ROLE OF DISSIPATIVE CONNECTIONS ON THE SEISMIC RESPONSE OF ONE-STOREY INDUSTRIAL BUILDINGS

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ABSTRACT :

This paper presents alternative design solutions for one storey-precast industrial buildings, which allow to locate the dissipation capacity of the structure in the mechanical connections preserving the integrity of the structural elements. Two design approaches adopting dissipative connections are herein proposed and numerically investigated: one with partial isolation of the floor/roof system, i.e. dissipation in the connections and still moderate damage/dissipation in the columns and another solution with total isolation of the roof-system respect to the substructure, where relative displacements between the roof-system and the substructure, i.e. columns, are opportunely controlled in the design process. Preliminary numerical results confirm the enhanced performance of the one-storey industrial buildings with dissipative connections respect to the traditional solutions, where the dissipation capacity is relied only on the formation of plastic hinges at the bottom of the columns. The "isolated floor" solution seems to be most effective since the structural damage and, hence post-earthquake costs of repair, is limited to the substitution of the mechanical devices of the connections.

KEYWORDS: Precast industrial buildings, isolated floor, dissipative roof-to-beam connections

1. INTRODUCTION

Since the 1970-80's there has been a great development of precast structures, especially for some typologies of buildings such as industrial and commercial halls: in fact they cover more than 80% of the category in Italy and in South Europe. This fact justifies the growth of research activities in Europe and in Italy within the last years in order to investigate the seismic response of both existing and new precast concrete buildings.

Moreover, during the recent severe South-European earthquakes of the last decade, significant damage has been registered in the structural elements of one-storey precast industrial building designed according to the current codes; beam-to-column and roof element-to-beam connections of these 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 dissipative zones, located at the bottom of the columns, in proximity of the foundation, where a plastic hinge is expected to develop when an earthquake occurs.

In the previous editions of Eurocode 8, this type of structure was particularly 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" (G6RD-CT-2002-70002) gave the contributions in this direction through wide experimental tests [Negro et al. 2006] supported by numerical investigations [Biondini & Toniolo, 2007; Palermo et. al., 2007].

On behalf of the results obtained within the above mentioned European Project, recent research investigations are now focused on the experimental characterization of the cyclic behavior of the traditional connections between the structural elements, i.e. roof-to-beam and beam-to-column. The intent is to reduce the structural damage in the columns due to the formation of plastic hinges in proximity of the column-to-foundation region, by introducing slight modifications to the typical adopted commercial connections. In this paper, preliminary numerical investigations on one building prototype are presented referring to two design approaches adopting dissipative connections: one solution with partial isolation of the floor/roof system (Partial Isolated Solution, P.I.S), i.e. dissipation in the connections and still moderate damage/dissipation in the columns and another solution with total isolation of the roof-system respect to the substructure (Isolated Solution, I.S.), where relative displacements between the roof-system and the substructure, i.e. columns, are opportunely controlled during design. After a short summary on experimental tests carried out on roof element-to-beam connections, the innovative partially and totally isolated floor solutions have been numerically investigated and compared with the traditional solution adopting non-dissipative connections.

2. CONNECTIONS BETWEEN STRUCTURAL MEMBERS

Precast industrial buildings in South Europe are typically given by structural prefabricated elements assembled during the construction process through "dry connections" consisting of mechanical devices. These connections become key elements affecting the seismic behaviour of precast structures; in fact, even if all the structural elements (columns, beams and roof elements) are correctly designed based on criteria codified in modern seismic codes, the global structural behaviour of the building is strictly related to the correct performance of the connecting system.

