

# Precast Structures with Adaptable Restraints

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## SUMMARY:

In Southern Europe the heritage and the everyday practice of precast structures consists mostly in dry or semi-dry assembled frames with hinged beams and cantilever columns restrained at their base. Such structures can provide a large energy dissipation, but they are intrinsically very flexible, especially if moving from industrial mono-storey buildings to commercial or residential multi-storey buildings. Their design becomes influenced by the need of reducing such flexibility, thus largely over-dimensioning the members and obtaining a complex seismic behaviour due to the influence of higher modes.

Two solutions are envisaged to solve this problem by keeping the key feature of precast structures, that is the speed of erection: the introduction of dry or semi-dry assembled precast walls (particularly favourable in case of residential buildings) or the stiffening of the classical statically determined frame by means of introduction of moment resisting beam-column connections made with mechanical devices (mostly envisaged in the case of commercial buildings).

In such enhanced frames, the realization of clamped joints can be performed in a second stage of the erection process, adapting the existing hinged connection realized at the first stage to clamped by activating the mechanical devices.

Precast structures with adaptable restraints (PRESAR) are therefore presented in their essential features, among them the possibility of mix of a high quality dry precast construction technology with high speed of erection and the stiffness and redundancy provided by a traditional cast-in-situ frame. If properly designed, the horizontal members can fully be exploited by transferring the dead loads in a simply supported scheme before being adapted to clamped, with improvements in the behaviour and design of the whole building.

A design comparison among 3 precast frames with similar geometries but different static schemes shows how this structural flexibility can be exploited to optimize the structure. A further comparison made on the real non-linear dynamic behaviour of 4 different structures, ranging from flexible hinged frame to stiff coupled wall-frame. This work regards the basic concept followed in the design of the full scale 3-storey building experimentally tested at ELSA/JRC within the SAFECAST project (European Programme FP7-SME-2007-2; Grant Agreement n. 2184172009).

*Keywords: Precast concrete structures, seismic performance, mechanical connections, restrain adaptation*

## 1. INNOVATIVE SOLUTIONS TO REDUCE FLEXIBILITY OF PRECAST STRUCTURES FROM MONO TO MULTI-STOREY BUILDINGS

In developing of prefabricated systems in southern Europe, the structure has been usually conceived as a seismic resistant frame with horizontal members simply supported on columns, linked with a hinged joint. All the mono-storey industrial buildings are made with variously shaped slabs, for instance to create a slope for ventilation and water collecting or transparent surfaces heading north for lighting. The slab typologies have been evolving from double sloped beams to complex asymmetric slab members, until reaching bays within the limits imposed by transportation with typological variations imposed by fire and thermal insulation. The static scheme has always remained with a floor linked to the columns with hinges.

The conquest for P.R.C. prefabrication of commercial structures has been made, in accordance with the practice, with whole columns restrained at the base with supports for the 2 or 3 intermediate floors,

thus for horizontal actions the columns are designed to resist to the vertical and the horizontal loads according to their cantilever scheme.

Such technology is very efficient in terms of speed of erection, since all joints are realized with a dry or semi-dry (small completing pouring) process. In situ pouring of concrete is usually realized for the topping, necessary to create a rigid slab that uniformly distributes the storey forces to the columns, though not changing their cantilever static scheme. The creation of rigid slabs avoiding in situ poured topping is another challenge of the present.

This type of structure can still provide a large energy dissipation, in comparison with cast-in-situ frames, as pointed out by large investigations in the past, but is usually very flexible. With the introduction of the new standards, the traditional hinged precast frames have received a hard smash, due to the introduction of the “ $\Theta$ ” factor, that imposes a limit to the deformation capacity of the structure to reduce the P- $\Delta$  effects, and the introduction of SLS for seismic displacements, that impose a limit to the interstorey drift under seismic excitation. In the practice, the columns are in most cases highly over-designed with respect to the resistance, in order to fulfill the flexibility limitations.

