

Comparison of different computer programs to predict the seismic performance of the SPEAR building by means of Pushover Analysis

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SUMMARY

Nowadays several computer programs allow the development of simple and complex nonlinear models and analyses. However, the potentialities and limitations of each program and each type of model should be more clarified in order to turn the application of such methods more common and straightforward, especially in Design offices. The main goal of this study is to compare several nonlinear 3D models of the SPEAR building, using different software packages. In this work, different approaches of a lumped plasticity model are developed using SAP2000, being their performance evaluated by means of nonlinear static results (N2 method is used). The results of the lumped plasticity models are compared with a distributed plasticity model developed in SeismoStruct; a comparison between results is presented and discussed. The results obtained allow a better understanding of the characteristics and potentialities of all procedures, helping the users to choose the best approach to perform 3D nonlinear static analysis.

Keywords: Nonlinear Models, Pushover Analysis, Computer Programs

1. INTRODUCTION

The use of Nonlinear Static procedures (NSP) on the seismic assessment of existing structures and also in the design verification of new ones has become a common practice in earthquake engineering. Therefore, it is necessary to understand the potentialities and limitations of the most commonly used computer programs in design offices.

In this work, the selected case study is the well known “SPEAR building”, experimentally tested in full-scale within the European research project SPEAR (Fardis and Negro, 2006). It was chosen essentially due its plan-irregularity. The building was analysed in two different programs: the software SAP2000 v14.2.4 (Computers and Structures, 2010), that is oriented essentially to structural design; and SeismoStruct v5.2.2 (Seismosoft, 2011), a fibre-based structural analysis program essentially focused for Nonlinear analysis. The results obtained with the distributed plasticity model of SeismoStruct were considered in this work to be the reference ones. In fact, several comparisons between experimental results and computational ones (such as Bento et al., 2010) prove the accuracy of this software. A similar study was developed by Araújo and Delgado (2011) for the seismic assessment of bridges.

The NSP used in this work was the N2 method proposed by Fajfar (Fajfar and Fischinger, 1998; Fajfar, 2000) and also recommended and described in Eurocode 8 - EC8 (CEN, 2004). More recently, Fajfar and his team proposed the Extended N2 method (Fajfar et al. 2005), which leads to better results in SPEAR building as shown by Bento et al. (2008). However, the choice of the method is not important since the main goal of this study is to compare several nonlinear 3D models of the analysed building.

2. CASE STUDY - SPEAR BUILDING

The storey heights amounted to 2.75 m for the first floor and 3.00 m for the upper storeys. The building features a non-symmetrical plan configuration but is regular in elevation (Figure 1). Only one

column (C6) presents a rectangular section of 250×750 mm. The rest of them have a square cross-section of 250×250 mm. The column C6 and the presence of a balcony on the east side of the structure are the causes for the in-plan irregularity, shifting the centre of mass away from the centre of stiffness, which is very close to the central column (C3), thus causing the eccentricity to be larger in the y direction.

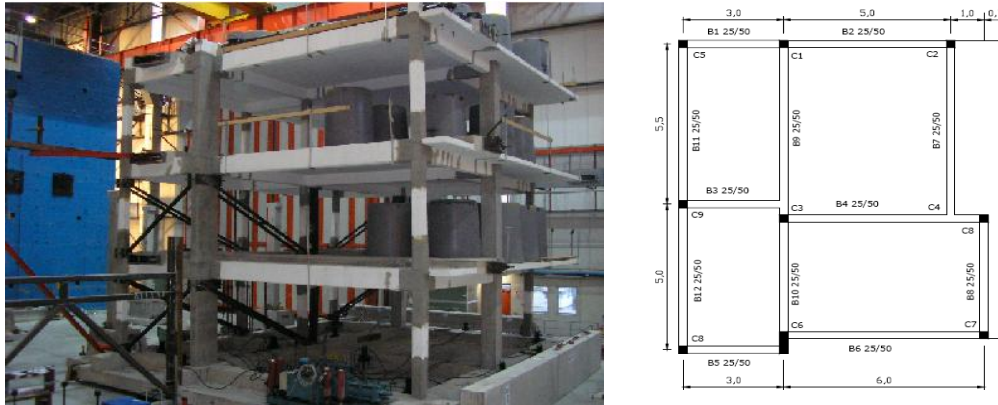


Figure 1 – The full-scale SPEAR Building (Fardis et al, 2006)

The concrete is considered unconfined, due to the almost inexistent transversal reinforcement, and a mean compressive strength of 25 MPa was assumed. Average yield strength of 360 MPa and an ultimate strength of 450 MPa were considered for reinforcement steel.

