Proceedings of the Ninth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Society
14-16 April, 2011, Auckland, New Zealand

Feasibility of high-rise buildings with PRESSS-technology

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ABSTRACT: This paper presents a first investigation on the feasibility of high-rise buildings using PRESSS-Technology and its seismic performance capabilities. This solution, in fact, can develop a very stable inelastic behaviour without leading to structural damage in the plastic hinge regions and while guaranteeing residual displacements within the operational limits due to its self-centring behaviour. On the base of an existing reference building, composed by the interaction of different resisting systems strategies, two different systems, namely wall systems and frame systems will be afterwards analysed in details. While looking at the responses of these two different resisting systems, the seismic response when implementing a monolithic type of connection is then compared to that of a PRESSS-technology alternative for both wall and frame systems. The Hybrid solution, proposed and analyzed on four different heights of Tall Buildings, from 15 to 45 stories, appear to be a valid alternative to the monolithic connection, having comparable response in terms of interstory drifts and shear-moment demand, but considerably advantages when looking at the overall performance in terms of low-damage, negligible residual-permanent displacements as well as when including possible benefits in the constructability of the whole system. On the contrary the displacement design approach, although it has shown great potentials and flexibility, in its actual formulation is still lacking of a reliable accuracy on the peculiar response of taller structures.

1 INTRODUCTION

The recent and quick spreading world wide of tall structure has recently highlighted the need to dedicate special attention to the seismic design of Tall Buildings (TB) also in moderate-to-high seismic regions, raising these structures from isolated cases up to more standard construction practice. This new status would require an exhaustive knowledge of the expected response of these structures in many different scenarios. Instead there is still a significant lack of code-based design procedures specific for Tall Buildings. Furthermore, the behaviour of Tall Buildings is quite atypical if compared to ordinary-high structures, as it is affected by very high aspect ratio and thus sensitivity to higher modes effect, very low level of inherent damping, relatively high floor displacements and accelerations or exposition to wind and slenderness not experienced by lower (more ordinary-high) structures. The particular behaviour, especially in seismic areas, as well as the relevant importance of Tall Buildings as structures, due to their impact on the communities and urban areas, and their economic value, suggested the need of specific investigation on the design procedure and construction techniques.

The PREcast Seismic Structural System, or PRESSS-technology, based on jointed ductile connections, assembled together by means of unbounded post-tensioning techniques represents a very powerful emerging solution for high-performance (whilst still being a cost effective) damage-resisting system.

Originally developed from the late 90s for multi-story precast structures (Priestley, 1991; Priestley et al., 1999), these systems are nowadays widely implemented with different arrangements in several countries worldwide in both residential and commercial buildings (NZCS, 2010).

Due to its nature of dry precast connections and the possibility to exploit pos-tensioning to reach longer span and open space, PRESSS technology would be well suited to improve important and typical requirements of Tall Buildings such as fast and modular construction, clean and safe work area. Furthermore, the peculiar concentration of the deformation at the joint level, controlled by the rocking motion between beam and column or column/wall and foundation guarantees no damages in the structural elements. The seismic performance of the building is enhanced also for low intensity of the seismic demand, since the resisting connections and systems can be designed to remain basically undamaged and in operational conditions almost regardless of the magnitude of the earthquake event and actual shaking level felt by the structure.

In addition, one of the main lessons learnt from recent earthquake events is that even when buildings were able to sustain the impact of the ground motion, satisfying the target collapse prevention limit state, very often the cost (direct and indirect, including business interruption) of repairing those plastic hinge regions, typically accepted as sacrificial area within the structure to accommodate the inelastic mechanisms, is well beyond expectation and acceptable values. Furthermore, while often the design criteria refers to maximum displacements/drifts as a damage indicator, residual (permanent) displacements and deformations result in the effective partial or total loss of structural safety for post-event occupation as well as in an increased cost of repair or replacement of the non-structural elements (NZCS, 2010). Recent works in the framework of performance-based design have suggested to adopt residual displacements as a fundamental complementary damage indicator in combination with the more typical maximum displacement parameters (Pampanin et al., 2002), and to directly incorporate residual deformation in a Displacement-based Design Procedure, in order to better control the actual performance of the structure. The flexibility design of the PRESSS connections, due to various combinations of post-tensioning and dissipation at the chosen performance level is very attractive when tackling the design of a Tall Buildings. The feasibility of PRESSS solution for Tall Building application is therefore an interesting whilst challenging research area that may lead to the opportunity to improve and overcome current limits in the current structural systems performances.

2 CASE STUDY

2.1 Case Study Building

The aim of the study was to test the current design practice and the feasibility of the proposed concepts starting with an overall idea of the real behaviour of a standing structure. The case study was originally designed for 25 storeys, with a total height of 100 m at the roof level. The geometry in plan was a 45 m square with a homogeneous vertical development and clear symmetric design. The structural system is a combination of an inner shear wall core, consisting of a coupled RC service shaft outrigger with RC columns for the whole height, and an outer frame-tube system composed of 6 short bays frame with deep beams (Figure 1). Although at a first look the structure may appear redundant, the case study building was designed for a high seismic zone, and its design respects the Japanese current practice’s criteria (AIJ, 2000). For the sake of simplicity, the structural systems will be considered only to resist seismic actions, decoupling their effect from the gravity load case and different source of lateral forces as wind or impacts. The attempt to simplify the case study scenario means to clearly assess the possible contribution of the proposed concept technology, without being affected by the propagation of errors and uncertainties due to the complex structural approximations: if the hybrid-PRESSS solutions are reliable then they may be implemented in more elaborated configurations. For this reason, before analysing the interaction of the two structural systems within a Tall Building, the authors preferred to estimate the response of the two resisting systems independently, wall and frame alone, with no attempt to be exhaustive on the topic and referring to some more accurate studies on wall-frame, i.e. Sullivan et al., 2006.