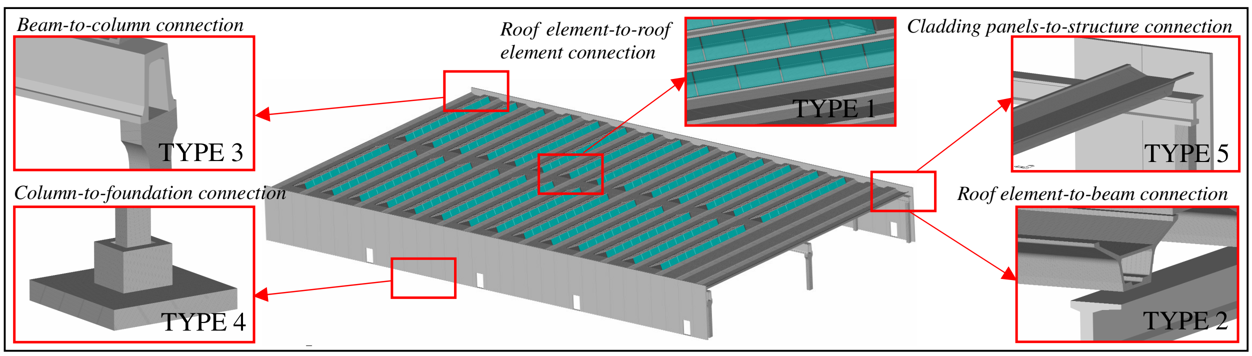

Figure 1 – Different typologies of connections between structural members

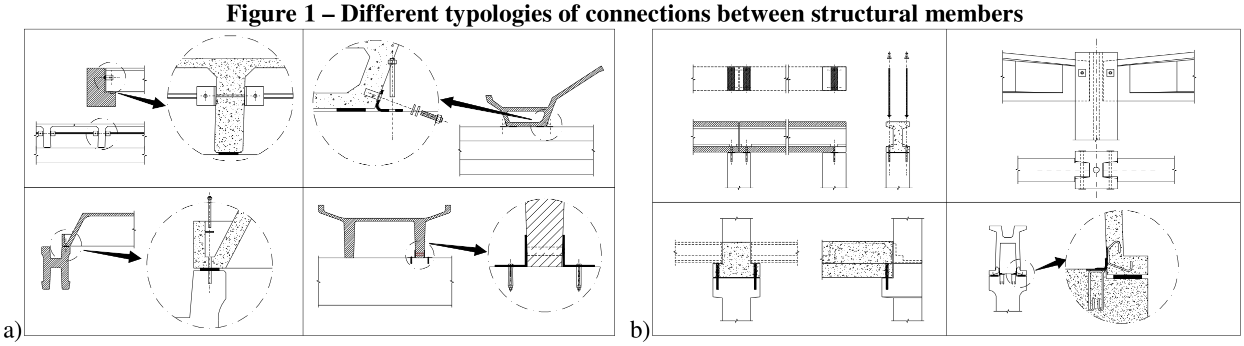

Figure 2 – a) Roof element-to-beam connections; b) Beam-to-column connections

For precast industrial buildings five typologies of connections can be identified (Figure 1): connections between roof elements (Type 1) which is typically given by steel plates interposed between the structural elements or cast in situ concrete topping; roof element-to-beam connections (Type 2) and beam-to-column connections (Type 3), where the most common typologies are respectively reported in Figure 2a, 2b; (Type 4) column-to-foundation connection, which is made of a precast socket foundation in which column is inserted and fixed with mortar. Finally Type 5 regards the connection of horizontal or vertical cladding panels to the structure (beams or columns). More details on the most common types of connection can be found in [Mandelli et al. 2007]. 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 need to be analyzed. For sake of brevity, in this paper the attention will be focused on rigid diaphragm only.

3. STRUCTURAL BEHAVIOUR WITH DISSIPATIVE OR NON-DISSIPATIVE CONNECTIONS

Due to their peculiar "quasi statically determined structural schemes", the seismic design of precast reinforced concrete buildings cannot be assimilated to tradition cast-in-situ concrete buildings. For this reason the basic rules of the capacity design ("strong column – weak beam"), cannot be applied to these systems. For these structures, the application of the "capacity design" leads to identify a "resistance hierarchy" considering both structural elements and the connecting system.

