SOLUTION USING HYBRID CONNECTIONS (PRESSS TECHNOLOGY) TO IMPROVE THE SEISMIC BEHAVIOR OF INDUSTRIAL BUILDINGS

Andres Reyes Bonilla, ROSE School, IUSS Pavia, Pavia, Italy
Stefano Pampanin, Dept. of Civil Engineering, University of Canterbury, New Zealand
Davide Bolognini, Centro Europeo di Formazione e Ricerca in Ingegneria Sismica-EUCENTRE, Italy

ABSTRACT

Improvements on high-performance seismic resistant systems have been observed in the recent past providing structural elements able to sustain major ground motions with limited levels of structural damage. The development of precast concrete hybrid system/connections (PRESSS Technology) exhibiting "flag-shape" behavior, characterized by the combination of self-centering and energy dissipation capacity, significantly improve the use of precast concrete structures around the world, however there is no evidence of its implementation on precast industrial buildings. In this contribution, the consequences of the different choices and assumptions made all along the use of Force Based Design (FBD) and Direct Displacement Based Design DDBD methodologies for designing concrete industrial buildings are assessed. The methodologies are applied to simple structures (SDOF system) and then solutions using PRESSS technology and reinforced concrete (RC) are implemented to reach an optimal structural strength to achieve a given performance limit state for a specified seismic intensity. This paper shows results of good control of limit displacements using DDBD and precast concrete hybrid connections, as well as the consequences of the design of the structure using FBD. Additionally, some important results like column moments, storey shear distribution and yield displacements for the study cases are shown. A design example, using the DDBD and FBD on an industrial building is presented.

Keywords: Hybrid Connection, Industrial Building, Displacement Based Design

INTRODUCTION

Precast frame buildings are used throughout the world for industrial, commercial and manufacturing facilities. These structural systems have many advantages for these specific uses like providing large open areas (20m-30m) and high storey heights (6m-8m).

Statically determined structural schemes (Fig 1) with beams simply supported or hinged on cantilever columns are the solutions typical and widely adopted in the construction practice of Mediterranean countries like Italy, Greece, Spain Turkey, etc. However, for seismic design there is a controversial debate around the expected performance of this type of precast structures, principally due to the limited redundancy of the system.

(a) (b)

Fig. 1. Typical Precast Industrial Building, a) Turkey[1], b) Italy[2].

The potentially high vulnerability of those precast industrial buildings is related to the absence of a moment-resisting-frame scheme, which could lead to significant structural deformability and expected increase of structural and non-structural damage. Additionally, an inadequate seating length of roof beams on columns or any weakness of the hinge connection (varying appreciably from producer to producer) can easily result in a partial or total collapse of the structure.

Several alternative solutions to provide moment-resisting connections between precast elements for seismic resistance have been studied and developed in literature, however in the European countries they are not used for precast industrial buildings.

In typical emulation of cast-in-place concrete solutions, as adopted in Costa Rican construction practice (Fig 2), the connections are localized within the beam-column joint with total casting-in-place of concrete, this technique provide an equivalent "monolithic" connection (wet connection).

These latter systems have not been adopted at international level as much as they could have for several factors. These factors depend on the wet connections cost efficiency, mainly associated to workmanship and labour, and the reduction of construction speed. However, in high-seismic regions the redundancy provides by the moment connections reduce significantly the moment and displacement demands on the structure and therefore the total cost.


(a)


(b)

Fig. 2. Precast Industrial Building in Costa Rica. a) Main span b) Roof beam-column wet connection

In spite of the clear merits of such precast solutions in some high-seismic regions, the well-recognized advantages of precast construction, namely quality control, construction speed and costs are not fully exploited. Due to these, an alternative developed in the 90's (PRESSS technology) based on dry jointed ductile connections capable of achieving high performance (low-damage) at low-cost has been analyzed to provide moment resistance joints (roof beam-columns) on industrial buildings.

DESCRIPTION OF THE VIRTUAL BULDINGS CONSIDERED

In order to define the most representative structural typologies of industrial buildings developed in Italy a geometrical configuration is used as described by *Bolognini et al*[3] (Fig. 3), where they explain the traditional way to build this kind of structure in Italy.

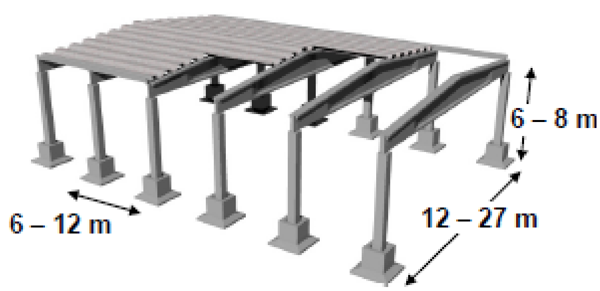


Fig.3. Basic main structural typologies used in Italy for one-storey structure with portals.

The virtual industrial building selected for the analysis has been characterized by long-span roof girders of 24m and four longitudinal bays of 10m each. The Storey height is 7.5m from floor level (Fig.4 and Fig. 5).

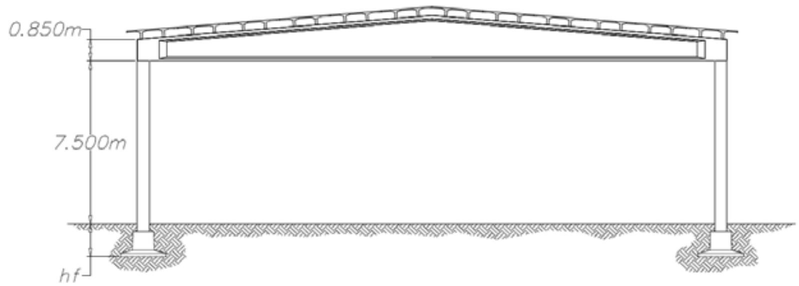


Fig 4. Main span elevation of case study

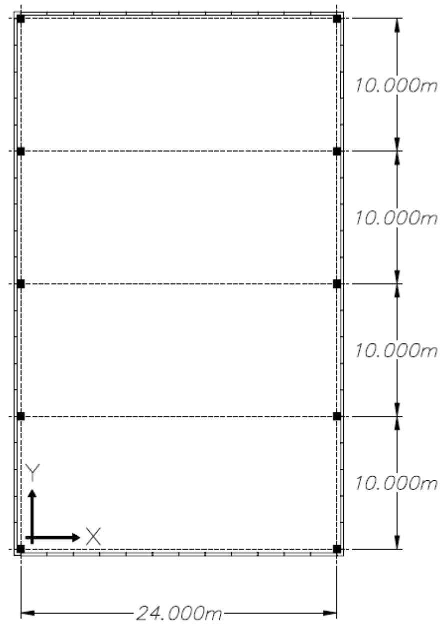


Fig.5. Plan view of case study

The base of each precast column is grouted in a socket footing (typically precast) to form a fixed connection. Long-span roof beams are oriented along the transverse axis of the building. The depth of these beams varies along their length, forming a triangular shape. Gutter beams were oriented along the longitudinal axis of the building to collect water from the roof.