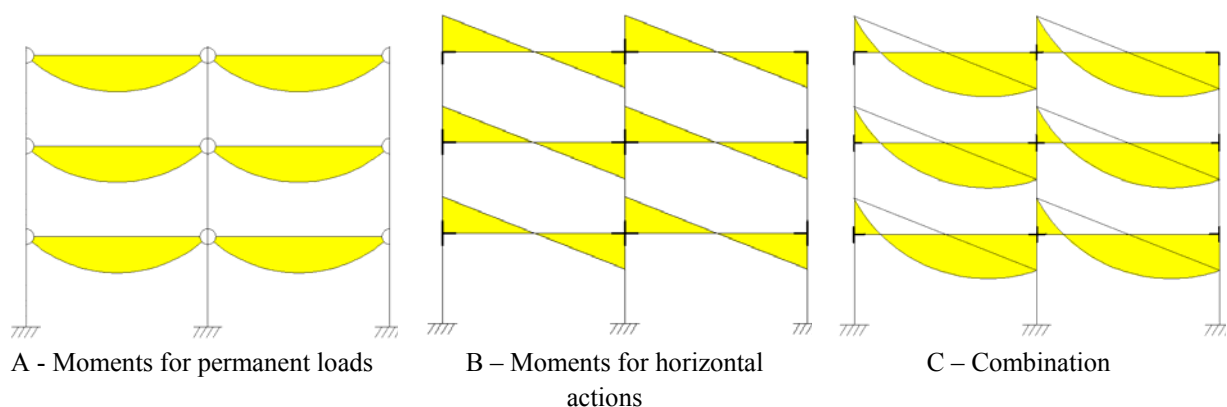
Two possible solutions are proposed:

- To adopt fully precast walls, dry or semi-dry assembled and properly connected to the floor (more envisaged for the residential market, since the presence of walls is vastly required, and the traditional precast solution consists in coupling a precast frame with a cast in situ core, thus reducing any economical advantage in the process)
- To provide the beam-column joints with dry or semi-dry moment resisting connections (more envisaged for the commercial market, since, for large units, concentration of stresses in few members becomes problematic)

The present paper is mainly focused on the second solution, of which interesting features and design basic concepts are highlighted.

The challenge is then to conceive a unique system with variable structural configuration, able to be realized either with floor hinges in case of low horizontal actions (until  $PGA = 0,10$  g for seismic) or with moment resisting connections in case of higher actions, in free quantity and position, for instance just at the last floor or at each node according to SLS and “ $\Theta$ ” demand, exploiting the possibility of designing highly dissipative and ductile structures with improved stiffness.

Such substantial innovation of the structural scheme is based on the possibility that precast structures have (cast-in-situ do not) to realize a frame where all the structural weights and permanent loads are taken by simply supported members (hinged frame), where it can be chosen afterwards for which nodes to become a restraint, thus flexurally subjected only to variable loads (live loads, wind, earthquakes, etc.).



**Figure 1.** Enveloping bending moments for sum of the 2 loads. Note that, inverting the horizontal applying direction, the envelope at the lower side of the beams becomes almost constant, requiring a same amount of reinforcing, easily provided by the pre-stressing

In the frame it is after erection that the connections are modified (hinges to restraints), exploiting the pre-stressing that, if properly designed, can reduce to zero the deflections due to the combined creep of concrete and steel relaxation, in order to admit a juxtaposition of the actions derived from the two

different static schemes and to realize moment resisting nodes, that are subjected to much smaller maximum moments than the ones due to dead loads, too, typical in cast in situ nodes.

If for instance the live loads were approximately equal to the dead, the precast joint would be subjected to a maximum moment equal to half of that of a cast in situ.

Such reduced moments have to be compared with the moments due to seismic action, calculated with a design behaviour factor  $q$  that needs to be redefined (as more in detail explained later) as the factor that makes the seismic moments equal to the maximum moments due to live loads.

Finally, the possibility to transform, wherever needed, the hinged nodes in restrained, makes the precast frame much more advantageous than the cast in situ.

Note that at the top floor the live load is usually the snow, thus its moment can easily be less than the seismic moments calculated with very high  $q$  factor ( $5 \div 5,85$ ). This can suggest that the solution with restrained nodes just at the top is very interesting. Another important advantage in this solution is that at the last floor there is no need of capacity design, since the position of the plastic hinge does not change the global mechanisms. When and how it is possible to realize such high ductility restraint is therefore presented. A brief description of the commercial construction system designed with such assumptions, the **Pandal**<sup>®</sup> system, is therefore presented.

The beam-column joint that initially gives support to the beam with a corbel and dowels has to be designed with some special features:

- 1) The beam shall have a stable straight profile, obtained by a proper pre-stressing, designed in such a way that the end section does not rotate in time and, which is the same, the precamber of the beam is almost constant and small.  
The deflection control with pre-stressing is possible in beam sections with low center of the mass.
- 2) The dowel, after the connection of the longitudinal bar in the node and the mortar filling, will become a valid stirrup of the statically undetermined beam, that “forgets” to have a corbel.
- 3) The joint has to be prepared for the sealing of every contact surface with mortar, and the vertical sides are made rough for this purpose.
- 4) The reinforcement that will be connected afterwards has to be inserted at the upper and lower sides of the precast beam through proper devices.