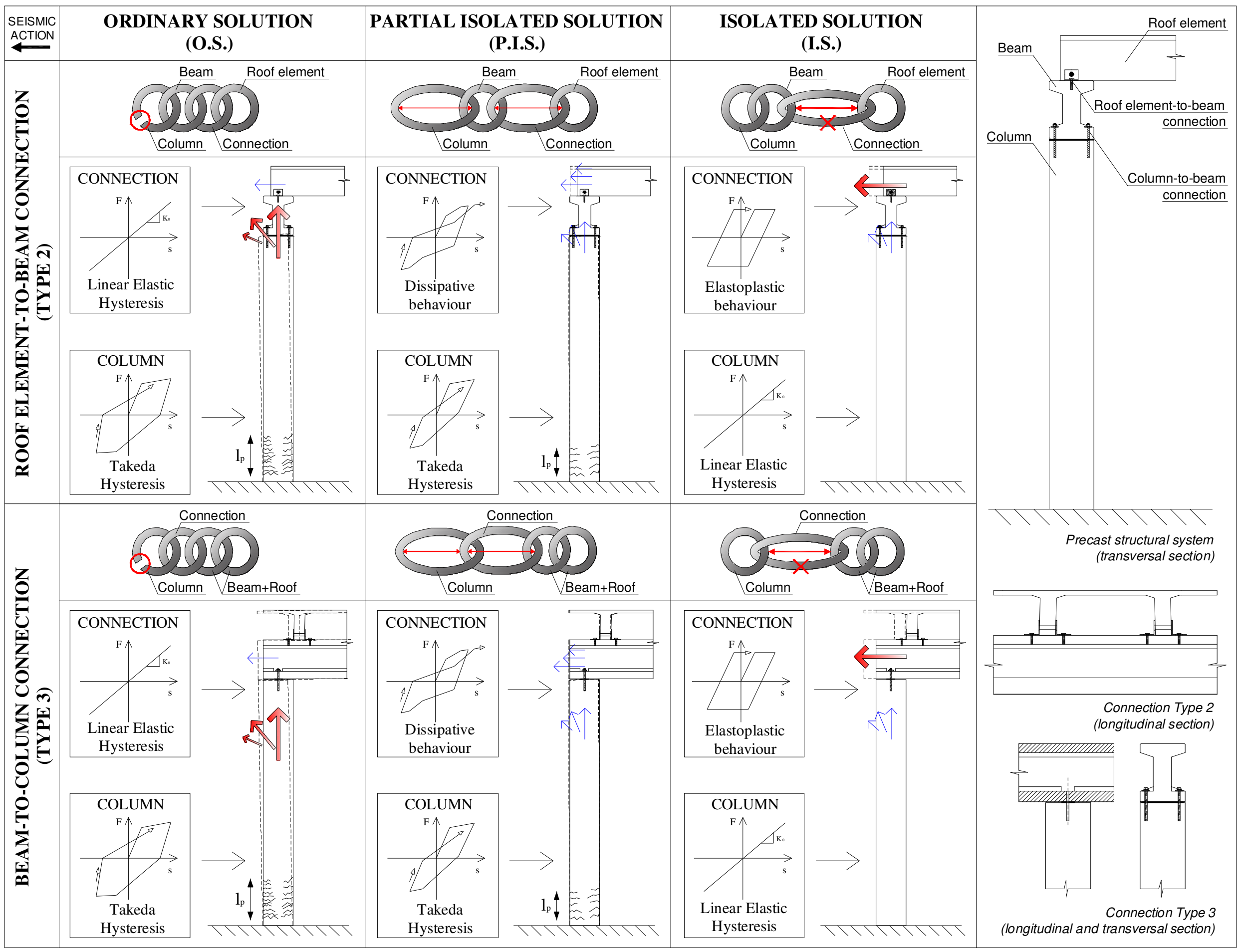

Figure 3 – Different solutions of roof element-to-beam connections and beam-to-column connections

Focusing on roof element-to-column and/or beam-to-column connections, three different design approaches can be adopted as shown in Figure 3. 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. Due to the significant column drifts reached during a seismic event, this solution inevitably leads to excessive damage, typically incremented by P-Δ effects [Saisi & Toniolo, 1998]. For the ordinary solution (O.S.), the roof-to-beam and beam-to-column connections has to transfer the seismic forces to the structural elements and are typically over-designed both in terms of strength and stiffness.
An alternative solution 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 paragraph 3.2, 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.
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 and properly adapted for this purpose need to be implemented. For sake of brevity the technological aspect of the connection is not herein 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: for example it can be used roof-to-beam connections (Type 2) to dissipate energy in the direction orthogonal to the beam and beam-to-column connections (Type 3) along longitudinal axis of the beam. 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.