The use of perimeter shear walls or rigid frames are not a common and largely utilized solution in a precast industry, due to this fact the connection detail is assumed such that the wall panels do not contribute to the lateral stiffness of the building but they contribute to the seismic masses.

DESIGN CRITERIA AND ASSUMPTIONS

Properties of materials, loads (dead, live, wind, snow, etc), load and mass combinations, seismic design spectra and other criteria are based principally on Eurocode 2[4] and Eurocode 8[5] .

To calculate the actions on the structure permanent and temporal loads of $0.2kN/m^2$ and $0.5kN/m^2$ respectively have been used. Additional wind loads for Terrain category III (q_b=490 N/m^2/) and snow loads (s=$0.64kN/m^2$) for zone II in Italy are assumed.

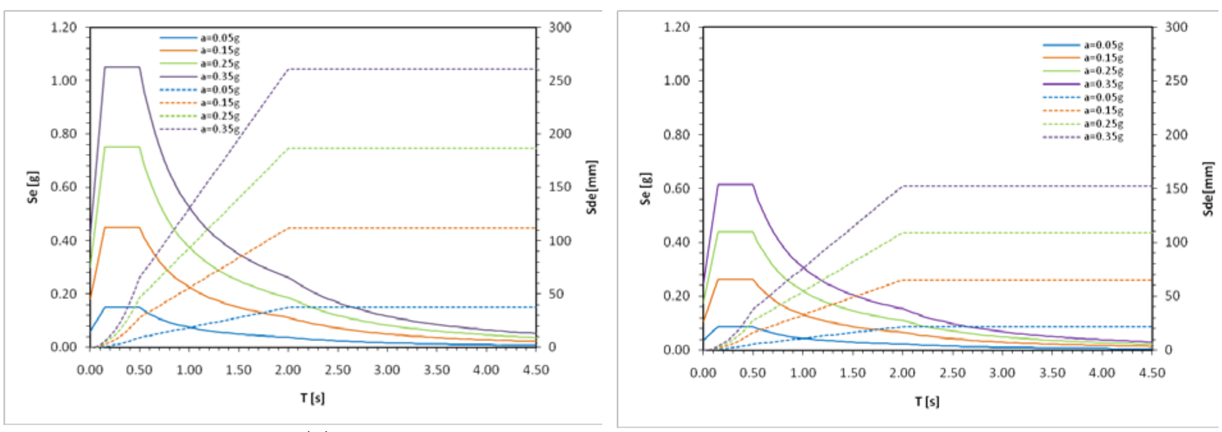
To calculate the seismic masses (see Table 1) equation 1 is used where the self-weight of the elements, snow and permanent loads are included. For vertical façade panels approximately 60% of its mass is considered.

$$\sum_{j\geq 1} G_{k,j} + P + A_{Ed} + \sum_{j\geq 1} \psi_{2,i} Q_{k,i} \qquad (1)$$

Table1. Seismic masses for case study

Item	**Mass (ton)**
Gutter Beam	2.55
Roof Beam	8.81
Double Tee Beam	9.26
Columns(500mmx500mm)	2.36
Panels	21.00
Snow	3.08
Permanent Load	2.4
Total	**49.46**

For the horizontal components of seismic actions, the elastic acceleration response spectrum for ultimate limit state (ULS, No-Collapse) and serviceability limit state (SLS , Damage control) are defined from Eurocode 8[5] associated with a reference return period of 475 years and 95 year respectively (Fig. 6).



(a) (b)

Fig.6 Acceleration and displacement design spectra a) No-collapse b) Damage Control

The ultimate limit displacement is related to the sensitive coefficient, which is defined as the maximum displacement allowed for which the equation 2 is accomplished

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \leq 0.1 \tag{2}$$

The limitation of the interstorey drift for service limit is taken as $d_i = 0.01 * H$, considering buildings having non-structural elements of brittle materials attached to the structure.

FORCE BASED DESIGN (FBD) VS DISPLACEMENT BASED DESIGN (DBD)

Current force-based design has many fundamental problems with the procedure, particularly when applied to reinforced concrete structures. In order to examine these problems, it is necessary to review the design procedure (Fig 7) and the example shown below, as currently applied in modern seismic design codes.

The fundamental difference between DDBD and FBD is that DDBD characterizes the structure to be designed by a single degree of freedom representation of performance at peak displacement response, rather than by its initial elastic characteristics. The design procedure determines the strength required at designated plastic hinges locations to achieve the design aims in terms of the defined displacement objectives.

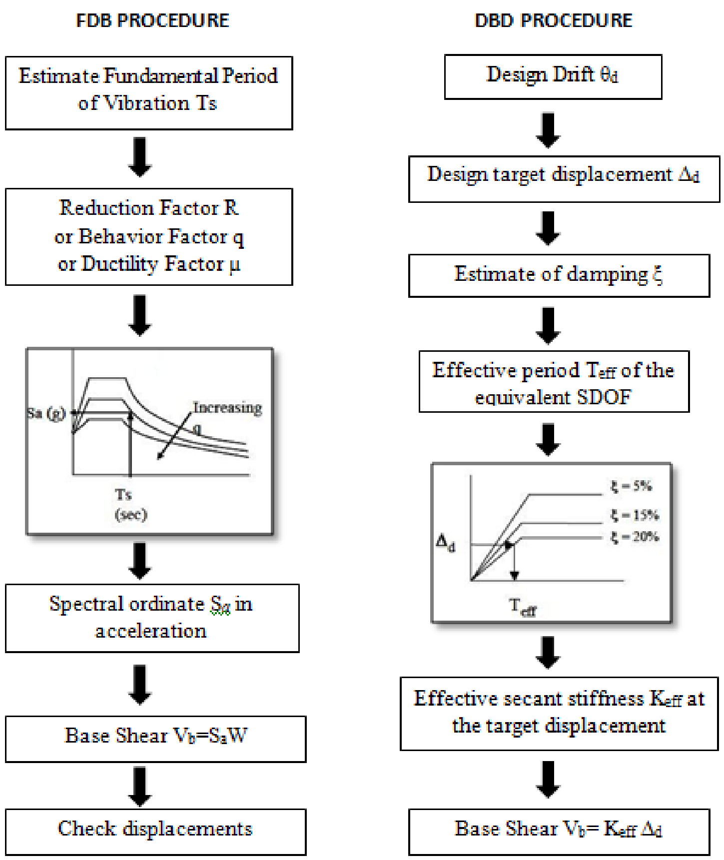


Fig 7. FBD and DDBD chart flow design procedure. Pampanin et al[9]

Following systematically Eurocode to design a top pinned structure (cantilever) under specific conditions (Soil B, a_g =0.35$_g$ and ULS) the next example is presented to compare and choose the best design methodology for industrial buildings using the design criteria and assumptions described above.

Following the force based design procedure SAP2000 has been used to model the structure and to obtain the elastic (assumed) period. The inertia has been reduced, in all non prestressed elements (columns) EIc=0.5EIg .The elastic period is $T_{elastic}$=1.372s.