In order to better understand the height limits both of the structural systems and the reliability of the design procedure, four different heights were considered for each structural system in this study. With an imposed storey height of 4 m, the four models were of 15 st (60 m), 25 st (100 m), 35 st (140 m) and 45 st (180 m). Having kept the in plan geometry equal for all four building models, the aspect ratio of the structural systems changed significantly. As a consequence the behaviour and response of the buildings themselves also varied. With the increasing slenderness of the structures, the P-Δ effects can become critical; as a consequence, the remedies to control the reduced performances of the structure may affect and control the design of the structural members, and in some cases reduce the impact of a seismic design of those elements. In the first part of the study, P-Δ effects have been only briefly analysed, with the intention of focusing further studies specifically on this fundamental aspect of the design. It is worth noting that previous studies on self-centring dissipative systems based on a flag-shape hysteresis loops have already shown a lower sensitivity to P-Δ effects of these systems when compared to traditional cast-in-situ solutions (Pampanin et al., 2002). The results of time-history analyses with and without P-Δ effects indicated that the re-centring systems were less affected than traditional systems showing lower mean value of the P-Δ amplification factors as well as lower dispersion (standard deviation) of the results.

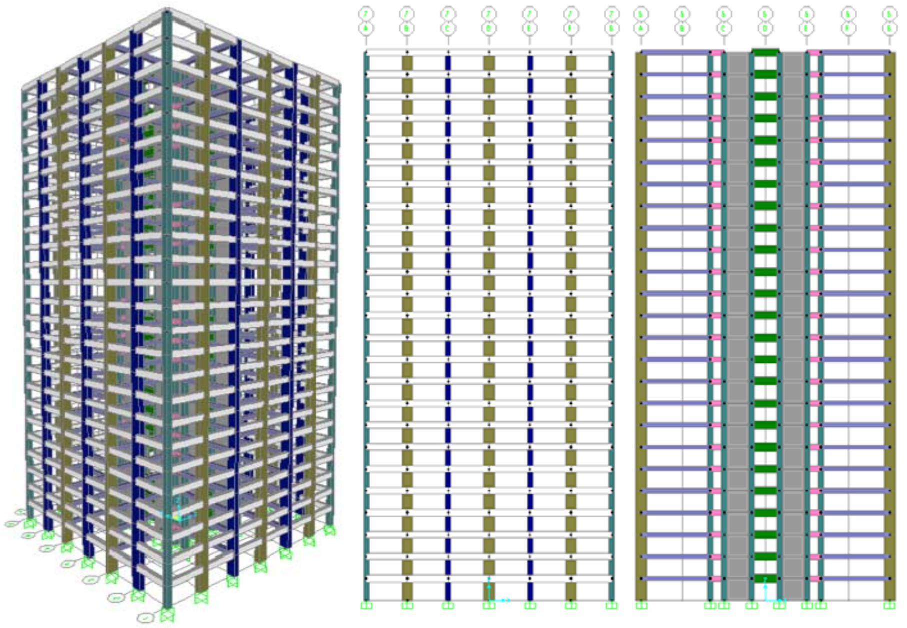

Figure 1 Original elevation model of the case study later simplified in the different structural systems and modelled with Ruaumoko 2D (Carr, 2004)

The buildings were studied for a scenario representing a high seismic region with a Maximum Credible Earthquake (MCE) equal to a Mw 7.5 with an expected PGA = 0.7 g for a return period of 2500 years. For ordinary structures, such a seismicity level, corresponding to an event with 2% in 50 years probability of exceedence, would typically correspond to a Collapse Prevention Limit State, meaning that the structure should not collapse, but it may not be possible to economically repair after the earthquake (Priestley et al., 2007). However, when dealing with Tall Buildings hosting such a substantial number of residents/workers, it could be argued that the importance level should be higher than that used for ordinary residential or commercial/office buildings, requiring a higher performance level for the same intensity. As a result, a 2500 year return period might require the adoption of a Life Safety limit state, as it would typically done for standard structures under a 500years return period event. Furthermore, since the types of building considered in this study were quite different, also covering a wide range of fundamental periods, the scenario was selected in order not to favour the response of any of them. In fact, a soil-class B/C was chosen in the design, corresponding to a good condition and no soil amplification effects expected and the distance to the fault set at 20 km, therefore on the border line between a far fault (FF) and near field (NF) event, considering that FF events would be more demanding for higher fundamental periods and NF events would be more critical for smaller and stiffer buildings. It is acknowledged that the selected scenario is definitely not exhaustive to generalize the results and a wider combination of case study buildings should be used to better understand the response of the different typologies of buildings and structural systems.