**Figure 2.** Whole columns with cross shaped corbel for the support of the shallow hollow core beam and detail of one beam edge. Note the rough surfaces for better interlock, the innovative mechanical connections placed in correspondence of the column and the hole for the dowel connection, in correspondence of the rib

The special hollow core beam, with its width of until 2,50 m and its weight identical to the one of a classical inverted T shaped beam, allows to reduce the bay for the floor members and becomes part of the slab, with clear economic advantages, providing also high torsional stiffness.

Its section, with centroid of the mass lower than half the height, admits a calculation with zero deformation that allows the designer to sum the actions due to dead loads with hinged scheme with the live actions with fixed restrained scheme.

The hollow core floor members have the same height of the beam and a special member put in correspondence of the columns forms a frame in the direction perpendicular to the beam, with the possibility of adaptation to clamped joints.

The floor members are maximum 2,50 m wide and are distanced with a trussed slab that allows great project flexibility and strongly reduces the height of the floor.

The fixed restraint to the column is obtained by activating the mechanical devices at the upper and lower sides in the Pandal beam or the frame floor member. The connection, in this case an innovative bolted device, realizes the equivalent of a reinforcement passing through the node, granting an horizontal zone of energy dissipation (critical zone) that makes the node ductile.

The insertion of such connection system in the building can also allow a restraint with a provisional connection (before mortar pouring) which blocks the rotation around the dowel bar, to avoid the danger of instability of a column that, designed for a moment resisting frame, would be too slender with the cantilever scheme.

## 2. COMPARISON STUDY

A comparison study has been performed on a three storey frame building with commercial function. The building is made of a regular frame with 4x5 bays with 12x10 m span and the inter-storey height is 5 m. Three different structural configurations have been designed and compared:

1. Precast frame with hinged beam-column connections
2. Precast frame with moment resisting beam-column connections activated at the roof
3. Precast frame with moment resisting beam-column connections activated everywhere

In this case, a direct comparison with a cast-in-situ equivalent frame is hardly meaningful, due to the large spans, that require the pre-stressing technology for reasonable member dimensions.

The main hypotheses for the comparison are the following:

- Same vertical and wind loads; same PGA
- Same spans
- Same member typology (square section columns; box hollow core section for beams and floor members) but free height
- Perfectly rigid diaphragm
- External stair core structurally separated to the frame (therefore not considered)
- Cladding panels statically determinate connected to the frame, therefore acting just as masses without stiffness
- Column stiffness reduced by 50% as suggested by the standards
- Centre of the mass put in correspondence of the center of inertia (no torsion)
- Beam and floor member with corbels (uniform slab height)
- Columns have constant section along the height
- Beam-column connections are considered as perfect hinges and, when the innovative connections are activated, perfect clamps

For each configuration, a complete design process has been performed. The diagrams of the critical members are then reported in order to ensure a meaningful comparison.

The loads applied to the structure are indicated in table 1.

**Table 1.** Static loads

LOAD	kN/m <sup>2</sup>
live	6
snow	1,2
dead non structural	1,5
dead ns roof	1,2
wind (peak at the top)	1,05
wind (peak at the base)	0,75

The structural weight varies for each prototype. The seismic design is performed with a modal dynamic analysis with response spectrum. The PGA has been taken equal to 0,25g, with a EC8 spectrum corresponding to soil type B.

## 2.1 Dynamic parameters

In table 2 the dynamic parameters of the three prototypes are reported. It is noticeable the decrease of the fundamental period while the structural redundancy is increasing. The participation factor related to the first mode, on the other hand, is increasing, providing a more “clear” response. The influence of higher modes, therefore, decreases while the structural redundancy increases. Please note also that the period of higher modes does not considerably change with the redundancy.

**Table 2.** Dynamic parameters of the 3 structures

PRECAST BUILDING WITH HINGED BEAMS			
MODE	PERIOD [s]	PART. FACTOR X	PART. FACTOR Y
1	1,51	0,71	0
2	1,51	0	0,71
3	1,17	0	0
4	0,27	0,14	0,09
5	0,27	0,09	0,14
PRECAST BUILDING WITH ROOF BEAMS CLAMPED			
MODE	PERIOD [s]	PART. FACTOR X	PART. FACTOR Y
1	1,38	0,8	0
2	1,32	0	0,81
3	1,21	0	0
4	0,28	0,16	0
5	0,26	0	0,15
PRECAST BUILDING WITH BEAMS CLAMPED			
MODE	PERIOD [s]	PART. FACTOR X	PART. FACTOR Y
1	0,89	0,86	0
2	0,74	0	0,89
3	0,70	0	0
4	0,28	0,11	0
5	0,25	0	0,10

## 2.2 Design highlights

The design highlights are collected in table 3. The storey forces are calculated according to the chosen design methodology, without the amplification factors that so flexible structures would require.