Once having the elastic period the base shear (Eq 3) and moment (Eq 4) are calculated using a reduced design spectrum with q=3 and assuming H_e equal to building height.

$$V_b = S_a(T) * g * m \tag{3}$$
$$M_b = H_e * V_b \tag{4}$$

The last step of FBD procedure is to check the displacement and satisfy an interstorey sensitive coefficient less than 0.1 ($\theta \leq 0.1$). In table 2 there is a summary of the most important characteristics and results from the analysis.

Table 2. Analysis results for a structure wirh columns 600mmx600mm

Seismic Zone	V_b[kN]	M_b[kN-m]	d_e [mm]	d_i [mm]	θ
a_g=0.35g	64.17	527.72	61	183	0.0853

Normally designers using FBD methodology never review the displacement with another analysis procedure, however with the intention to prove the unrealistic design assumption of constant stiffness, a pushover analysis (first trial) has been developed to check displacements and ductility demands through the comparison of S_a-S_d curve (ADRS)

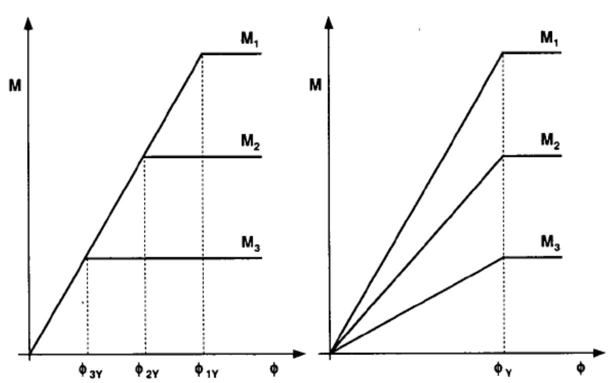


Fig 8. Influence of strength on moment-curvature relationships. (a) Design assumptions, constant stiffnes (b) Realistic conditions, constant yield curvature. Priestley et al[6]

The design assumption shown in figure 9(a) has been based on equal displacement theory, as well described by Paulay and Priestley[7]. As it is clearly noticed in the figure 9(b) the first trial stiffnes is less than the assumed at the beginning of design, this means that ultimate displacement is bigger, thus, the ultimate limit state is not satisfied because the sensitive coefficient is greater than 0.1. Therefore FBD procedure has to be followed again using greater stiffness (increasing cross section or steel ratio). Figure 9(b) shows different pushover trials for each design iteration. For third trial the sensitive coefficient is satisfactory, and the base shear and moment demand using an equivalent q=1.25 (μ=1.25 instead of μ=3 initially assumed) are respectively, V_b=118.4kN and M_b =973.4kNm on each column. These results show the poor efficiency of FBD to control de displacement and therefore the forces demands.

For countries with fixed target displacement (in Costa Rica for frames is used 2%) it is more critical the use of an implicit and unreal assumption of constant member stiffness (which implies that the yield curvature is directly proportional to flexural strength), because there will be the need to run several iterations until reaching a stiffness value equal to the initial assumed.

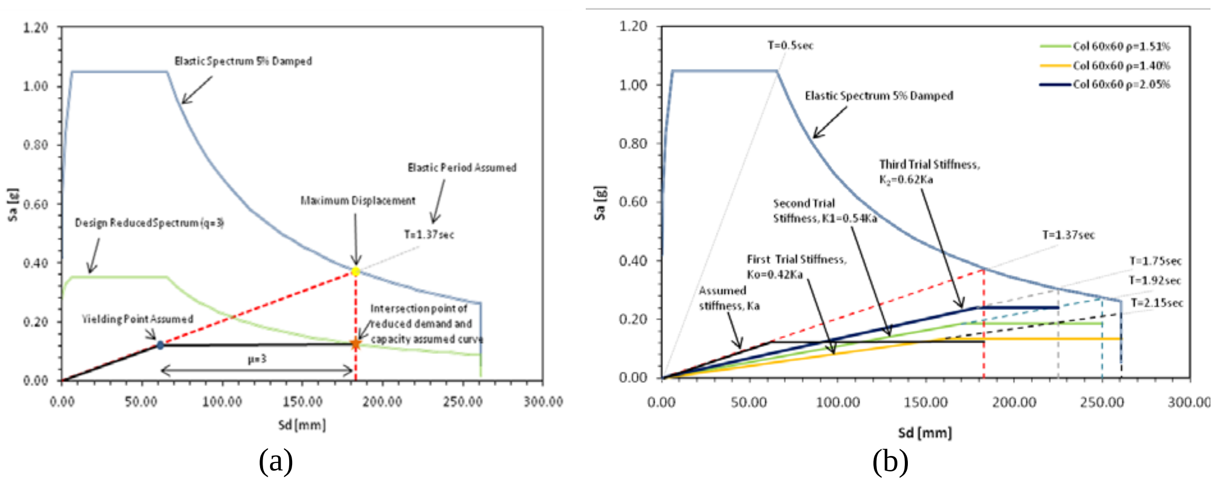


Fig.9. S_a-S_d vrs pushover analysis to compare the behavior of FBD methodology (a) initial design assumption for q=3 (b) analysis of different structure stiffness (iteration).

Direct displacement based design is used as an alternative to design precast industrial buildings. This methodology is supported by the assumption that the yield curvature is essentially independent of strength for a given section (see Fig 8 (b)).

As first design step is necessary to know the target displacement Δ_d or drif θ_d, which normally are fixed and defined by codes, however, as described above Eurocode 8 uses the sensitive coefficient as a parameter to control ultimate displacements. Due to the intention to prove the effectiveness of DDBD Δ_d=225m is taken from FBD example, nevertheless this value will have to be iterated for another cases.

Given the target displacement, material properties, axial load and seismic masses the ductility demand expected using equations 5 and 6 is found. A single degree of freedom (SDOF) as shown in figure 10(a) was considered.

$$\Delta_y = \emptyset_y * \frac{(H + L_{sp})^2}{3} = 7.875E^6 * \frac{(8220mm + 277.2mm)^2}{3} = 189.5mm \tag{5}$$

$$\mu = \frac{\Delta_u}{\Delta_y} = \frac{225mm}{189.5mm} = 1.19 \tag{6}$$

The equivalent viscous damping can be obtain using a graph equivalent to the figure 10(c) or applying the equation 7 with which the reduce factor is calculated (Eq 8). This factor converts the elastic displacement spectrum to an inelastic one (see fig 10(d)).