2.2 Design methodology

Both wall and frame systems were redesigned according to the direct displacement-based design as proposed in Priestley et al., 2007. The aim of this approach was to explore the potential of the DBD procedure on structures taller than the ones used to validate the method, in order to highlight its possible limits, as presented in details in Palmieri, 2010. The tallness of the buildings became relevant while assessing several design parameter: i.e. equivalent viscous damping ξ_{eq}, displacement shape of the MDOF system Δ_i and design displacement spectrum S_d. Furthermore, in case of PRESSS hybrid connections, the equivalent damping is not only dependent on the structural system in use (i.e. wall or frame in this study, Eq.(1)) but also varies according to the relative contribution between the re-centring (post-tensioning and/or axial load) and the dissipation (mild steel or other devices), defined by the parameter λ. The resultant hysteresis damping can thus be expressed as a weighed sum of the two main contributions as shown in Eq.(2) and (3). At the time of writing no specific work has been developed for the calibration of the equivalent damping ξ_{eq} for taller structures, therefore for both traditional design and hybrid connections the damping values were addressed as in the existing literature, Priestley et al., 2007 and NZCS, 2010 respectively.

$$\xi_{eq} = 0.05 + kost.\left(\frac{\mu - 1}{\pi\,\mu}\right) \qquad \textbf{(1)}$$

$$\xi_{Hybrid} = \frac{M_{PT} + M_N}{M_{total}} \cdot \xi_{PT} + \frac{M_s}{M_{total}} \cdot \xi_s \qquad \textbf{(2)}$$

$$\xi_{PT} = \frac{M_{PT} + M_N}{M_{total}} \cdot 5\% \quad \text{and} \quad \xi_s = \frac{M_s}{M_{total}} \cdot \left[5\% + 30\left(1 - \frac{1}{\sqrt{\mu}}\right)\%\right] \qquad \textbf{(3)}$$

Furthermore, the displaced shapes represent mainly the inelastic shape at the target displacement Δ_d, i.e. the inelastic mode shape of the building; thus assuming that higher mode have low participation mass and, for their shape function nature, do not substantially contribute to the maximum displacements can be less accurate when dealing with Tall Building. However, the DBD procedure itself already suggests some modifications to the original shape to take into account the higher mode effect for frame buildings, although validated for medium-height building, allowing a drift amplification, as suggested in Pettinga et al., 2005 and adopted in the latest version of the procedure in Calvi et al., 2009.

As per the displacement shape, also the lateral force distribution is typically proportional to mass distribution and displaced shape, and it may thus underestimate the contribution of the higher mode too. One of the proposed solution according to Priestley et al., 2007 , at least for frame building (calibrated with non-linear time-history analyses, NLTHA, on 16 storeys models), consists in re-arranging the lateral force increasing the contribution at the roof level by a 10% of the total base shear force and allocating along the building the remaining 90%. Therefore it should be expected that also for taller buildings, the scheme adopted to distribute the base shear along the height could have a non-negligible impact on the variation and uncertainties of their response.

2.3 Selection of ground motions and displacement spectra

A key factor of the DBD design procedure, to be integrated or validated with subsequent Non-linear Time History Analysis (NLTHA), is the selection of the seismic input in the form of a displacement design spectrum. The usual procedure suggests to refer to an elastic displacement response spectrum to retrieve the effective period of a structure by entering with the design displacement of the structure and selecting the curve corresponding to the appropriate reduction factor, function of the equivalent viscous damping as in Eq.(4). In order to have a better and more reliable approximation of the displacement demand (in terms of spectral ordinates) in the period range of the buildings under study, the authors preferred not to implement the displacement spectra proposed by codes (EC8 (CEN, 2004) or NZS1170.5 (NZS, 2004). Both code spectra are considered to be reliable in the 2-4 seconds period range, beyond which they do not give any reliable correlation between the target displacement and an effective period of the SDOF more than a constant plateau displacement (either equal to the peak ground displacement or kept constant from the maximum displacement at the corner period). For the characteristics of the site selected (high seismic zone with events up to Mw 7.5) and the high fundamental periods expected, the displacement spectra proposed by Faccioli et al. (2004) and computed with a database including also large earthquakes covering the long period range up to 10 seconds, were adopted in this study. The elastic displacement response spectrum for the proposed scenario, computed from Eq.(5) and (6) according to the aforementioned study, has a corner period T_c = 5.5 seconds and a peak displacement Δ_{max} = 989 mm as shown in Figure 4 with a dot-line.

$$\eta = \sqrt{\frac{7}{\left(2+\xi_{eq}\right)}} \tag{4}$$

$$T_c = 1.0 + 2.5\left(M_w - 5.7\right) \quad [s] \tag{5}$$

$$\delta_{\max} = C_s \cdot \frac{10^{\left(M_w - 3.2\right)}}{r} \quad [mm] \tag{6}$$

$$\eta = \sqrt{\frac{7}{\left(2+\xi_{eq}\right)}} \tag{7}$$

$$T_c = 1.0 + 2.5\left(M_w - 5.7\right) \quad [s] \tag{8}$$

$$\delta_{\max} = C_s \cdot \frac{10^{\left(M_w - 3.2\right)}}{r} \quad [mm] \tag{9}$$