3.1. Experimental Tests on Roof Element-to-Beam Connection

Within the research project "Cyclic Behavior 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 Politecnico di Milano, 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 4). Push-over and quasi-static cyclic tests were carried out; the increasing displacement history was applied to the central block through two hydraulic jacks.

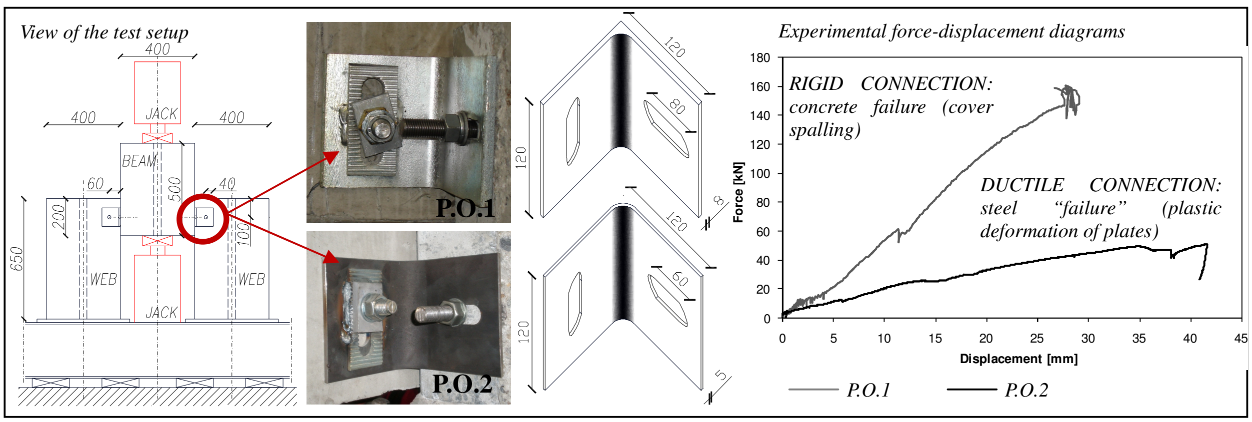

Figure 4 – Push-over test: comparison of two different connections' behaviour

Firstly, two pushover tests were performed (Figure 4, 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 5). 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 ξ_{eq} ranging from 10 to 15%. More details on the above mentioned experimental tests can be found in (Biondini et al., 2007).

Subsequently, Wayne-Stewart [Stewart, 1987] hysteresis rule (force-displacement curve, red line) has been adopted to describe the force-displacement cyclic behaviour of the connection through a translational spring by using RUAUMOKO 2D [Carr, 2006]. This hysteresis rule, implemented for plywood nailed timber walls, if properly calibrated successfully matches the experimental cyclic curves as shown in Figure 5. Referring to the numerical-experimental results, obtained by these preliminary tests, these enhanced roof-to-beam connections will be numerical investigated in the following paragraphs considering the above-mentioned partial isolated solution (P.I.S.).