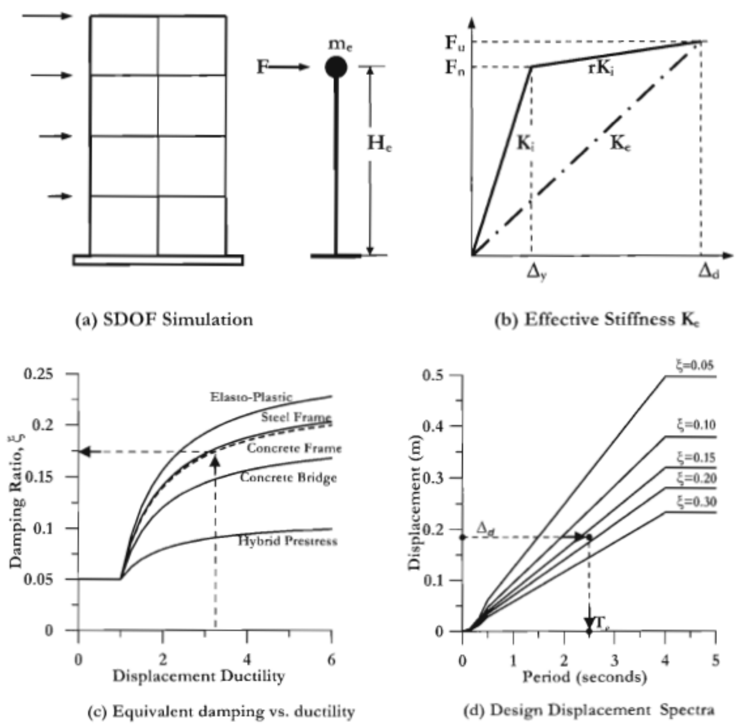


Fig 10. Fundamentals of direct displacement based design Priestley et al[6]

$$\xi_{eq} = 0.05 + 0.444 * \left(\frac{\mu - 1}{\mu\pi}\right) = 0.05 + 0.444 * \left(\frac{1.19 - 1}{1.19\pi}\right) = 0.0723 \tag{7}$$

$$\eta = \sqrt{\frac{10}{5 + \xi}} = \sqrt{\frac{10}{5 + 7.23}} = 0.87 \geq 0.55 \tag{8}$$

The effective period is taken from figure 11 and the effective stiffness is calculated using this period (Eq 9) (for this specific case T_{eff}=1.95sec). The last step to complete DDBD procedure is to obtain the base shear and moments on each column, for that purpose the results of equations 10 and 11 are divided by 2 (because those values are for complete structure). Base shear and moment on each column are V_b=114.78kN and M_b=943.45kN-m.

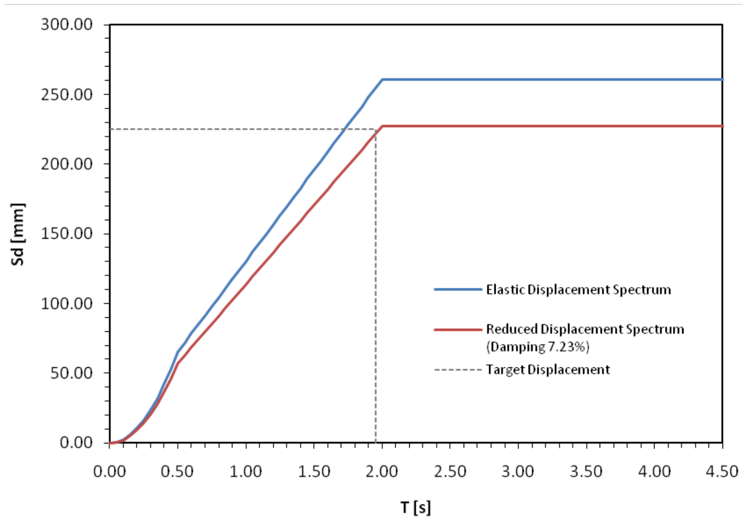


Fig 11. Reduced displacement spectrum and effective period

$$K_{eff} = \frac{4\pi^2 m}{T_{eff}^2} = \frac{4\pi^2 98.26ton}{1.95^2} = 1020.2 \text{ kN/m} \quad (9)$$

$$V_b = \Delta_d * \mathrm{K}_{eff} = 0.225m * 1020.2\frac{kN}{m} = 229.55kN \quad (10)$$

$$M_b = H_e * V_b = 8.22m * 229.55kN = 1886.9kN\ m \quad (11)$$

As it is verified through the results obtained from DDBD, this methodology provides an accurate procedure to design precast industrial buildings, controlling principally displacements and giving similar force demands than iterative FBD with a less laborious process.

SEISMIC DESIGN OF PRECAST INDUSTRIAL BUILDING

In the present study seismic designs with 3 different concepts have been done to analyze, compare and find the best solution for precast industrial buildings. The alternative proposes are the following, use of cantilever columns (top pinned connections), monolithic joints at top and bottom of columns (double curvature columns) and the hybrid solution (PRESSS Technology).

To verify the design results, non-linear time-history analyses have been developed for each case using RUAUMOKO. An ensemble of 7 Earthquake records from PEER record database are selected (Table 3).

A wavelet-based procedure has been used for the generation of spectrum compatible time-histories, WAVEGEN is one such computer program developed by Mukherjee and Gupta[8]. This wavelet-based procedure (program) uses the decomposition of recorded accelerograms in a desired number of time-histories with non-overlapping frequency content, and then each of the time history has been suitably scaled to match the response spectrum of revised accelerogram to the specified design spectrum (Fig12).

Table 3. Properties of selected ground motions from PEER data base

Earthquake event	PEER ID	Epicentral Distance [km]	Mag	Depth [km]	PGA [g]
Northridge 1994	NORTHR/PKC090	19.28	6.69	17.5	0.3482
Loma Prieta 1989	LOMAP/YBI090	95.16	6.93	17.5	0.0557
Victoria, Mexico 1980	VICT/CHI192	36.67	6.33	11.0	0.1179
Landers 1992	LANDERS/CLW-TR	82.12	7.28	7.0	0.3733
Imperial Valley 1940	IMPVALL/I-ELC270	12.99	6.95	8.8	0.2584
Parkfield 1966	PARKF/C05355	32.56	6.19	10.0	0.3768
Trinidad 1980	TRINIDAD/B-RDE270	76.75	7.20	15.1	0.1474

