configuration hinged frames in both directions. In order to increase speed of construction, buildings are totally prefabricated using precast concrete structural members and mechanical dry connections. The connections between the structural members are designed for transferring shear only (hinge), without providing supplemental dissipation energy to the building. This design approach is adopted in the current practice and leads inevitably to accept permanent structural damage in the columns, i.e. significant post-earthquake cost of repairs.

The research community is aware of the limitations of the current design philosophy and aims to experimentally and numerically characterize the cyclic behaviour of the traditional connections between the structural elements, i.e. roof members-to-beam and beam-to-column. The intent is to possibly reduce the structural damage in the columns by introducing possible dissipation capacity into the commercial connections which are adopted in the current practice.

In parallel within the improvement of existing connections, alternative design philosophies are also under investigation for both new-designed and existing buildings with the intent to preserve the integrity of the structural elements after an earthquake event. Numerical investigations in the final part of the paper will highlight the benefits of using these new approaches respect to the traditional solutions.
INTRODUCTION

European market for precast concrete buildings is totally different from Asia and Oceania. In fact, in South Europe, 80% of industrial and commercial buildings are built with precast concrete technologies while less than 5% are multi-storey residential buildings. Industrial buildings are typically limited to one-two storeys (Figure 1a) while commercial buildings / malls are not taller than four storeys (Figure 1b).

![Figure 1 – a) One-storey Industrial buildings / warehouses; b) two-three storey commercial buildings](image)

As shown in Figures 1a, 1b different shapes and forms have been implemented in the last 30 years within the intent to improve structural performance, aesthetics without affecting total cost of the structure. Large investments have been faced improving precast technologies in order to increase quality of products and favourite standardization of casting process. Examples of roofing members and beams, respectively reported in Figures 2a, 2b also confirms the high flexibility and versatility achieved in this market to model different appealing shapes and longitudinal profiles. All these aspects, i.e. standardization of production, versatility, aesthetic appeal and cost effectiveness have certainly contributed in achieving the leadership in the European industrial market.

![Figure 2– a) Roofing members; b) beams](image)

Partial precast members and cast-in-site beam-to-beam and/or beam-to-column joints emulative of reinforced concrete [1], [2], behaviour are not commonly adopted in Europe since they drastically reduce speed of construction. Moreover since these commercial / industrial buildings are limited up to three-four storeys hinged frames possibly connected to structural walls are preferred to moment resisting frame option. Structural prefabricated elements are assembled during the construction process through “dry connections” consisting of mechanical steel devices (plates, bolts, C-channels etc.), [3].
Five typologies of connections can be identified (Figure 3): connections between roof elements (Type 1) which consist of steel plates interposed between the structural elements or cast in situ concrete topping; typical roof element-to-beam connections (Type 2) and beam-to-column connections (Type 3) are respectively reported in Figure 4a, 4b; (Type 4) column-to-foundation connection, where the column is accommodated and fixed in a precast socket foundation. Finally Type 5 regards the connection of horizontal or vertical cladding panels to the structure (beams or columns). An extensive state of art of the most common types of connection can be found in [4]. Due to the different section and longitudinal profile of the precast roof and beam elements, a great variety of connections, especially for Type 2 and 3 and different structural configurations of the roof-systems can be found in everyday practice. Each connection is generally produced by specialized producers/manufacturers which directly interact within precast company producers and designers.

**Critical Issues**

Due to the above mentioned peculiarities, precast concrete industrial buildings have particular key structural aspects that need to be addressed during design phase.

**“P-Δ” effects**

One-storey industrial buildings might achieve heights ranging from 5 to 10 meters with 300x300 - 500x500mm column sections. The high slenderness of columns inevitably leads to second order “P-Δ” effects which reduce the overall capacity and stiffness of the building with possible extended structural collapses as witnessed during Izmit earthquake, 1999 (Figure 5b, Figure 6, left side). Moreover, the larger displacements induced by “P-Δ” effects causes additional distortions which can possibly damage or bring to the premature failure of the connections.

**Diaphragm**

Roofing systems of one-storey industrial buildings are typically constructed without concrete slab on top of roof members and the presence of discontinuities, like roofing windows, creates a diaphragm effect which might not be enough stiff to uniformly distribute the seismic action to the structural members. In fact a disconnected response leads to relevant distortions of the joints and non-structural elements.