**Table 3.** Design highlights for the three buildings

PRECAST BUILDING WITH HINGED BEAMS				PRECAST BUILDING WITH ROOF BEAMS ADAPTED TO CLAMPS				PRECAST BUILDING WITH BEAMS ADAPTED TO CLAMPS			
member dimensions [mm]				member dimensions [mm]				member dimensions [mm]			
column side	1000			column side	850			column side	750		
slab height	600			slab height	600			slab height	600		
roof slab height	400			roof slab height	600			roof slab height	600		
displacements[mm]				displacements[mm]				displacements[mm]			
storey	1	2	3	storey	1	2	3	storey	1	2	3
wind in x	2,5	8	14,6	wind in x	1,4	2,9	3,7	wind in x	1,1	2,3	2,9
wind in y	2,5	8	14,6	wind in y	0,9	1,6	2	wind in y	0,7	1,3	1,5
equake in x ULS	35	115	212	equake in x ULS	45	98	127	equake in x ULS	40	88	114
equake in y ULS	35	115	212	equake in y ULS	43	86	105	equake in y ULS	37	74	91
equake in x SLS	14	46	85	equake in x SLS	18	39	51	equake in x SLS	16	35	45
equake in y SLS	14	46	85	equake in y SLS	17,00	34,00	42,00	equake in y SLS	15	30	36
theta wind x	0,20	0,22	0,13	theta wind x	0,11	0,06	0,02	theta wind x	0,09	0,05	0,01
theta wind y	0,20	0,22	0,13	theta wind y	0,06	0,03	0,01	theta wind y	0,06	0,02	0,00
theta equake x	0,18	0,12	0,07	theta equake x	0,23	0,08	0,02	theta equake x	0,27	0,09	0,08
theta equake y	0,18	0,12	0,07	theta equake y	0,18	0,05	0,01	theta equake y	0,21	0,06	0,04
seismic storey forces [kN]				seismic storey forces [kN]				seismic storey forces [kN]			
equake x ULS	2375	4750	3156	equake x ULS	2396	4792	3184	equake x ULS	1818	3635	2416
equake y ULS	2375	4750	3156	equake y ULS	2483	4965	3299	equake y ULS	2169	4338	2883
behaviour factor		1,5		behaviour factor		1,5		behaviour factor		3,0	

Regarding the building with hinged nodes, horizontal members are designed for gravity loads, since they do not affect the response of the building to horizontal loads and free relative rotations can occur at beam-column joints. The building is very flexible, and its first period is of 1,51 s. Therefore, the seismic demand is reduced, since a smaller acceleration in comparison with the ground is expected to act.

The critical request during the design process has been the  $\Theta$  factor at ULS, rather than the deformability at SLS or the resistance. It is remarkable that the  $\Theta$  factor is directly proportional to the behaviour factor  $q$ , since within the force based design the horizontal forces are divided by  $q$ , while displacements are not. Such coefficient gains importance also in the load case of wind. The cross section of the columns is therefore the key parameter to control the stiffness of the building. The prescription of EC8, that suggests to adopt a minimum side dimension equal to a tenth of the shear length, has not been respected (it would have brought to a 1,5 m side dimension).

The behaviour coefficient  $q$  has been taken equal to 1,5 in such a way that the value of  $\Theta$  does not overcome 0,3 under the wind load. It is remarkable that the maximum value occurs at the second floor. Regarding the building with clamped roof, while the horizontal members at the lower storeys are not involved yet in the horizontal load resisting scheme, the roof beams and floors are in this case adapted to clamps, forming a portal frame in two directions with the columns. The seismic design of the building has been again determined by the  $\Theta$  factor, while the resistance of members and the SLS have been widely satisfied. The height of the roof slab has been increased with respect to the previous case both for resistance of the beam and for global stiffness. The cross section of the column is again the key parameter in the design process. The prescription of EC8 on the minimum side length of the column would have led to 0,75 m, less than the adopted. Furthermore, in this case the value of  $q$  can not overcome 3,5, in accordance with the standards.

In the case of the fully clamped frame, all the beams are involved into the horizontal loads resisting mechanism. The key parameters for the design have been the again the  $\Theta$  factor, in addition to the deviated axial-flexural resistance at the base of the columns and the application of the capacity design in its strong column-weak corollary. The prescription of EC8 on the minimum side length of the column would have led to 0,25 m, much less than the adopted. The maximum value of  $q$  factor that can be taken is 5÷5,5.