Total translational masses, obtained from the seismic combination, amounted to 67.3 tonnes each for the first two floors and 62.8 tonnes for the roof.

3. NUMERICAL MODELING

A lumped mass modelling strategy was adopted, in which masses were lumped at the nodal points according to its tributary area.

The floor slabs of the building possessed very high in-plane stiffness compared to the out-of-plane one, therefore these elements can safely be modelled as ‘rigid diaphragm’.

In this work, such diaphragms were modelled by imposing kinematic constraints on the lateral displacements of all nodes at each floor. Therefore, nodal displacements can be expressed by three rigid body motions of the respective floors, namely two horizontal translations and one rotation about the normal to the floor-plane. This reduces significantly the number of dynamic degrees of freedom and hence increases the efficiency for large parametric studies. The effects of flexural stiffness of the slab were considered by assigning appropriate flange widths to the beams. More details about relative accuracy of other slab modelling approaches can be found elsewhere (Pinho et al., 2008).

3.1. SeismoStruct

The SPEAR building was modelled by an assemblage of inter-connected frame elements using distributed material inelasticity through displacement based formulation along with geometric nonlinearity. Each element was discretized into four sub-elements with two integration points each. Fiberized cross-sections – representing sectional details such as cover and core concrete and longitudinal reinforcements – were then defined at respective integration points, whereby every fibre was assigned to an appropriate material constitutive relationship.

The constitutive relationship proposed by Mander et al. (1988) was deployed to model the behaviour of unconfined concrete. In absence of sufficient transverse reinforcement, the confinement effects were not considered for core concrete.

The constitutive model used for the steel was proposed by Menegotto and Pinto (1973) including the modifications due to isotropic hardening proposed by Filippou et al. (1983).

The software, by means of the defined masses, automatically computed the sustained gravity loads. The rigid diaphragm effect was modelled by means of Lagrange Multipliers.

3.2.SAP2000

In the case of confined and unconfined concrete, the stress-strain relationships are defined manually or automatically according the Mander et al. (1988) proposals. In the case of reinforcement steel, the stress-strain relationships can be defined also manually or automatically assuming, in the second case, a Park and Paulay constitutive relationship.

In SAP2000, although the concentrated plasticity is the only choice in nonlinear modelling, different ways of modelling the nonlinearity of the structure were tested to obtain the capacity curve of the structure. The nonlinearity can be considered adopting the following modelling solutions: defining Plastic Hinges with hysteretic relationships automatically based on FEMA-356 (FEMA, 2000) or on CALTRANS (CALTRANS, 2009), or manually following one of the solutions: Uncoupled M2/M3 Hinges or Interaction PMM Hinges.

Since the nonlinear behaviour of the element is characterized independently in both transverse and longitudinal directions, the type of models with uncoupled M2/M3 hinges is more indicated for 2D analysis. Since pushover analysis in 3D buildings is performed independently for each direction, this seems to be a valid approach if the axial force is included in the definition of moment-curvature ($M-\phi$) relationship. In the Interaction NMM Hinges the program requires a previous definition of the MM or NM interaction diagrams of the element cross-section. Another important aspect is related to the number of $M-\phi$ curves that should be defined depending of the type of cross-section.

If the Fibre NMM Hinges is used, the material $M-\phi$ relationship is computed in any bending direction for varying levels of axial force and by assigning particular material stress-strain relationships to individual discretized fibres of the cross section.

4.SEISMIC ACTION

For this study the seismic action was defined assuming an Earthquake Type 1, and a soil type C (EC8) with different intensity levels: 0.05g, 0.1g, 0.2g and 0.3g. Therefore, the values recommended for the soil factor and the spectral periods are: $S=1.15$, $T_B=0.20$, $T_C=0.60$ and $T_D=2.0$. The response spectra for all intensities are displayed in the Figure 2.