**Cladding / Infills**

The recent earthquakes in L’Aquila, Italy (6 April 2009) and Concepcion, Chile (27 February 2010) have further highlighted the very severe impact on the overall recovery effort due to the widespread damage to claddings, even within buildings with well performing structural skeletons. In particular for precast concrete
buildings with infilled concrete / masonry panels shear failure of the short column (Figure 6, right) can occur as confirmed by Izmit earthquake of 1999 in Turkey. In fact, in the ordinary practice the seismic analysis of precast industrial buildings is referred to the bare structure, neglecting the stiffening contribution of the infills. This contribution actually is relevant and leads often to a higher shear capacity but with possible reduction of the overall ductility of the system.

Figure 5 – a) Failure of precast concrete panels due to inadequate anchorage (Chilean earthquake 2010); b) collapse of building due to P-∆ effects and pour roofing connections (Izmit, 1999)

Connections
The lesson learned from the earthquakes clearly shows that it is not possible to entrust the seismic force transmission at the bearings (beam-to-column) only to the friction due to gravity loads. In fact, for example in Italy many precast concrete buildings built between the 50s and 70s in high seismicity areas have bearings without connectors, because in the past the national code did not give any specification at this purpose. Friuli earthquake of 1976 provided a meaningful lesson and after that catastrophic event Italian Association of Precast Producers and Manufacturers forbids relying on “friction connections” (Figure 6, centre). However, as also confirmed during Izmit earthquake in 1999, though mechanical connectors are properly designed according to performance based design criteria ([6]) for buildings with disarticulated roofing diaphragm and slender columns (Figure 5b), the excessive displacement demand might lead to additional forces not computed during the design.

In the following paragraphs, limitations of this current design philosophy will be emphasised through experimental and numerical analyses; alternative design approaches which overtake most of the all the above mentioned issues will be successively compared with traditional precast concrete systems.

TRADITIONAL PRECAST CONCRETE SYSTEMS
As previously mentioned, beam-to-column and roof element-to-beam connections of industrial/commercial buildings are designed for transferring shear only (hinge), not providing supplemental dissipation energy. This design approach is currently the most common adopted and leads inevitably to accept structural
damage in the columns in proximity of the foundation, where plastic hinge is expected to develop when an earthquake occurs.

In the previous editions of Eurocode 8, this type of structure was penalised with a lower behaviour factor \( q = 2 \), i.e. indicator of global dissipation capacity of the system. While with the recent and final version of Eurocode 8 (EN1998-1:2004), by introducing a set of more precise design rules and details concerning the precast structures, the behaviour factor \( q \) has been equalised to the values of cast-in-situ concrete frame systems. Different research activities, such as the European research programme *Seismic Behaviour of Precast Structures with respect to Eurocode 8* [7] gave the contributions in this direction through wide experimental tests [7] supported by numerical investigations [8], [9]. In the following paragraphs a summary of experimental and numerical investigations on a full scale building prototype are herein reported.

**Full scale testing of industrial building prototype**

A series of pseudo-dynamic tests with different levels of intensity and quasi-static cyclic tests on two typical precast concrete prototypes have been carried out. For sake of brevity results for one building prototype are herein reported (Figure 7); more details can be found in [7]. The results have been validated through refined lumped plasticity modelling [9] using RUAUMOKO 3D [10].

![Figure 7 – Test set-up of building prototype (all dimensions in cm)](image)

The prototype herein presented is composed of six columns, 5 m high, having a square cross section (400 x 400mm). The roof-elements are disposed parallel to the seismic action (four dynamic actuators). Three fixed base hinged knee joint one-storey portal frames represent the seismic resistant system of the building. The beams and roof elements were 8 m long. Classical precast foundation sockets were been adopted. They were designed to withstand the axial gravity load transmitted by the columns and bending moment imposed by the seismic action. More details on the geometry of the structural elements and steel reinforcement adopted can be found in ([7], [9]). The connections between the structural members (roof element-to-roof element, roof element-to-beam, beam-to-column) were designed to transfer the seismic action, without providing additional dissipation capacity. The details of the mechanical connections between beams and columns and beams and roof elements are not herein illustrated but their layout is very similar to the ones previously reported in Figure 4. More details can be found in [7] and [11].