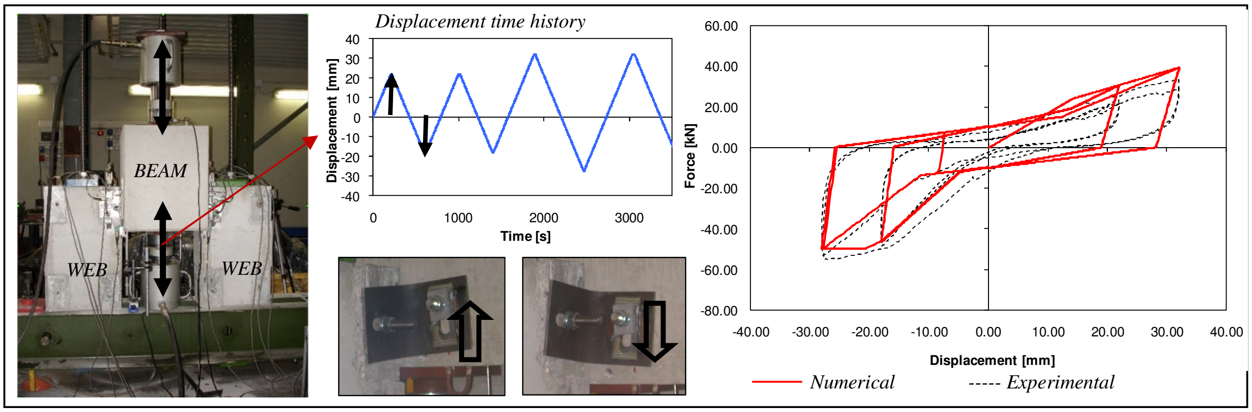

Figure 5 – Cyclic test: experimental-numerical comparison of force-displacement diagrams

4. NUMERICAL ANALYSIS

A typical one-storey precast industrial building is investigated. The building is the test prototype used during the pseudo-dynamic and quasi-static cyclic tests carried out at ELSA Laboratory (Ispra, Italy) [Negro et. al., 2006]. The prototype is composed of six columns, 5m high, having a square cross section; the roof deck π elements are disposed parallel to the seismic action (see Figure 6). The prototype with traditional non-dissipative connections (O.S.) has been design according to EC-8 [EN1998-1:2004] 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 is 450x450mm within a yielding displacement Δ_y=37mm while reinforcing steel is different for the three above-mentioned solutions.
Quasi-static push-pull and time history analyses have been carried out with RUAUMOKO 2D [Carr, 2006]; the structure is reproduced in a 2D-model, in the x-y plan where the roof systems lies (Figure 6). 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 6). Takeda hysteretic rule [Takeda et. al., 1970] 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, while for the isolated solution (I.S.) elasto-plastic and flag-shaped histeresis rules have been adopted. Benefits and disadvantages of these solutions are exposed in the following paragraphs.

4.1. Quasi-static cyclic push-pull analysis

Quasi-static 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 EC-8 for precast concrete buildings (Figure 6). As shown in Figure 7 (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).