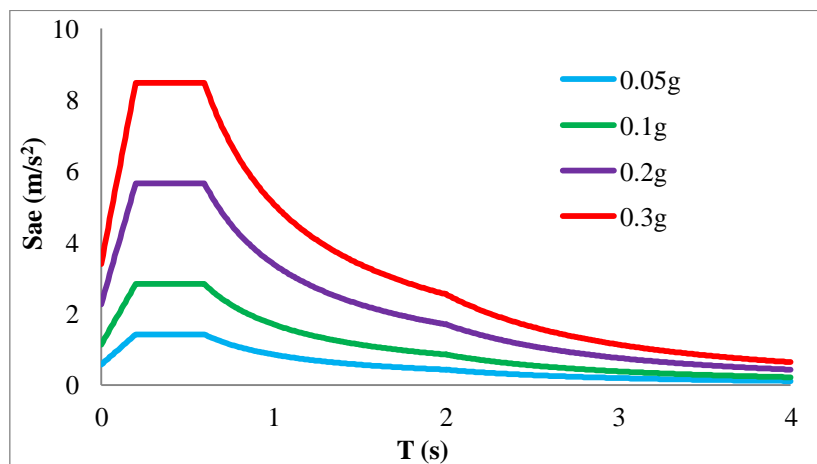


Figure 2 – Design Response Spectrum for all the intensities considered

5.RESULTS

The results obtained in the performed analyses are presented in this section. The monitored nodes are

schematically represented in Figure 3, where SE represents the stiff edge, FE the flexible edge and CM the centre of mass. A central node corresponding to the central column was named as node C.

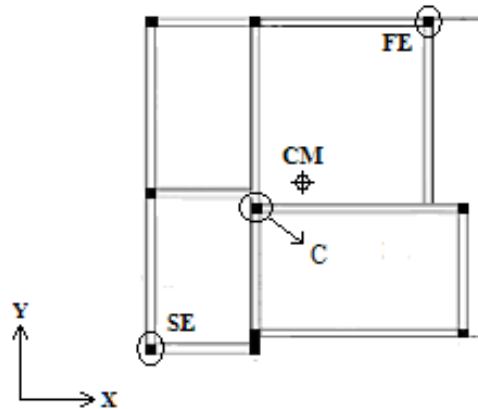


Figure 3 – Monitored Nodes

5.1.Dynamic Properties

The modal properties, in terms of periods (T) and effective modal mass percentages (UX and UY), of the SPEAR building are reported in Table 1 for both programs. According to the results obtained the structure is classified as torsionally flexible in the y direction. As it is shown, the periods of vibration for the first three modes obtained with the two programs are similar. The same conclusion was reached with the values obtained for the higher modes.

Table 1 – Periods and effective modal mass percentages for SeismoStruct and SAP2000 models

	<i>Translation mode along x</i>			<i>Translation mode along y</i>			<i>Torsional mode</i>		
	T (sec)	UX (%)	UY (%)	T (sec)	UX (%)	UY (%)	T (sec)	UX (%)	UY (%)
SeismoStruct									
1st modes	0.62	60.5	7.8	0.44	3.1	31.6	0.53	23.5	43
2nd modes	0.22	7.4	0.8	0.14	0.4	7.9	0.18	2.8	3.9
SAP2000									
1st modes	0.66	60.2	8	0.47	3.6	29.8	0.56	23.3	45
2nd modes	0.23	7.3	0.8	0.15	0.4	7.6	0.19	2.8	4.2

5.2.Capacity Curves

In the current study, pushover analyses were performed in both positive and negative senses in each direction. The x direction capacity curves are quite similar in the positive and negative senses. On the other hand, some differences were found in the y direction, mainly due to the presence of the elongated column C6. Since the main objective of this study is to compare computer programs and modelling approaches, it was assumed to display the results only for the positive sense.

Figure 4 displays the Pushover curves for both directions obtained with Seismostruct, compared with the results defined by means of Nonlinear Dynamic Analyses performed in the SPEAR building by Bento et al. (2009, 2010) and Bhatt (2012).