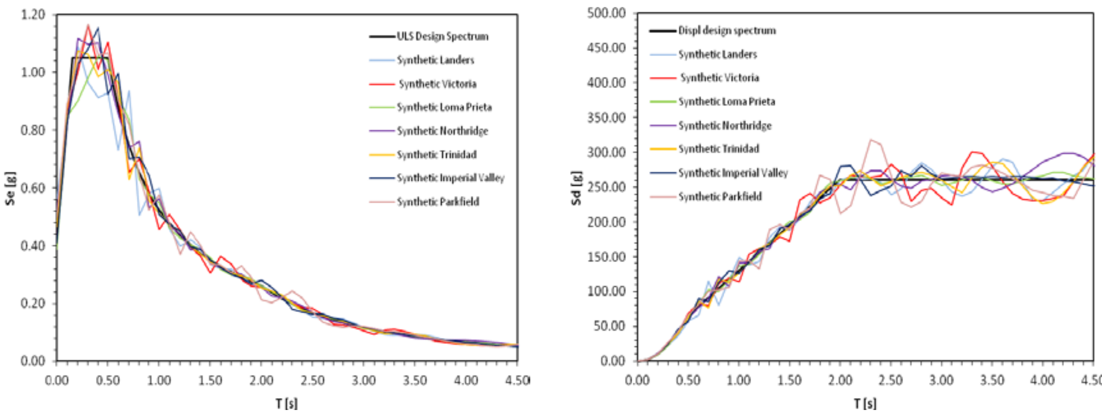


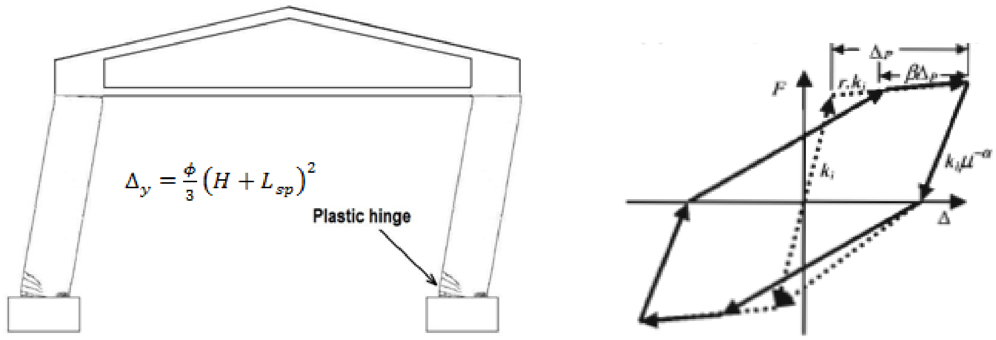
Fig 12.Synthetic acceleration and displacement spectra

The design method aims to control the inter-storey drift as part of a performance-based design approach. As such, the success of the design proposal is gauged by comparing the recorded drifts to the target drift for each case study. Because scatter of results in non linear time histories is unavoidable, and because the design spectrum represents the average seismic demand for the hazard level under consideration, the average of maximum recorded drifts has been used for the comparison with targets.

SIMPLE SUPPORTED JOINTS AT TOP OF COLUMNS

Because the cantilevered columns provide all the lateral stiffness for the one-story buildings, the yield displacement and displacement capacity could be calculated using the scheme shown in fig 13(a), where Δ_y is the yield displacement at the roof level.

Fat Takeda modified rule (Fig 13(b)) has been used in the bottom of columns assuming than axial load for this structures is very low. The trend of these columns is to behave like vertical beams. This hysteretic rule has been taken into account for THA.



(a) (b)

Fig 13. Top pinned industrial building assumptions (a) Displacement expected scheme (b) Hysteretic rule-Fat modified Takeda. Priestley, et.al[6].

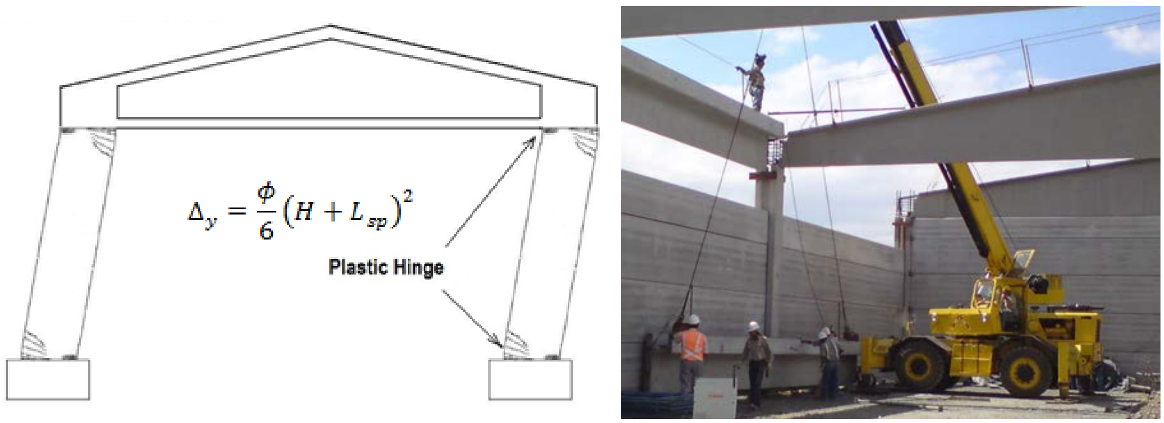
The table 4 shows the results from wind and seismic designs using FBD and DBD methodologies. Comparing shear and moment demands from design (ULS) is proved that FBD is unconservative; however further in this article it will be also corroborated through time history analysis (THA). For seismic zones with a_g=0.05g and a_g=0.15g the wind design governs, and just for the 2 highest zones (a_g=0.35g, a_g=0.25g) seismic design is necessary.

Table 4. ULS design results of one-storey industrial building for top pinned connection.

	Wind design governs (green) / Seismic design governs (blue)	**Top pinned connection**					
		Wind design		**Seismic Desgin (Soil B)**			
		Option 1	Option 2	a_g=0.35	a_g=0.25	a_g=0.15	a_g=0.05
FBD	Demands/Column						
	V_b [kN]	40.3	40.3	64.2	45.8	27.5	9.2
	M_b [kN-m]	335.7	335.7	527.72	376.98	226.18	75.44
	Ductility [μ]	-	-	3	3	3	3
	Interstorey Drift θ_d [%]	-	-	2.22%	1.59%	0.95%	0.32%
	Section Properties						
	B [mm]	500	600	600	600	600	600
	L[mm]	500	600	600	600	600	600
	ρ[%]	1.51%	1.01%	1.05%	1.01%	1.01%	1.01%
	bars	12	8	12	8	8	8
	diameter [mm]	20	24	20	24	24	24
DBD	Demands/Column						
	V_b [kN]	40.3	40.3	109.6	90.6	35.3	35.3
	M_b [kN-m]	335.7	335.7	900.8	745.0	290.5	290.5
	Ductility [μ]	-	-	1.2	1.0	1.0	1.0
	Interstorey Drift θ_d [%]	-	-	2.75%	1.69%	1.36%	0.45%
	Section Properties						
	B [mm]	500	600	600	600	500	500
	L[mm]	500	600	600	600	500	500
	ρ[%]	1.51%	1.01%	2.36%	1.69%	1.01%	1.01%
	bars	12	8	16	16	8	8
	diameter [mm]	20	24	26	22	20	20

MONOLITHIC CONNECTION AT BOTTOM AND TOP OF COLUMNS

In this case study the displacement shape shown in figure 14 is assumed. Knowing that the roof beam is prestressed, columns are made with normal reinforced concrete and the geometry of the beams has more inertia than columns (which average is 50 times greater), the displacement behavior might be taken as a valid proposal. However, this will be verified through THA.



(a) (b)

Fig 14. Displacement assumption for top and bottom fixed connections (a) Displacement expected scheme (b) Industrial building under construction (Costa Rica), top joint still unfilled.

In some countries like Costa Rica, the precast industrial buildings are constructed avoiding the use of cantilever columns and therefore, there is some evidence of the use of wet connections to join beams at top of columns. Figure 15, shows a schematic and real joint solution.