Comparing the target design displacement at effective height required for the design of taller buildings and the maximum displacement demand provided by the proposed displacement spectrum as shown in Figure 4, it can be noticed that for buildings taller than 25st there is no direct relationship between displacement and effective period. Taller buildings are often characterised by very large yield displacement, therefore their target displacement can be actually limited by the spectral demand, rather then limiting ductility; this means that the intensity of the seismic motion is not sufficient to drive the building to the assumed level, or in other terms, the building will reach a much lower level of drift than the design level while also most likely responding in the elastic range. In spite of the high level of seismicity and the long return period selected, the design might still be dominated by the gravity design and/or the effect of building' slenderness (i.e. P-Δ effects). At the time of writing, the debate on the most appropriate design procedure for effective periods higher than the corner period is still open; in order to be consistent with the overall procedure followed in this work, the authors chose to retrieve the effective period by an extrapolation of the linear branch of the displacement response spectra as suggested in Calvi and Sullivan2009) through the following formulation:

$$T_{eff} = \frac{\Delta_d}{\Delta_{D\xi}} \cdot T_c \tag{10}$$

$$K_{eff} = \frac{4\pi^2 m_e}{T_{eff}^2} \cdot \frac{\Delta_{D0.005}}{\Delta_d} \tag{11}$$

The basic results of the whole design procedure are presented in the following chart (Table 1), where the equivalent base shear forces of the SDOF, to be distributed along the structure, are presented.

Details on the full design of the systems and of the structural elements design are detailed in Palmieri, 2010.

Table 1 DBD properties of the systems under study

	15 st		25st		35st		45st		
	Wall	**Frame**	**Wall**	**Frame**	**Wall**	**Frame**	**Wall**	**Frame**	
$\mathbf{m_{eff}}$	22284	30821	34336	50957	42546	71081	52679	91200	[t]
$\mathbf{H_{eff}}$	42.1	39.9	71.6	65.9	104.5	91.9	135.7	117.9	[m]
$\mathbf{\Delta_d}$	652	623	926	1038	1195	1451	1969	1869	[mm]
ξ_{sys}	14.4	13.5	10.1	13.6	5	13.6	5	13.6	[%]
$\mathbf{T_{eff}}$	4.80	4.35	6.65	6.92	6.57	9.85	10.0	12.5	[sec]

2.4 Modelling

Although torsional modes and effects have been noticed to affect the response of TB, for the main purposes of this work and in order to keep the simplicity of the study, the numerical models of the structures were analysed as 2D models (the symmetry of the buildings makes the approximation not far from a standard design approach). In fact, the attention is focused more on the comparison of the building response of two different systems (i.e monolithic and hybrid) rather than on a local and very detailed behaviour. With similar intentions the accuracy and simplicity of a lumped plasticity model was also preferred among many other modelling solutions. Furthermore, the implementation of jointed ductile PRESSS connections, characterized by a rigid body rocking where all the deformations are concentrated at a single section, fits quite well the hypothesis and approximation of the numerical model adopted, and is convenient for extensive parametric study on the seismic performance of complex MDOF structures and PRESSS systems (Figure 2).

This numerical approach, especially the emulation of the PRESSS systems connections, has been calibrated and validated against experimental results in a wide range of case studies and applications (presented in details in many studies at the University of Canterbury such as Marriott et al., 2007, NZCS, 2010). In particular for this work, the analysis were developed with the use of the software Ruaumoko2D (Carr, 2004).

2.4.1 *Modelling of traditional connections*

The structural elements of the “monolithic” or traditional systems were modelled as Giberson members (Carr, 2004) with plastic rotation concentrated in the plastic hinge regions at the end of the elements and modelled via appropriate hysteresis loops.

In particular the member properties of the frames were characterised by a *thin Takeda* hysteretic behaviour with an unloading stiffness coefficient $\alpha = 0.5$ and 0.3, for columns and beams element respectively, while a reloading stiffness coefficient $\beta = 0.2$. In the same way, the wall critical elements were calibrated with a hysteretic behaviour equivalent to a *fat Takeda* with α=0.1 and $\beta = 0.5$.

2.4.2 *Modelling of PRESSS connections*

The design of the PRESSS structural systems was kept similar to monolithic buildings presented earlier in this paper, whereas the intrinsic damping is actually lower for hybrid solutions and has a specific coded design (NZCS, 2010). On the other side, in the numerical modelling of this emulating solution the strong moment connection was substituted with a different technique, shown in Figure 2. The moment resisting capacity of structural members was modelled by elastic Giberson beam members and a series of rotational springs elements (refer to Carr, 2004) giving the moment capacity, as proposed and specified in fib (2003), NZS (2006) and NZCS (2010). The hysteresis loop of the PRESSS connection, in fact, has the typical flag-shape discussed earlier in this paper. This cyclic behaviour can be approximated as a combination in parallel of two or three rotational springs, depending whether it is a beam-column joint or a vertical element, e.g. column-to-foundation connection or wall system. The overall moment-rotation hysteresis flag-shape behaviour can thus be obtained by assigning a non-linear elastic hysteresis loop to the rotational spring representing the re-centring moment contribution of the post-tensioned tendons, M_{pt} (and where present of the axial load, Mn),and a dissipative hysteresis loop (elasto-plastic, Takeda, Ramberg-Osgood etc) to represent the moment contribution of the mild steel bars or generally speaking dissipation devices, M_s. The M_s contribution in the case of this study is assumed given by the mild steel grouted longitudinal reinforcement crossing the rocking sections, since the analysis were developed assuming the closest "emulation" of cast-in-place connections. It is worth noting that a general PRESSS (self-centring/dissipative) system can comprise alternative forms of dissipation and type of devices (external and replaceable instead of internally grouted), ranging from yielding-type, to friction, viscous or viscous-elastic. Clearly an appropriate hysteresis loop should be adopted to model such dissipative systems (Pampanin, 2005, Kam et al., 2008, NZCS, 2010).