The following sections are aimed at comparing the experimental and numerical results respectively obtained through the quasi-static cyclic and pseudo-dynamic tests. In particular, attention is focused on the inelastic behaviour of the columns subject to quasi-static cyclic tests with imposed displacements and on the influence of diaphragm behaviour of the roof system on the column displacement ductility demand.

**Quasi static cyclic tests**

The displacement loading protocol is properly described by [7]. Figure 8a shows the maximum drift profiles along the columns, reached during the test. Column drifts are extremely high (6% drift). Referring to the yielding displacement/drift a displacement ductility of more than 3.1 for the central column and 2.9 for the external column was observed. The numerical and experimental displacement profiles are similar with discrepancies of about 7% for the external columns and about 2% for the central columns. The good agreement between numerical and experimental results is confirmed by Figure 8b, where the force-displacement curves for internal column is represented. While the unloading curve of the hysteresis loop
is well predicted by the numerical modelling (Takeda hysteresis rule [12], $\alpha=0.08, \beta=0.35$), the initial stiffness and the re-loading curves are slightly over-estimated. The decreasing force amplitude of cycles of the columns, after yielding, emphasised in both numerical and experimental results, is mainly due to P-$\Delta$ effects occurring during the test, because of the high slenderness of the columns.

Figure 8 – Prototype 1 - quasi static cyclic test: a) column drift % profile; b) force-displacement profile of the central column

![Figure 8](image)

Figure 9 – a) hysteretic damping %-ductility for the central column; b) distortion of top connections and damage at pier-to-foundation

Figures 9a shows the numerical and experimental hysteretic damping versus ductility curves. The hysteretic damping for the central column (Figure 9a), at the highest level of displacement ductility, both numerically and experimentally is equal to 28%. Figure 9b shows the structural damage of the building prototype during testing. The large displacement demand has brought to unexpected distortion of connections at beam-to-column level, while the extensive damage in plastic hinge regions brought to the rupture of longitudinal bars due to low-cycle fatigue phenomena.

Several pseudo-dynamic tests with different level of earthquake intensity have been also carried out. More details on the pseudo-dynamic tests and parametric analyses on the buildings prototype can respectively be found in [7], [9], [11]. Results were aligned within the quasi-static cyclic tests.

INNOVATIVE PRECAST CONCRETE SYSTEMS

As confirmed by the results in the previous chapter and also by the recent severe South-European earthquakes of the last decade, significant damage has been registered in the structural elements of precast industrial building designed according to the current codes. The research community is aware of the limitations of the current design philosophies and aims to experimentally and numerically characterize the cyclic behaviour of the traditional connections between the structural elements, i.e. roof-to-beam and beam-to-column. The intent is to possibly reduce the structural damage in the columns by introducing modifications to commercial connections which are adopted in the current practice. The European Project Performance of innovative mechanical connections in precast buildings structures under seismic conditions, started in 2009, will be focused in improving existing connections, implement new devices and introduce innovative design philosophies. In this section, preliminary numerical investigations on one building prototype are presented. Two design approaches adopting dissipative connections are also described. After a short summary on experimental tests carried out on roof element-to-beam connections,
the two methods are numerically investigated and compared with the traditional solution adopting non-dissipative connections.

**Alternative design philosophies with dissipative connections**

Focusing on roof element-to-column and/or beam-to-column connections, three different design approaches can be adopted as shown in Figure 10 left side. The design approach for traditional precast industrial buildings, i.e. ordinary solution (O.S.) is to totally rely on the dissipation capacity of the columns with formation of plastic hinges close to column-to-foundation region.

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<td>Ordinary Solution (O.S.)</td>
<td>Partial Isolated Solution (P.I.S.)</td>
<td>Isolated Solution (I.S.)</td>
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**Figure 10 – Different solutions of roof element-to-beam connections and beam-to-column connections**

An alternative approach herein proposed is the adoption of a “hybrid” or partial isolated solution (P.I.S.), where part of the total dissipation capacity of the system is provided by the floor connections (roof-to-column, column-to-beam). The connections adopted are slightly modified respect to the typical ones adopted for O.S., as it will be shown in the following section, in order to improve their dissipation capacity. This would limit the displacements/drift of columns, and consequently the post-earthquake cost of repair of the system (Figure 10, centre).