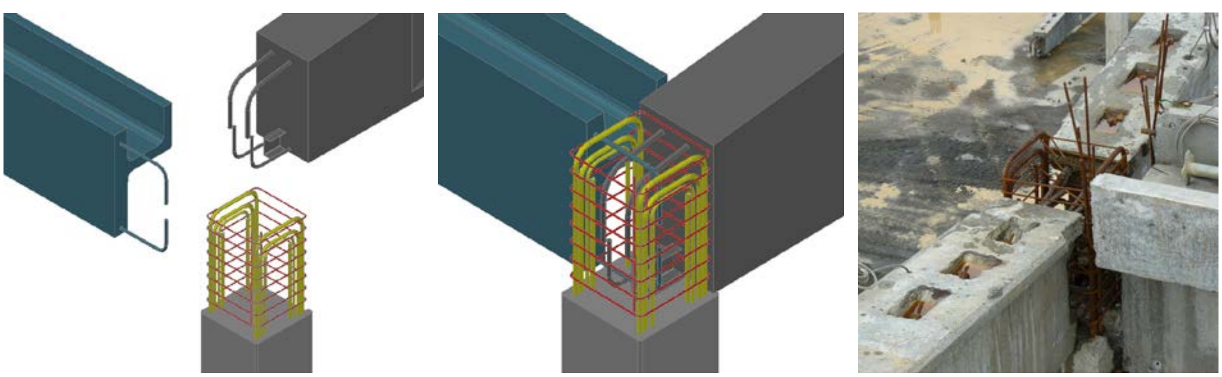


Fig 15.Fixed wet connections details

In Table 5 are presented the seismic and wind design for the condition of columns with both ends fixed. These results present similar characteristic than top pinned output shown above in terms of where governs wind or seismic and the exposing of unconservative FBD.

The most important output from table 4 is obtained by comparing top pinned results (Table 3) and both ends fixed results, where it can be observed that demands on columns with double curvature are almost half than those obtained for the cantilever case. The main advantage of having less demands on columns is the reduction of reinforcement and concrete colocated on elements (less cross sections area). Additionally , the dimensions of foundations will be less for the cases of both ends fixed, but this is not demostrated numerically in this manuscript.

Table 5. ULS design results of one-storey industrial building for top and bottom monolithic connection.

	Wind design governs / Seismic design governs	Top and bottom monolithic fixed connection					
		Wind design		Seismic Desgin (Soil B)			
		Option 1	Option 2	a_g=0.35	a_g=0.25	a_g=0.15	a_g=0.05
FBD	Demands/Column						
	V_b [kN]	46.2	46.2	69.32	49.51	29.71	9.90
	M_b [kN-m]	208.6	208.6	290.01	207.15	124.29	41.43
	Ductility [μ]	-	-	3	3	3	3
	Interstorey Drift θ_d [%]	-	-	2.00%	1.42%	0.86%	0.29%
	Section Properties						
	B [mm]	400	450	450	450	450	450
	L[mm]	400	450	450	450	450	450
	ρ[%]	1.57%	1.01%	1.79%	1.01%	1.01%	1.01%
	bars	8	8	8	8	8	8
	diameter [mm]	20	18	24	18	18	18
DBD	Demands/Column						
	V_b [kN]	46.2	46.2	92.6	74.2	37.3	37.3
	M_b [kN-m]	208.6	208.6	380.6	304.9	153.3	153.3
	Ductility [μ]	-	-	1.5	1.2	1.0	1.0
	Interstorey Drift θ_d [%]	-	-	2.32%	1.86%	1.36%	0.45%
	Section Properties						
	B [mm]	400	450	450	450	400	400
	L[mm]	400	450	450	450	400	400
	ρ[%]	1.57%	1.01%	2.10%	1.50%	1.00%	1.00%
	bars	8	8	8	8	4	4
	diameter [mm]	20	18	26	22	22	22

HYBRID JOINTS AT BOTTOM AND TOP OF COLUMNS

Hybrid system (PRESSS technology) is a particular and efficient solution with self-centering and energy dissipating properties, combined through the use of unbonded post-tensioning tendons and longitudinal mild steel (Fig 16). The peculiar dissipative-recentering hysteresis loop is also referred to as “flag-shape” (Fig 17(b)), which can lead to negligible residual deformations. The inelastic demand is accommodated within the connection itself (beam-column, column to foundation), through the opening and closing of an existing gap.

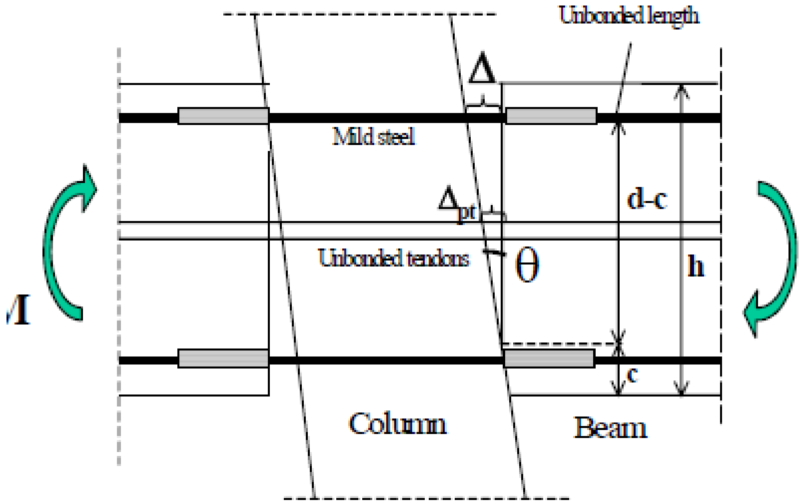
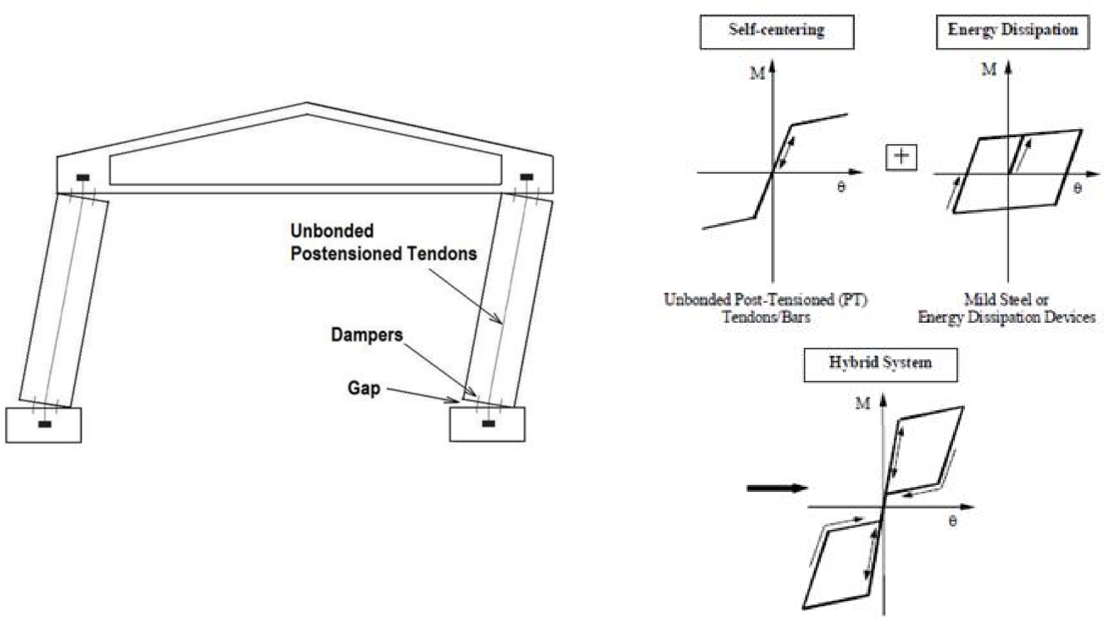


Fig 16.Connections mechanism of hybrid systems. Pampanin et al[9]

A conservative estimate of the yield rotation of a post-tensioned frame would be to use the same equation suggested for a monolithic frame (top and bottom with fixed connections). In the figure 17(a) are described the components and its locations on the system.