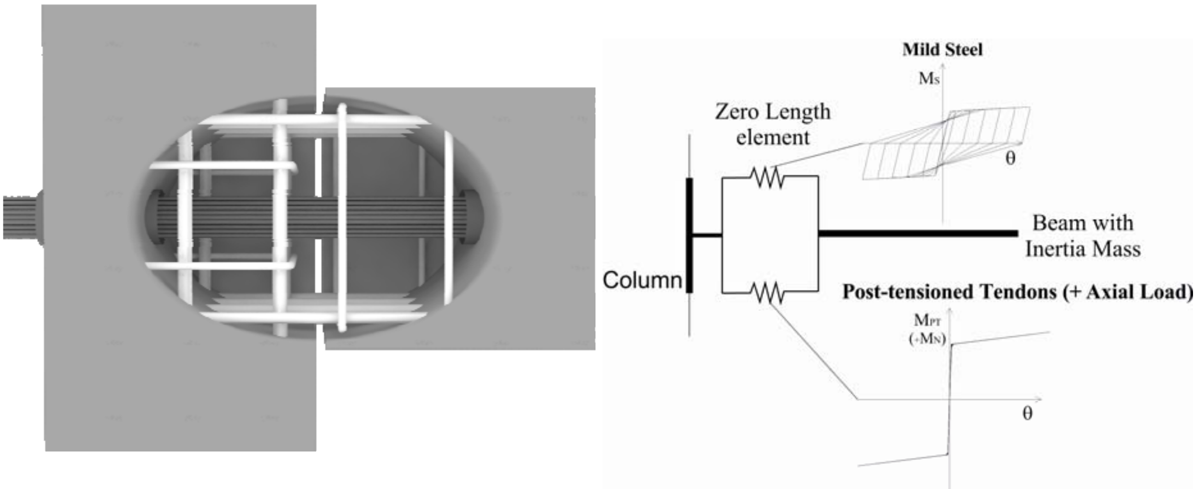

Figure 2 Sketch of the spring model for PRESSS (rendering courtesy of IFLA)

Figure 3 shows the design of one of the PRESSS wall-to-foundation connections where the flag-shape hysteretic response is given by the contribution of the three components Mpt, Mn and Ms and designed to match the moment-rotational envelope of the traditional cast-in-situ monolithic connection (only the vertical longitudinal reinforcement is shown for clarity and simplicity). In this section example is worth noting how the presence of post-tensioned bars allows evident reduction to the number of longitudinal reinforcing steel (heavily characterising the monolithic section), maintaining the same section capacity. The seismic performance of the TB designed with alternative connection solutions were finally assessed by a series of NLTHA in order to capture and compare the effects of higher mode and better represent the inelastic behaviour of the buildings. In the aforementioned 2D model scenario, a one direction dynamic excitation was implemented as input, ignoring vertical excitation. While the vertical propagation of seismic input in a Tall Building might be of great interest, due to resonance effect and dimension of the structure that may diffuse actions differently than a small building, the lack of investigation on this particular subject discourage from implementing an other possible source of uncertainties on the contribution of a PRESSS connection, at least at first analysis. Particular attention was given to the selection of ground motion inputs, given the expected high sensitivity of TB to the frequency content of the input signal as well as, to a minor extent, to the duration of the seismic input. In order to maintain a realistic frequency content of the input, a data set of natural ground motion, with characteristics similar to the selected scenario, were preferred instead of artificial accelerograms. The chosen ground motion inputs presented in Figure 4 were selected, based on their compatibility to the design spectrum, but also according to magnitude and fault rupture distance, out of a more comprehensive database used in design and experimental tests on jointed ductile PRESSS systems at the University of Canterbury. Within this database the accelerograms were selected and linearly scaled, when possible, in agreement with the indications of Bommer et al. (2004), accounting for the duration of the ground motion, whose importance is not recognized in a response spectrum. The necessity of further and more specific investigations on the design spectra to use in the case of long period structures can be seen in Figure 4, where displacement elastic spectra are compared against the code-based spectra. In accordance to the above considerations, in the modelling of the structures presented in this work, the damping level was selected at 1% of the tangent stiffness, according to the recommendations proposed by the LATBSDC (Los Angeles Tall Buildings Structural Design Council, 2008), stating that for building more than 50m and less than 250m in height, as in the case study, a damping ratio between 1% and 2% appears reasonable.