A third design approach, named isolated floor solution (I.S.), consists of a total seismic isolation of the roof systems from the substructure (columns); the global dissipation capacity of the system is totally provided by the dissipative beam-to-column (Type 3) and/or roof-to-beam (Type 2) connections while columns stay in the elastic field. For I.S., dissipative devices, typically adopted for bridge deck isolation (elasto-plastic and/or friction based) and properly adapted for this purpose need to be implemented. For sake of brevity the technological aspect of connections are not considered. The flexibility of I.S. allows differentiating the source of dissipation in the two directions of building if mono-directional dissipative devices are adopted. Otherwise it is possible to use a particular roof-to-beam or beam-to-column connection which contemporary allows bi-directional relative displacements in both parallel and orthogonal directions.
respect to the beam or the column. For sake of brevity, only partial and totally isolated floor solutions within roof element-to-beam dissipative connections are herein investigated.

**Quasi-static cyclic test of roof element-to-beam connections**

As part of the national research project *Cyclic Behaviour of Mechanical Connections for Precast Concrete Buildings* founded by ASSOBETON (National Association of Precast Concrete Producers), a first series of tests were carried out at Technical University of Milan. A simplified prototype of three concrete blocks designed to reproduce a typical roof element-to-beam connection was considered. Roof elements were represented by two lateral blocks of reinforced concrete that were connected to a central one simulating the underlying beam: the setup was thus intended to be symmetrical to avoid load eccentricities. The connection is made up of four “L” steel plates (two for each side) that held on to the adjacent elements through four fastener for the beam and two steel bolt anchors (Figure 11). Push-over and quasi-static cyclic tests were carried out; the increasing displacement history was applied to the central block through two hydraulic jacks.

![View of the test setup](image1)

![Experimental force-displacement diagrams](image2)

**Figure 11 – Push-over test: comparison of two different connections’ behaviour**

![Displacement time history](image3)

**Figure 12 – Cyclic test: experimental-numerical comparison of force-displacement diagrams**

Firstly, two pushover tests were carried out (Figure 11, left side); in the first pushover (P.O.1) a typical commercial roof-to-beam connection was reproduced. The steel plate connecting the two structural elements proved to be stiff and with high strength leading to the crashing failure of the “beam” cover. In order to improve the ductility of the connection, a new connection (P.O.2) was implemented adopting a thinner plate (5mm instead of 8 mm) with rounded angles. These modifications on the connection bring to a 60% increment of maximum displacement respect to (P.O.1), with a significant distortion of the steel plate preventing spalling failure of “beam” concrete edges. The cyclic behaviour of (P.O.2) connection was investigated too; four cycles of increasing displacement amplitude were applied to the internal block (Figure 12). The experimental force-displacement curve (dashed line) showed that for cycles of equal amplitude no evident stiffness degradation occurred, while typical pinching phenomena due to anchor bolt/plate slip are evident. The dissipation capacity of the connection, given by steel plate distortion, corresponds to an equivalent viscous damping $\xi_{eq}$ ranging from 10 to 12%. More details on the above mentioned tests can be found in [13]. Wayne-Stewart hysteresis rule [14] (force-displacement curve, red
line) has been successively adopted to describe the force-displacement cyclic behaviour of the connection through a translational spring by using RUAUMOKO 2D [10]. This hysteresis rule, developed for plywood nailed timber walls, if properly calibrated successfully matches the experimental cyclic curves as shown in Figure 12. Referring to the numerical-experimental results, obtained by these preliminary tests, these enhanced roof-to-beam connections were numerical investigated in the following section for the partial isolated solution (P.I.S.).

**Numerical Analysis**

A typical one-storey precast industrial building is investigated. The building is one of the two prototypes described in the previous paragraph [7], (Figure 13). The prototype with traditional non-dissipative connections (O.S.) has been design according to Eurocode 8 with PGA = 0.3g, soil type B, S=1.2. The P.I.S and I.S. have been compared designing the connections in order to obtain the same force-displacement monotonic curve. The column section has been changed to 450x450mm, reinforcing steel has been properly designed for the three above-mentioned solutions targeting a yielding displacement $\Delta_y=37$mm.