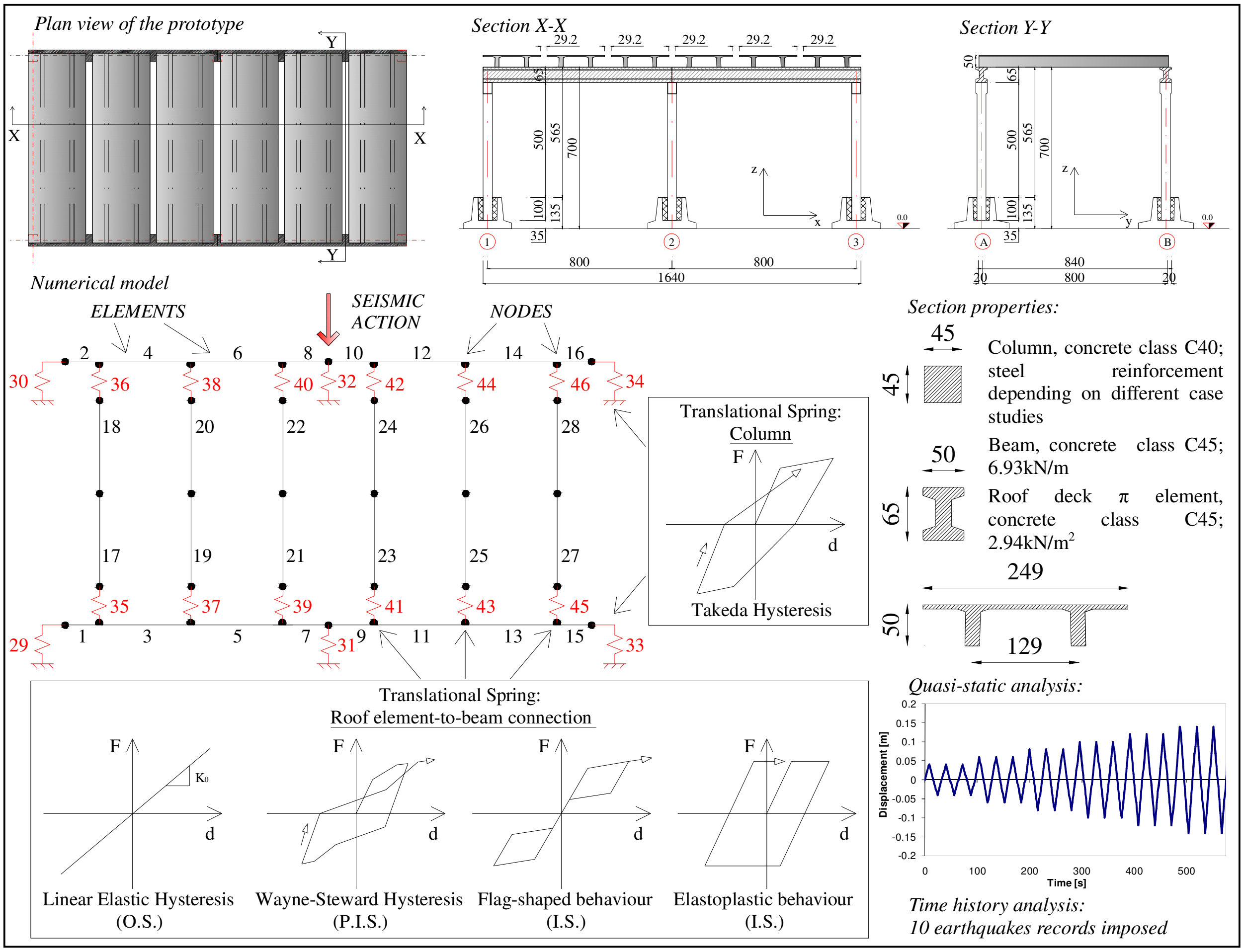

Figure 6 – Experimental prototype and numerical model

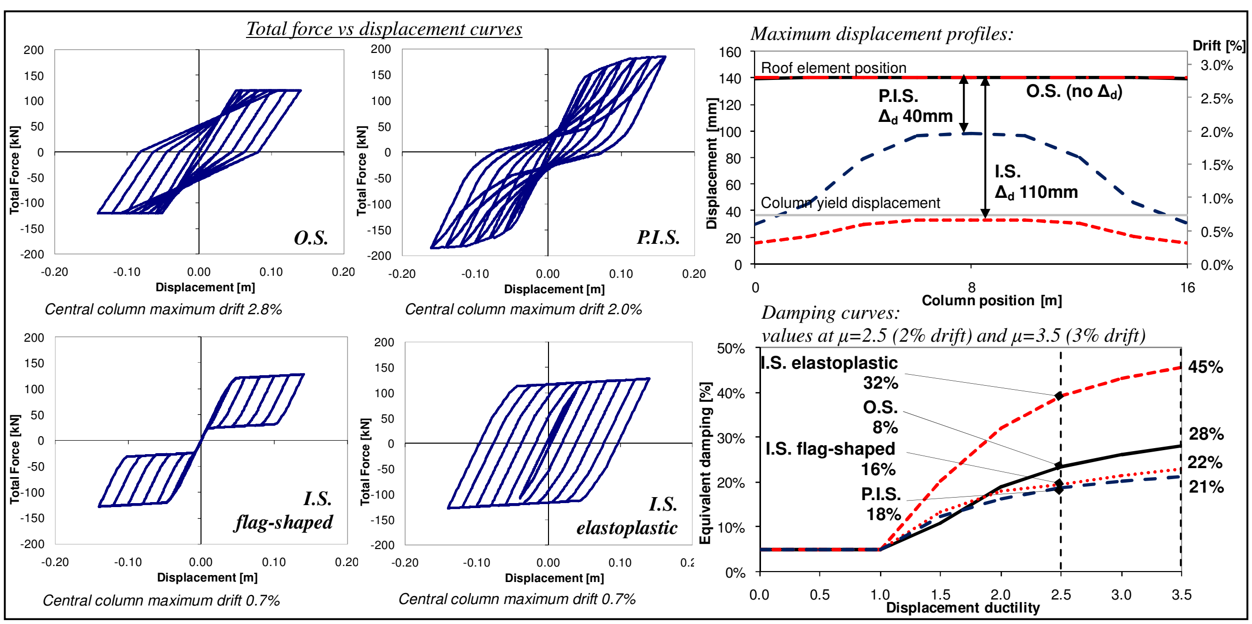

Figure 7 – Quasi-static analysis: total force vs displacement curves; displacement profiles; damping curves

Considering the global response of the structure in terms of base shear or total force-displacement for the four cases, a different cyclic behavior is evident. 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 hysteresis rule, a total self-centering capacity is guaranteed. This corresponds to negligible relative residual displacements between the roof and the beam, which assume a fundamental role as complementary damage index indicator for the structures [Pampanin & Christopoulos, 2002]. As shown in the equivalent viscous damping-ductility displacement curves, it's evident that the use of elastoplastic dissipative connections (I.S.) allows a greater dissipation capacity (μ=3.5, ξ_{equiv}=45%) compared to I.S. flag-shaped (μ=3.5, ξ_{equiv}=22%), O.S. (μ=3.5, ξ_{equiv}=28%) and P.I.S. (μ=3.5, ξ_{equiv}=21%). For P.I.S. differently for the other two solutions the dissipation capacity is equally distributed between the connections and the columns.

4.2. Time history analysis

Time history analyses use an ensemble of ten Californian earthquake records [Pampanin & Christopoulos, 2002], scaled to match the EC-8 design response spectrum with an acceleration of 0.3g. For sake of brevity only, Table 1a and 1b are herein reported; results in terms of maximum and residual displacements for each