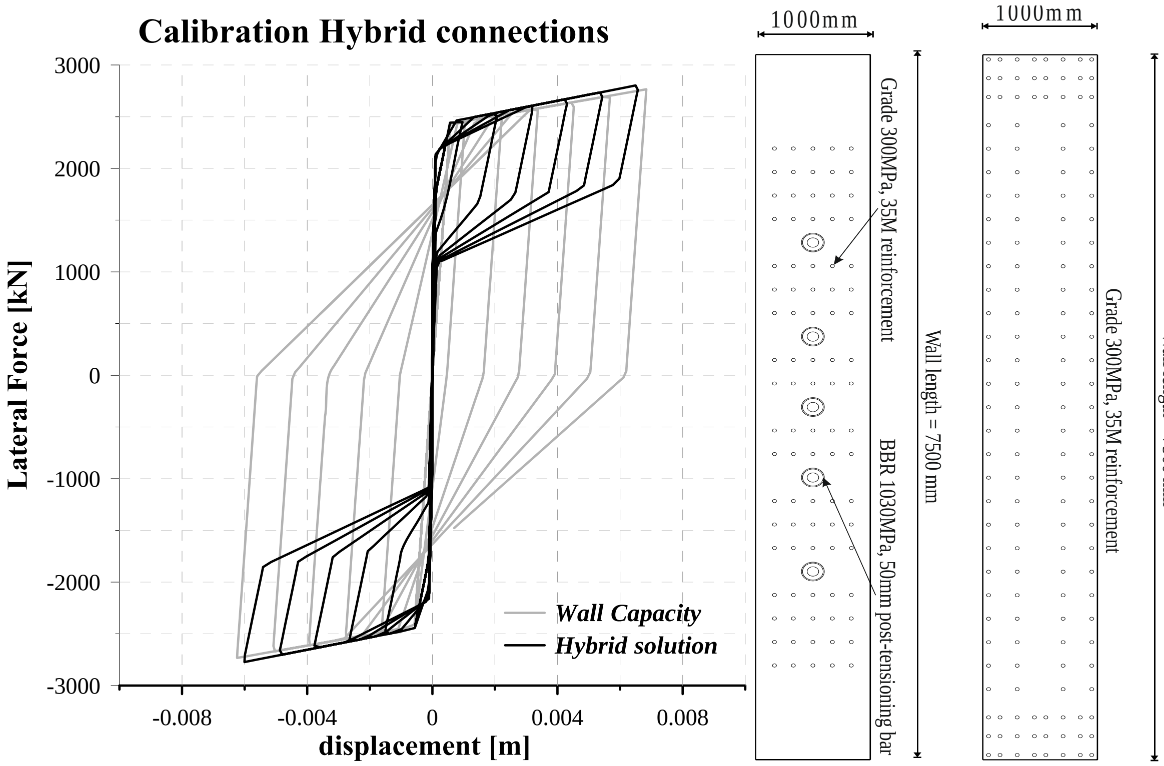

Figure 3 Wall-base connection: comparison of hysteresis behaviour of traditional (fat Takeda) vs. PRESSS connection (flag-shape). Comparison of a section design between a monolithic wall-base connection and its PRESSS counterpart

3 RESULTS

The observations derived from the analyses performed on the case study are herein summarized, in order to provide a brief overview of the limits related to the current approach to TB and focus the attention on the potentiality of the DBD and PRESSS-technologies. For the benefits of the discussion, the results are here shown in figures for the comparison between the two techniques, monolithic and hybrid solutions, for both wall and frame system. More details on the response at NLTHA of each solution can be found in Palmieri (2010).

3.1 Monolithic System Response

The displacement profiles resulting from the NLTHA developed in this work are summarised in Figure 4 and Figure 5. In both wall and frame systems, satisfactory agreement can be appreciated between predictions and actual responses, in average, for the 15 and 25 storey buildings. For the two remaining height levels (35st and 45st) the accuracy of the predictions becomes less satisfactory; in fact, not only the maximum displacement of the 35st structure is, in average, from 25% to even more than 50% less than the design value, but also the shape of the displacement profiles differs from the expectations. The difference between the prediction of the taller building and the actual response of the structures is far to be conservative. This lack of accuracy in the design phase may lead to miss the appropriate strength distribution along the building exposing the structure to unexpected seismic actions. Therefore, looking at those first results, the use of the coded inelastic displaced shape, validated only for lower structures, might be too much approximated and, as a consequence, the definition of the target displacement and effective stiffness can be overestimated.

Despite of the general good agreement shown by the displacement profiles for the 15st and 25st and much lower values (than the design one) for the 35st and 45st, the interstory drifts exceed the design limits assumed here as 2% already at lower floors for the 15st and 25st buildings and consistently at the higher floors for the 35st and 45st buildings. It is clearly noticeable how the upper part of the systems defects beyond the design limits; the increasing height leads to an earlier crossing of the drift limits but the overall drift shape tend to maintain a peculiar shape with a distinct bending on the top, probably also due to the distribution of equivalent lateral forces along the building in the design process. Interestingly enough, although the residual displacement does not apparently depend on the height of the structures, at any level the amount of residual (not recovered or permanent) displacement is relevant, and sufficient to impair the functionality of the structure.