(a) (b)

Fig 17.(a) Hybrid system components (b) Flag shape hysteresis

Table 6 has the results of hybrid design using a conservative yield curvature as monolithic case. Despite this, sections with less amount of steel (kilograms) are obtained due to the use of prestressed steel, however bigger strength capacity is developed.

Table 6. ULS design results of one-storey industrial building using hybrid joints at top and bottom

		Top and bottom hybrid fixed connection					
(green)	Wind design governs	Wind design		Seismic Desgin (Soil B)			
(blue)	Seismic design governs	Option 1	Option 2	a_g=0.35	a_g=0.25	a_g=0.15	a_g=0.05
DBD Hybrid	Demands/Column						
	V_b [kN]	46.2	46.2	104.7	80.5	43.1	43.1
	M_b [kN-m]	208.6	208.6	430.5	330.8	177.2	177.2
	Ductility [μ]	-	-	1.71	1.34	1	1
	Interstorey Drift θ_d [%]	-	-	2.63%	2.02%	1.69%	1.69%
	Section Properties						
	B [mm]	400	450	450	450	400	400
	L[mm]	400	450	450	450	400	400
	λ	-	-	1.22	1.24	1.23	1.23
	ρ[%]	1.57%	1.01%	1.22%	0.89%	0.64%	0.64%
	bars	8	8	4	4	4	4
	diameter [mm]	20	18	28	24	18	18
	Tendons	-	-	6	4	2	2
	diameter [mm]	-	-	0.6	0.6	0.6	0.6
	$T_{pt,init}$ [kN]	-	-	150	155	155	155
	ρ_{total} [%]	-	-	1.63%	1.16%	0.81%	0.81%

In all hybrid designs has been used λ=1.25 (at least close to 1.25) to optimize at maximum the hysteretic energy dissipation. There is no evidence of previous hybrid design in industrial buildings, therefore figure 18 shows some sketches that clarify the construction process of the proposal expressed in this document.

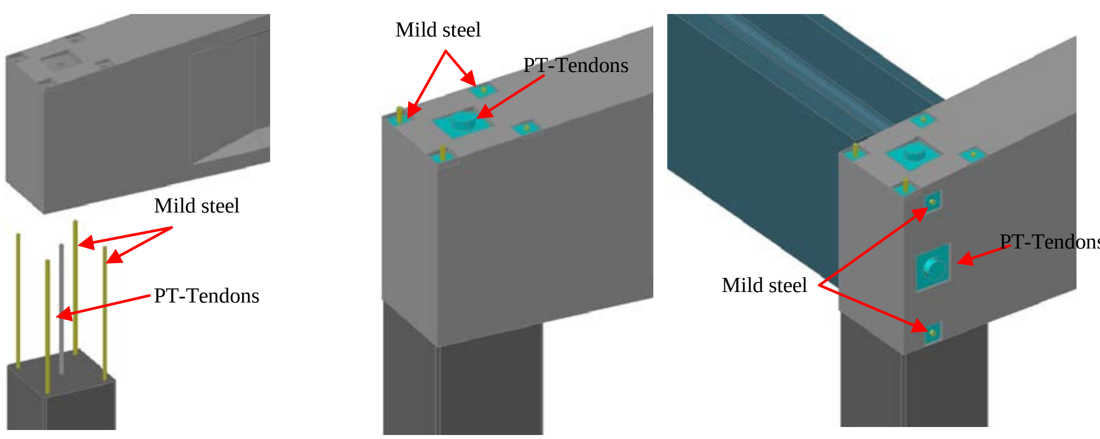
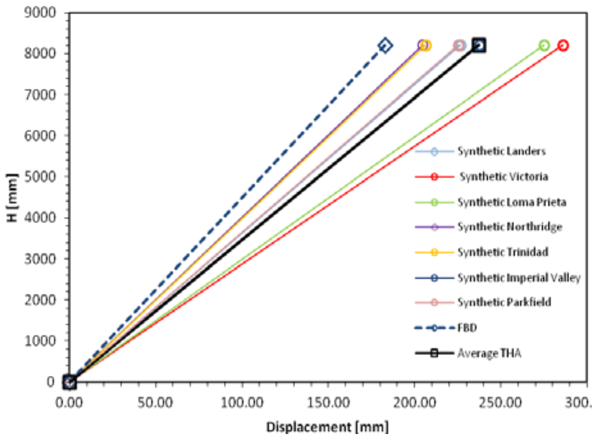


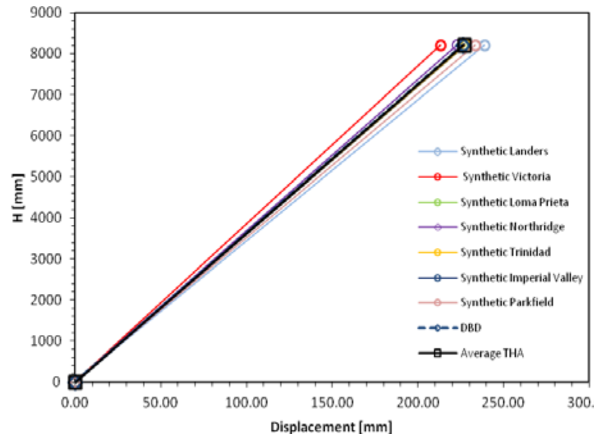
Fig 18. (a) Hybrid system construction sketches.

VERIFICATION OF DESIGN (TIME HISTORY ANALYSIS)

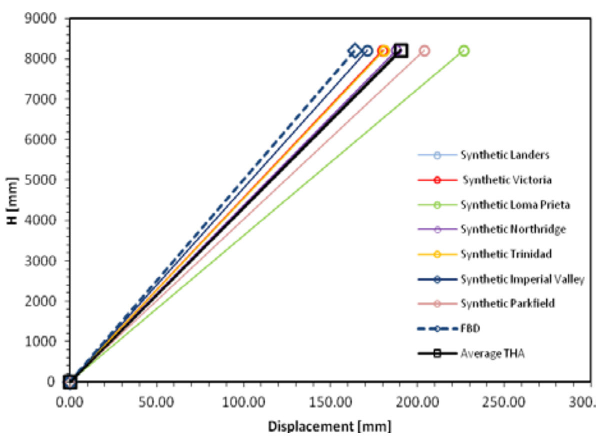
Figure 19 shows the maximum drift ratios recorded for every single time history (color thin lines), their average value and the target drift ratio profile (black and dash lines) for each of the case study structures using a_g=0.35g.