Resuming, the comparative design proves that:

- Global stiffness and structural redundancy increase passing from the hinged beams scheme through the clamped roof to the fully clamped.
- Beams and floor members are not involved in the horizontal loads resisting system if hinged. In this case, they can be designed only for gravity loads and the strong column – weak beam concept can be neglected.
- The precast beams, first simply supported and afterwards clamped, can be fully exploited, since in the seismic load combination the envelope of bending moments in critical zone (the edges) is such that the positive and negative bending moments are very similar, which would not happen if the beam is cast-in-situ. This is due to the transmission of all dead loads in the simply supported scheme, that carries null moments at the ends. Furthermore, the envelope of the positive bending moment shows an almost constant distribution. This leads to an optimization of the pre-stressing steel, that can cover alone all the demand of positive moments out of the critical zone.
- Clamped beams at the roof can avoid the strong column – weak beam concept.
- For frames with more than the roof storey clamped, the reduced bending moment of the precast clamped beam if compared to a cast-in-situ equivalent can lead to smaller column cross sections, since for these types of frames the strong column – weak beam concept is applied.
- The bending moment distribution in columns is more optimized passing from the hinged beams scheme to the roof clamped to the fully clamped.

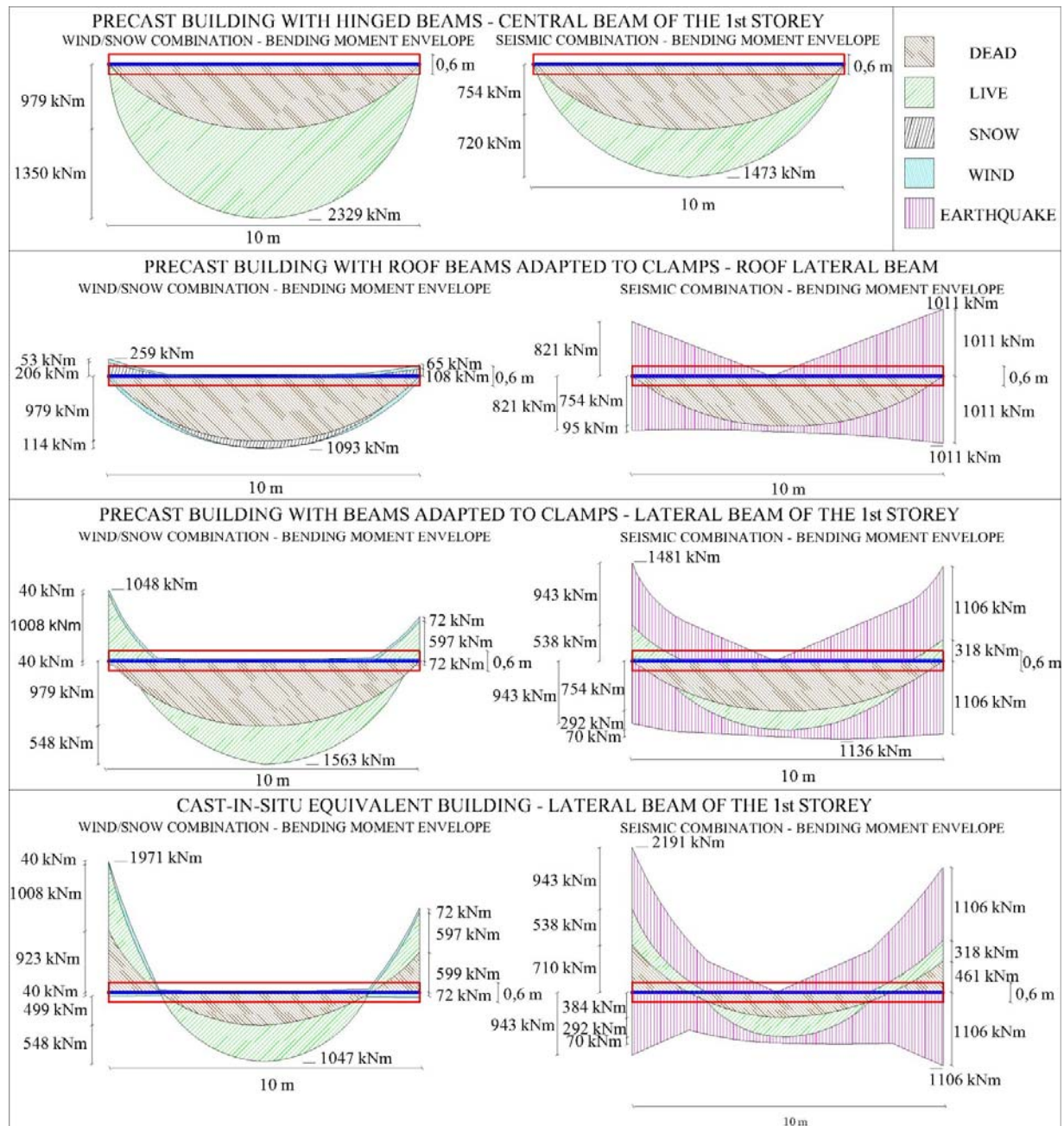
A graphic description of the dimensions and distribution of actions on the critical members is given in fig. 3 (beams) and in fig. 4 (columns).