![Figure 13 – Experimental prototype and numerical model](image)

Quasi-static push-pull and time history analyses have been carried out with RUAUMOKO 2D [10]; the structure is reproduced in a 2D-model, in the x-y plan where the roof systems lies (Figure 13). Roof elements and beam are modelled with linear-elastic beam-type elements, while columns and connections are represented by translational springs to simulate their cyclic behaviour in the Y-Y direction (Figure 13). Takeda hysteretic rule [12] represents column’s cyclic behaviour; longitudinal springs with non linear cyclic behaviour represent roof element-to-beam connections according to the different solutions proposed: in the ordinary solution (O.S.), Linear Elastic rule is adopted since connections remain in the elastic field; the partial isolated solution (P.I.S.) is modelled with a Wayne-Steward hysteresis rule [14], while for the isolated solution (I.S.) elasto-plastic and flag-shaped hysteresis rules have been envisaged.
Push-pull analyses are carried out imposing equal displacements to roof elements reaching a maximum of 140mm (column drift of 2.8%), correspondent to a column displacement ductility of 3.5, i.e. typical design q factor adopted in Eurocode 8 (or R equivalent to New Zealand Standard) for precast concrete buildings (Figure 14). As shown in Figure 14 (displacement profile), for O.S. there is no relative displacement between roof elements and beams because of the high stiffness of connections; columns reach the 2.8% drift imposed. For P.I.S. and I.S., even if the displacement history was equally imposed on the six roof elements, displacement profiles of the beam, corresponding to the top of the columns assume a parabolic shape. Consequently central columns reach higher displacements/drift than the external ones. For P.I.S. external columns drift (2.0%) remain in the elastic field while central ones go beyond yielding (blue dashed line).

The global response of the structure is expressed in terms of base shear or total force-displacement; the four cases have different cyclic behaviour (Figure 14). For I.S., where the total dissipation capacity is granted by the connections, if a mechanical device with a flag-shaped hysteresis is adopted, despite the minor energy dissipated compared to elasto-plastic hysterisis rule, a total self-centering capacity is guaranteed. This corresponds to negligible relative residual displacements between the roof and the beam, which means less post-earthquake costs of repair [15]. As shown in the equivalent viscous damping-ductility displacement curves, it’s evident that the use of elasto-plastic dissipative connections (I.S.) allows a greater dissipation capacity (\(\mu=3.5, \xi_{equiv}=45\%\)) compared to I.S. flag-shaped (\(\mu=3.5, \xi_{equiv}=22\%\)), O.S. (\(\mu=3.5, \xi_{equiv}=28\%\)) and P.I.S. (\(\mu=3.5, \xi_{equiv}=21\%\)). For P.I.S. differently for the other two solutions the dissipation capacity is equally distributed between the connections and the columns. Extensive time history analyses with different level of earthquake intensities, reported in [16] are consistent with the results of cyclic push-pull analyses and therefore are not herein illustrated.

![Figure 14 – Total force vs displacement curves; displacement profiles; damping curves](image)

**CONCLUSIONS**

The intent of the paper was to provide a brief overview of traditional and innovative design philosophies adopted for industrial and commercial buildings in the European market. From the results presented, innovative design philosophies which rely on dissipation of connections seems to be a viable design option to target, especially considering post-earthquake costs of repairs and relative possible disruption costs. The benefit of adopting a full isolated solution (I.S.) is to limit column drifts allows preserving the integrity also of the non structural elements such as façade panels, which are typically connected to the columns or beams. The partial isolation of floor (P.S.I.) is certainly a cheaper option since consists of slightly modifying the commercial connections typically adopted for precast industrial building, but despite a good drift column reduction some damage has to be expected in the columns.

Even if, these technical solutions have not yet been exploited in New Zealand, there is huge potential for precast concrete industry to compete with timber and steel in the area of industrial buildings. Despite, New
Zealand precast concrete market doesn't have an extensive variety of precast concrete production (beams, roofing members, columns, facades, etc.) as Europe has, the implementation of dissipative dry connections for (P.S.I.) or (I.S.) will be certainly more cost competitive than the European market.

REFERENCES