Looking at the performance of the structures, it is possible to notice the peculiar response of the wall systems when compared to the design assumptions: the double bended shape of the total moment envelopes is very different from the typical cantilever-type (first mode) design moment shape. Again, the contribution of, at least, the second and third modes is evident but only in the 15st building the moment prediction made by a capacity design is able to capture the response; already at 25st the design assumptions seems to be inadequate with overestimation in the lower part while (unconservative) dangerously underestimation in the upper part. For the taller buildings (35st and 45st) the design assumption are clearly far from the actual response under seismic loading, and more importantly on the unconservative side. On the other side the behaviour of frame monolithic systems seems to catch the trend on the moment designed, although the demand is higher than predicted. Both systems results confirm the crucial need for more comprehensive investigations on the seismic response of Tall Buildings.

3.2 PRESSS systems

Since the design and the hypothesis on the structural response for PRESSS connection were kept similar to the emulated cast-in-place solutions (only the numerical model was developed using the rotational spring instead of moment-curvature models), the seismic response of the PRESSS building systems were expected to be very close to the traditional systems. In fact, having a moment-rotation capacity identical in its backbone curve to the original system, the only significant parameter varying was the lower level of hysteretic damping due to the typical Flag-Shape, as aforementioned when compared to the Takeda model adopted for the cast-in-place solutions. The diminished damping should therefore lead to larger displacements and drift levels. However the results, summarized from Figure 5 to Figure 9 for the 15st-25st-35st and 45 storeys, for both wall and frame systems respectively, emphasise that the predicted increments in displacements and drift levels are instead very close to the traditional configuration; while an evident benefit in reducing the total residual displacement and maintaining similar shear and moment response propose the hybrid solution as a valid alternative to the monolithic technique. The main advantage of using a post-tensioned solution was limited to reducing the residual displacement. Although residuals are one of the main concerns after an earthquake, the exceeding of drift limits and the uncontrolled response at the higher mode effects are still open issues to be assessed with possibly more advanced post-tensioned designs. As already discussed for the monolithic system response, the good agreement between the results of the 15 and 25 storeys and the design makes significantly reliable the comparison. On the contrary, although the comparison between PRESSS connections and monolithic system for taller buildings seems to follow the same trends shown for smaller structures, the reliability of those results are affected by the uncertainties of the overall building response. On the other side, the design of the structural elements is no more restrict to the moment capacity design, and the limits on the designing phase may be overcome by the flexibility of the post-tensioning system, e.g. the second configuration of hybrid wall systems proposed in the previous comparison. Furthermore, by a quick design of the structural member sections, with and with-out post-tensioned tendons, is clearly understandable how the same capacity is much feasible and cost-effective when the PRESSS connection is adopted. If in the case of wall systems the easing of the connection and the overall workability of the PRESS-system are remarkable only at the base sections of the buildings, when tackling the design of frame systems, the hybrid solution could improve many of the connections multiplying the benefits locally and performing as good as the monolithic standard solution, if not even better

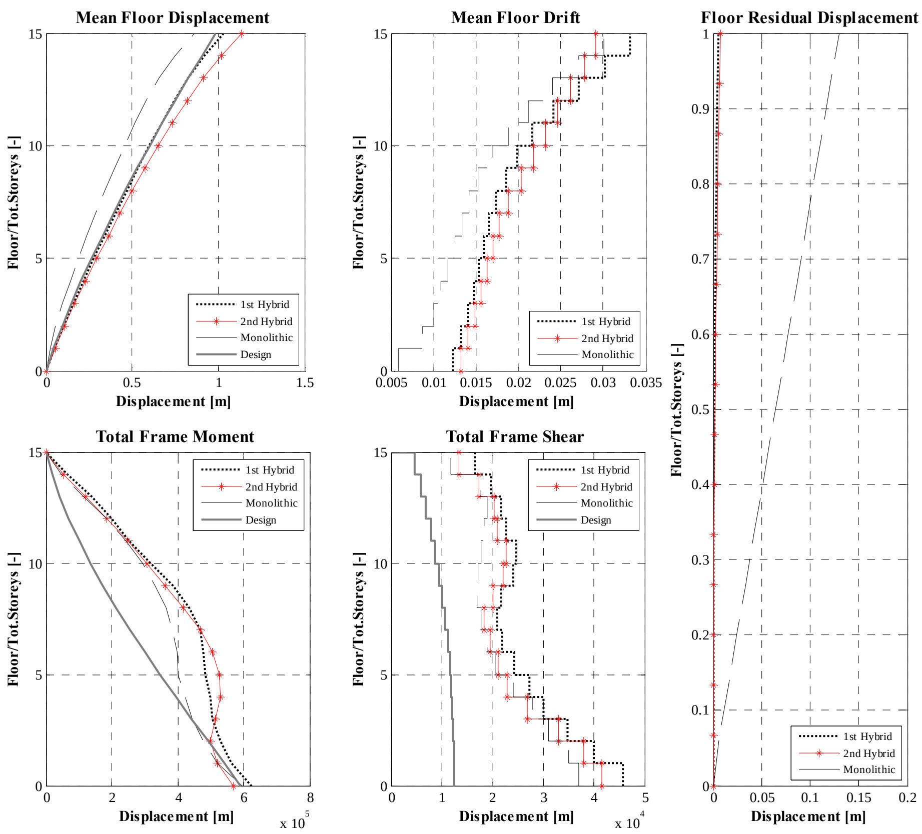

Figure 4 Comparison between cast-in-place and PRESSS solution (pure emulation and refinement of the post-tensioned connection) for 15st. wall system