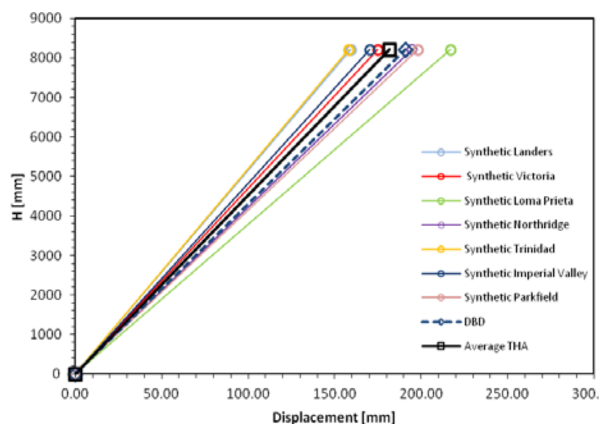
(a) FBD with top pinned connection



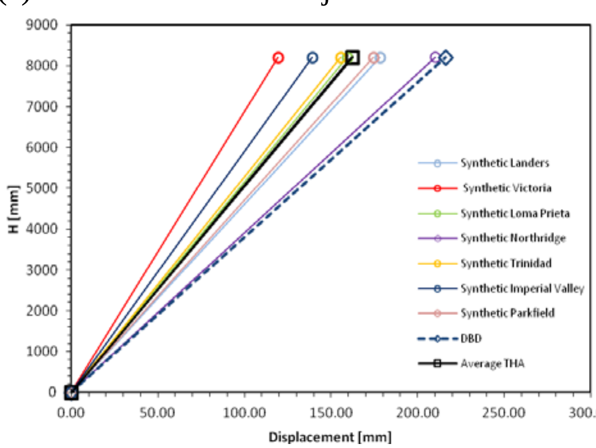
(b) DBD with top pinned connection



(c) FBD with monolithic joints at both ends



(d) DBD with monolithic joints at both ends



(e) DBD with Hybrid Joints

Fig 19. Displacement shapes comparison

Very good agreement can be observed between the target drift and the average value obtained from the analysis for DBD cases. This leads to the conclusion that the DBD procedure performs as expected. Some scatter can be observed between the different time histories; this was expected. Part of the reason may be related to the adopted input ground motions.

FBD profiles behave very different than target displacement, but these results were expected knowing some weaknesses like the unreal assumption of constant member stiffness.

In Figure 20, the displacement time-history response of both solutions (fixed cases) under Loma prieta synthetic record (a_g=0.35g) are presented, major differences should be highlighted in terms of residual drift with a residual displacement for the monolithic connection about 70mm and negligible values of the hybrid solution. This different behavior is further emphasized in Figure 21, where moment – rotation curves for both connections are shown. Residual rotation at base can be observed in the moment-rotation graphs presented.

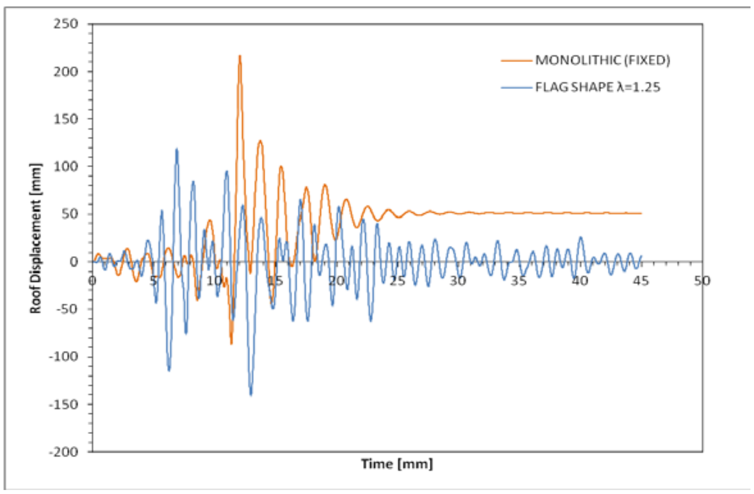
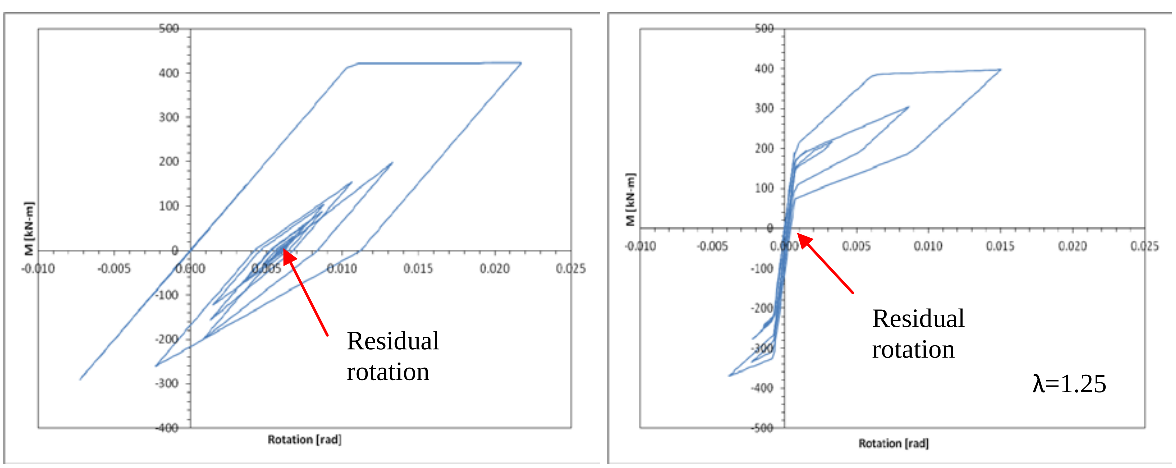


Fig 20. Residual displacement comparison



(a) (b)

Fig 21. Moment-rotation at base for fixed frames (a) Monolithic (b)Hybrid

CONCLUSIONS

The extension and application of hybrid or “controlled rocking” concept to industrial buildings is an efficient and promising alternative solution to the traditional systems. The possibility of accommodating the inelastic demand at the critical section interface where a rocking motion takes place, clearly leads to a significant damage reduction in columns. As a consequence, repairing costs after seismic events would be less.

The intrinsic and well recognized advantages of precast construction, known as quality control, construction speed and costs can be fully exploited using hybrid connections in precast industrial buildings.

Force Based Design approach is unconservative in terms of displacement control. To design precast industrial buildings it is recommended the use of a more accurate methodology like DDBD.

Seismic design must be performed for zones with a_g between 0.35-0.25, however in the other zones where wind design governs, a minimum seismic detailing should be allocated in the resistant elements.

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