earthquake record considering the four cases analysed in paragraph 4.1 are shown. Table 1b reports the maximum roof, column and connection displacement registered during the earthquake events. The maximum mean displacement occurs for O.S. (14.4 cm) with a correspondent drift of 2.88%, while the lowest displacement is given for P.I.S. (11.2 cm); this is mainly due to the slightly higher global strength capacity respect to the other solutions, as confirmed by force-displacement curve in paragraph 4.1. For I.S. with flag-shaped and elastoplastic roof-to-beam connections, since columns remains in the elastic range, 70% of the total displacement is given by the relative roof-to-beam movements activated by connections themselves. The I.S. flag-shaped, despite a lower dissipation capacity compared to elastoplastic connections, grants negligible residual displacements between the roof and the beam; however, for P.I.S. and I.S. elastoplastic residual displacements are small (4-5% of maximum total displacement), Table 1a. For O.S. residual displacements concern column only; values around 7-8% of maximum displacement/drift have been registered. Despite the slightly higher mean residual displacements for O.S. (11.9 mm against 4.7 mm for I.S. elastoplastic), post-earthquake cost of repair will be more expensive for O.S, since columns repair is required, while for I.S. solution it is limited to the substitution of the roof-to-beam mechanical devices.

Table 1 – Time history analysis: a) residual displacements; b) maximum displacements contributions

a)