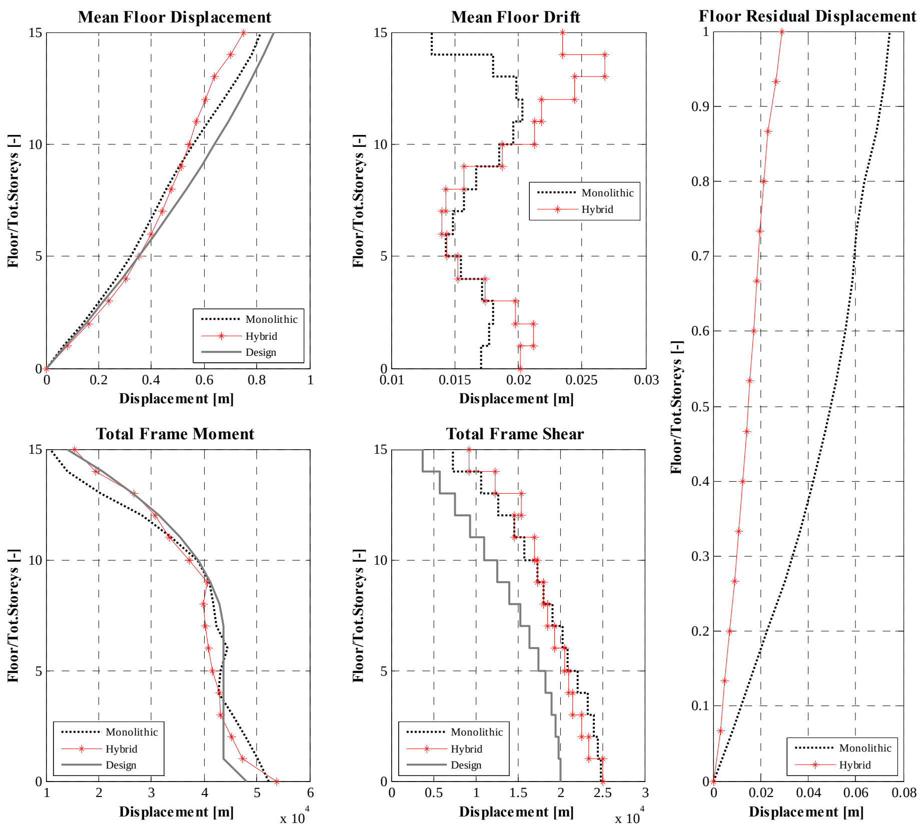

Figure 5 Comparison between cast-in-place and PRESSS solution for 15st. frame system

4 CONCLUSION

The main objective of the research project summarized in this paper was to numerically investigate the feasibility and structural efficiency of implementing PRESSS technology in high-rise building when compared to commonly used and well developed monolithic solutions which have already shown excellent performances and cost effective benefits. Most of the difficulties affecting the comparative evaluation of the performances are not directly related to the potentiality of one system versus the other but are more associated to the complexity of the problem of Tall Building Design and Response for which there is a critical lack of guidelines at international level.

The results of NLTHA developed for the series of case studies on wall and frame systems at different height levels showed a good agreement with the prediction for structures below the 25storeys while the accuracy abruptly decreased for taller structures. Those results, though quite unexpected, make sense with the fact that the displacement based design procedure has not been yet calibrated to predict response of building taller than 20storeys. The key issues are therefore the displacement shape approximation and the shear force distribution that lead to a moment and shear capacities far from the predicted response, even where the capacity design is enforced. These parameters in addition to the low control of the higher mode effects influence the results more than parameter as the intrinsic damping or the selection of ground motions, input of the NLTHA.

Based on these first analyses, the proposed concept of using PRESSS-technologies in Tall Buildings resulted to be already efficient and comparable at the cast-in-place solution, with a significant reduction of residual displacement and in terms of constructability and low congestion of the critical connections, as the post-tensioned tendons increased the capacity of the elements, while significantly reducing the amount of traditional mild steel reinforcement. Furthermore, PRESSS solutions could offer a higher flexibility in the design that can be easily tuned even during the life of the structure. The actual convenience of this concept needs a more accurate investigation, since by now it has been tested mostly for lower (up to 5-7 storeys) buildings, with the sole exception given at the moment by the 29-storey building in San Francisco). In fact, in the present study was only analysed the emulation of the original structures while the design procedure and the advance technology, as external dissipation devices or multi-rocking solution, are still tested only for low-rise buildings.

The current results of the analysis are still not sufficient to assess if the somehow inadequate approximation of the response for Tall Buildings in the range of 35-45 storeys is coming from a limit of the structural systems itself or from the limited analysis and case studies under examination. The interaction of different resisting systems is still a resource for the designer to provide better performances, but introduce another level of uncertainties in the whole procedure. Wall-frame systems or coupled walls and other systems are surely more efficient than simple wall or frame alone but they require a good knowledge of the basic systems as staring point.

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