### **2.3 New definition of the best behaviour factor $q$ to be adopted in the design**

The optimal design of the clamp adapted beams can be conducted by designing the pre-stressing on the maximum value of bending moments from gravity loads and then choosing the minimum behaviour factor that leads the edge maximum moment for seismic combination not to overcome the midspan maximum moment for gravity combination or, on the other hand, the mild steel can be



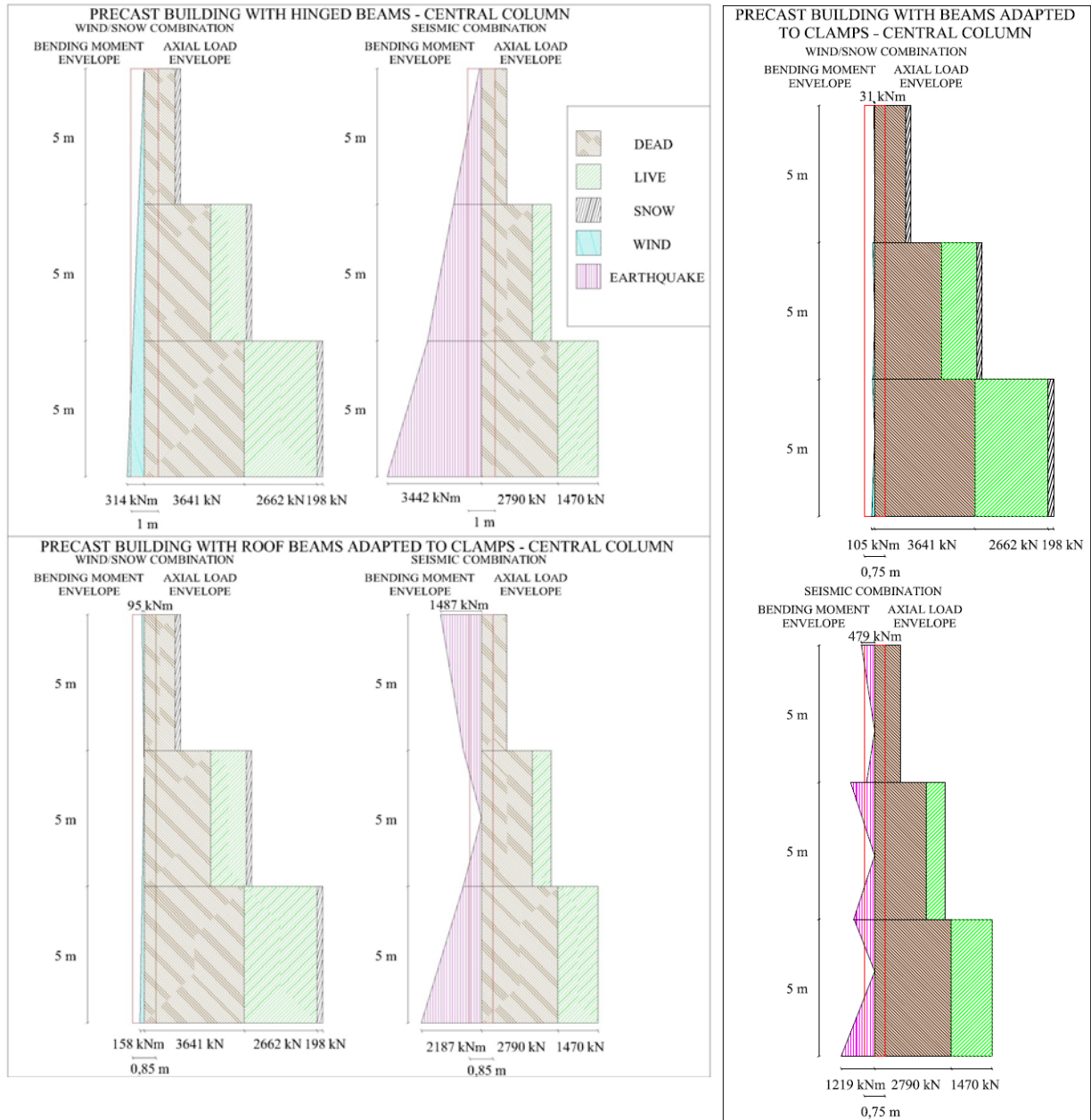
optimized by choosing the minimum behaviour factor for which the edge moments for the seismic combination are equal to those for gravity loads. It is remarkable that all the three columns shown in fig. 4 carry very similar axial loads, while the distribution of bending moments and shear force have very different shapes (also affecting the design of the foundations).