Earthquake record	*Station*	**O.S.**		**P.I.S.**		**I.S. elastoplastic**	
		Δ_{res} *[mm]*	*drift* $_{res/max}$	Δ_{res} *[mm]*	*drift* $_{res/max}$	Δ_{res} *[mm]*	*drift* $_{res/max}$
Cape Mendocino, 1992	Rio Dell Overpass-FF	3.9	0.02	2.80	0.03	8.33	0.07
Landers 1, 1992	Desert Hot Springs	23.0	0.13	1.55	0.01	1.48	0.01
Landers 2, 1992	Yermo Fire Station	27.4	0.15	1.43	0.01	3.75	0.02
Loma Prieta 1, 1989	Hollister Diff. Array	2.9	0.03	0.29	0.00	8.56	0.12
Loma Prieta 2, 1989	Gilroy Array #7	3.5	0.03	3.09	0.03	7.28	0.06
Northridge 2, 1994	Canoga Park-Topanga Can	38.1	0.17	20.59	0.10	5.55	0.02
Northridge 3, 1994	Beverly Hills 14145 Mulhol	0.8	0.01	1.59	0.02	3.44	0.03
Northridge 5, 1994	N Holliwoord-Coldwater Can	3.6	0.03	2.09	0.03	5.35	0.05
Northridge 10, 1994	Sunland-Mt. Gleason Ave	10.2	0.09	7.40	0.06	1.37	0.01
Superstition Hills, 1997	Plaster City	5.7	0.04	3.95	0.06	1.93	0.02
	MEAN	**11.9**	**0.07**	**4.5**	**0.04**	**4.7**	**0.04**
	STDV	12.9	0.06	6.0	0.03	2.7	0.04

b)

Earthquake record	**O.S.**			**P.I.S.**			**I.S. elastoplastic**			**I.S. flag-shaped**		
	total [m]	*column [m]*	*connection [mm]*	*total [m]*	*column [m]*	*connection [mm]*	*total [m]*	*column [m]*	*connection [mm]*	*total [m]*	*column [m]*	*connection [mm]*
EQ1	0.162	0.162	-	0.103	0.085	18.2	0.130	0.047	82.5	0.140	0.041	99.3
EQ2	0.176	0.176	-	0.181	0.058	122.6	0.197	0.022	174.9	0.209	0.037	171.4
EQ3	0.186	0.186	-	0.102	0.081	21.0	0.169	0.085	84.6	0.184	0.034	150.5
EQ4	0.086	0.086	-	0.066	0.038	28.7	0.066	0.029	37.0	0.065	0.029	35.5
EQ5	0.115	0.115	-	0.115	0.061	53.7	0.109	0.071	37.3	0.109	0.066	43.5
EQ6	0.220	0.220	-	0.198	0.113	84.6	0.218	0.007	210.5	0.202	0.051	151.0
EQ7	0.099	0.099	-	0.095	0.079	15.9	0.091	0.067	23.6	0.102	0.051	51.2
EQ8	0.137	0.137	-	0.073	0.051	22.2	0.102	0.036	66.1	0.102	0.018	84.2
EQ9	0.117	0.117	-	0.119	0.076	42.6	0.128	0.079	49.8	0.140	0.054	85.3
EQ10	0.140	0.140	-	0.066	0.047	18.7	0.083	0.030	53.7	0.069	0.032	36.4
MEAN	**0.144**	**0.144**	**-**	**0.112**	**0.069**	**42.8**	**0.129**	**0.047**	**82.0**	**0.132**	**0.041**	**90.8**
STDV	0.042	0.042	-	0.045	0.022	35.3	0.050	0.027	62.1	0.052	0.014	51.3

5. CONCLUSIONS

The preliminary investigations on the use of roof element-to-beam and/or beam-to-column dissipative connections confirm the enhanced seismic performance of the P.I.S. and I.S. solutions compared to the traditional solutions, which relies on the dissipation capacity of the columns. Especially for precast industrial buildings with rigid diaphragm behavior, the use of totally isolated floor solution, allow to drastically reduce column displacements limiting the cost of repair to the substitution of the mechanical devices. The benefit of limiting 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. However, if a partial isolation of floor is considered, by slightly modifying the commercial connections typically adopted for precast industrial building, a good drift column reduction can be achieved reducing the columns repair. On going investigations are focused on use of Displacement Based Design Approach (Priestley 2007) for the three afore-mentioned solutions, since it seems to be more accurate compared to a Force Based Approach, especially if partial isolated or totally isolated floor solutions are considered.

ACKNOWLEDGEMENTS

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