**Figure 3.** Beams: design comparison

The definition of the behaviour factor is a main importance step in the design phase. Hereafter the factors that contribute to its choice are underlined. The optimal behaviour factor for all columns is the minimum among:

- the maximum from standards
- the one for which  $\Theta = 0,2 \div 0,3$
- the one for which the maximum bending moment for seismic loads is equal to the maximum bending moment for gravity loads or for capacity design.

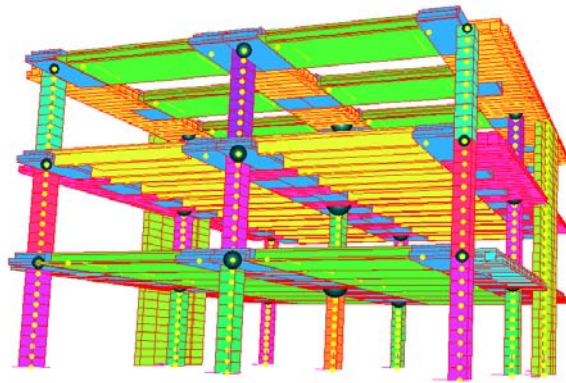


**Figure 4.** Columns: design comparison

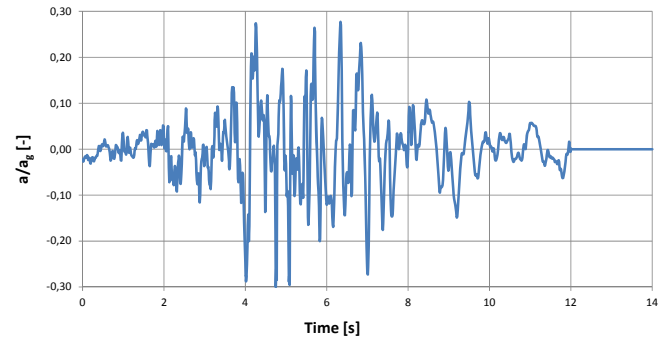
### 3. COMPARISON OF THE NON LINEAR DYNAMIC BEHAVIOUR

A further comparison study is proposed regarding the main structure that has been full scaled tested at ELSA/JRC laboratory within the SAFECAS project, the design of which follows the concepts described in the present paper. The building is a regular three storeys residential frame, made with 2x2 bays with 7x7 m span and inter-storey height of 3.4-3.2-3.2 m. Also the solution of semi-dry assembled precast walls has been considered, with the insertion of two innovative precast walls in the direction of the application of the earthquake. More details about the structure (see fig. 5) can be found with reference to the SAFECAS project. The 12 s long Tolmezzo earthquake (see fig. 6), modified to fit the response spectrum given by EC8 and scaled at a peak ground acceleration of 0,30g, that corresponds to the design PGA for ULS, has been applied to the structure.



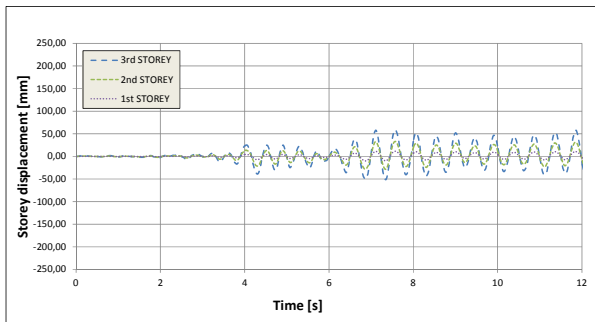


**Figure 5.** FEM model

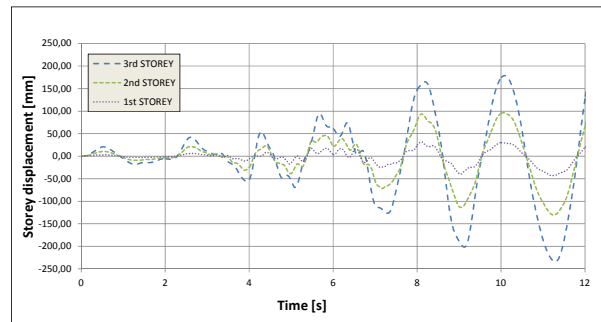


**Figure 6.** Modified Tolmezzo accelerogram

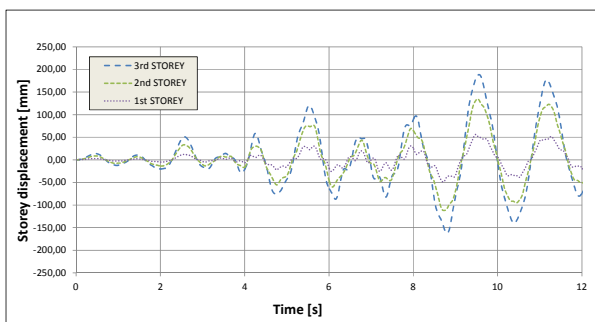
From the comparison among the four different displacement time histories (fig. 7) and base shear-top displacement diagrams (fig. 8) it can be noticed that the structure with walls is the most stiff, and it is subjected to the highest forces, to which correspond the lowest maximum displacements. Its response is scarcely influenced by higher modes effects, as can be noticed by the accordance of the storey displacements. A completely different situation can be noticed for the frame with hinged nodes, where a much lower stiffness is reported and where the effect of higher modes is predominant on the response, as can be noticed by the counter-acting displacements and the confused peaks of the shear-displacement plot. The high deformability leads to high displacements. The case with clamped roof is still characterized by the same phenomenon, but stiffness is higher and the effect of higher modes is reduced. The activation of the connections at all nodes furthermore improves the behaviour, though forces are higher.



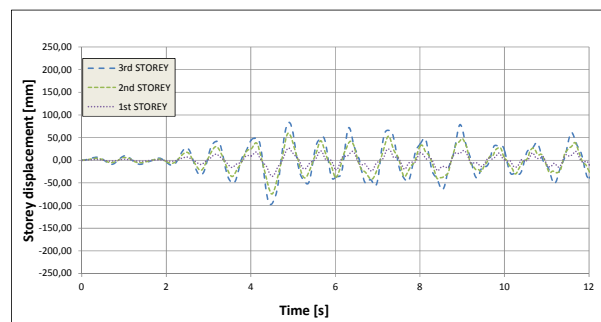
(a - Walls)



(b - Hinges)

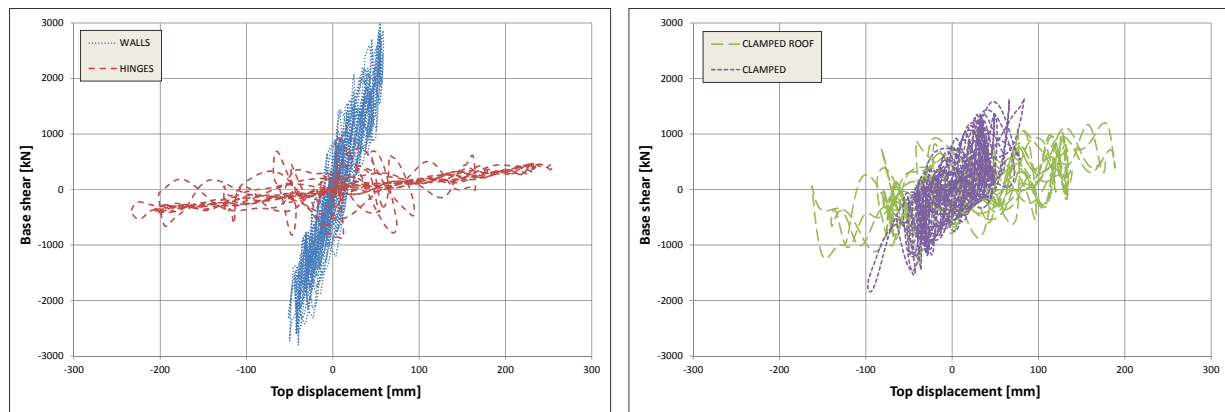


(c – Clamped roof)



(d - Clamped)

**Figure 7.** Displacement time histories



(a,b) – Walls, Hinges

(c,d) – Clamped roof, Clamped

**Figure 8.** Shear force vs Top displacement diagrams

## 5. CONCLUSIONS

A new solution for high erection speed dry precast construction systems is presented, where the traditional beam-column hinged joints can be adapted to clamped (PRESAR buildings). A design comparison made out of three different static schemes shows how the structure can be optimized with different configurations and a new definition of the best design behaviour factor to be used is proposed. A comparison on the real seismic response of four similar structures with static scheme ranging from cantilever column type to coupled wall-frame shows the effectiveness of the proposed solutions. The present work shows how precast frames can achieve better behaviour than cast-in-situ frames, with several advantages. A full scale three storeys PRESAR frame structure has been tested within the European SAFECast project (FP7-SME-2007-2; Grant agreement n. 218417/2009 - [www.safecastproject.eu](http://www.safecastproject.eu)) at the ELSA/JRC laboratory in Ispra (Italy).



**Figure 9.** Precast structure with adaptable restraints tested at ELSA/JRC, Ispra (Italy) behind (from the left) P. Negro, A. Dal Lago and G. Toniolo within the SAFECast Project

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