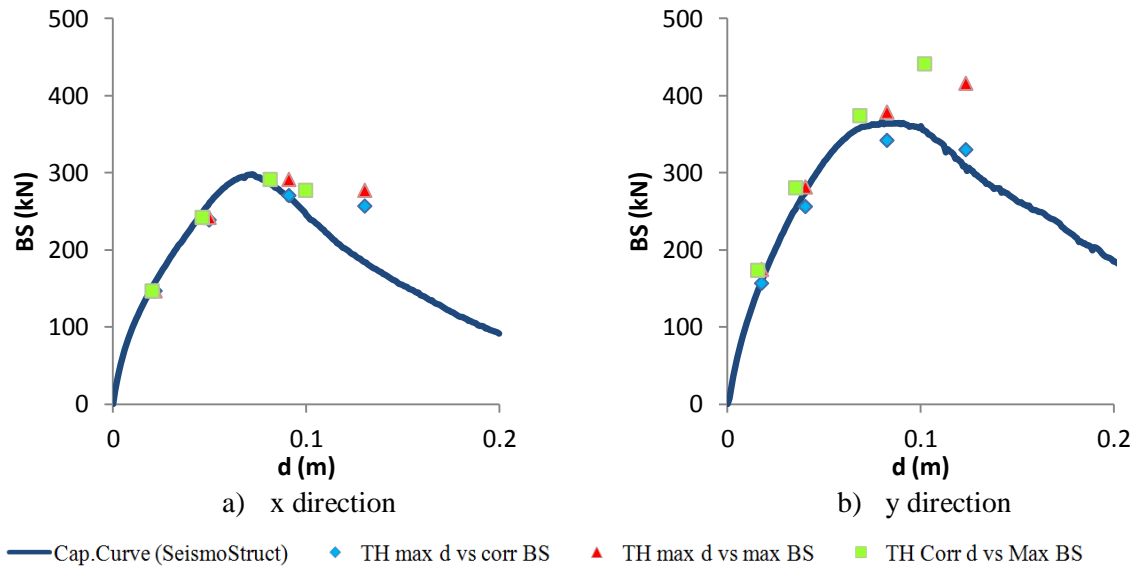


Figure 4 - Pushover curves in x and y direction obtained with SeismoStruct and Time-history results

From the results obtained it is possible to notice that, in general, the pushover curves from SeismoStruct fall close to the Time-history points maximum displacement versus maximum base shear (max Displ vs. max BS), displacement versus maximum base shear (Displ vs. max BS) and maximum displacement versus base shear (max Displ vs. BS). As it was mentioned before, in this study the results obtained from the distributed plasticity model of SeismoStruct were considered to be the reference when evaluating the lumped models of SAP2000.

Some of the modelling approaches explained before did not lead to good results. The most evident case is the option of defining Plastic Hinges with hysteretic relationships automatically based on FEMA-356 in SAP2000. Figure 5 shows the pushover curves for x and y directions obtained, applying Plastic Hinges according to FEMA-356 and from SeismoStruct.

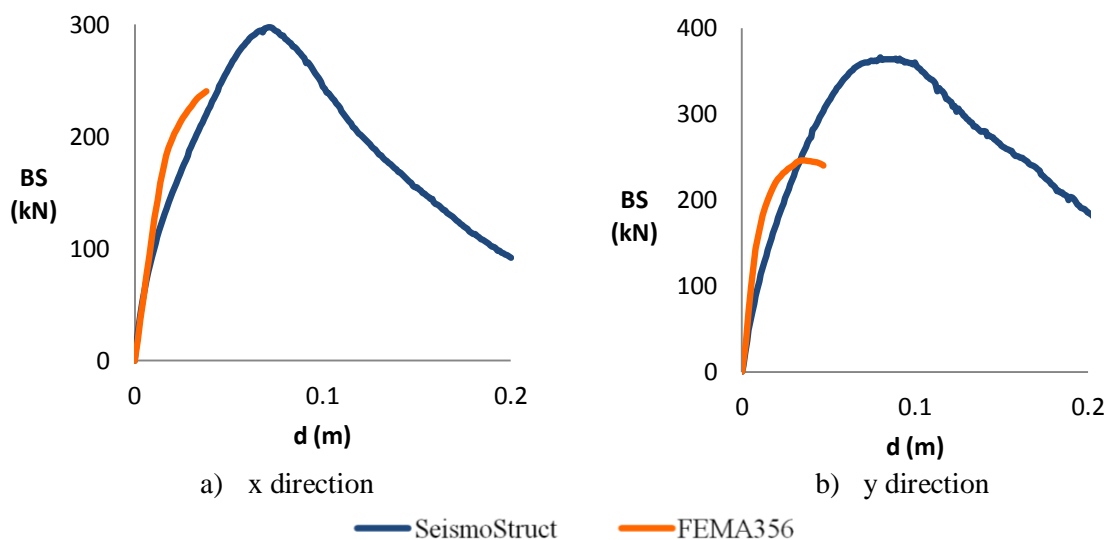


Figure 5 – Pushover Curves obtained with SeismoStruct and with SAP2000 according to FEMA356

From Figure 5, it can be observed that the pushover curves obtained with SAP2000 according to FEMA356 are significantly different comparing with SeismoStruct. In fact, this curve stops in a clearly early stage of the analysis. These results did not change much, even after vary the analyses options of the program.

For all the other approaches considered using SAP2000, more complete pushover curves were obtained for both directions. Herein are displayed the pushover curves for all these approaches in Figure 6.

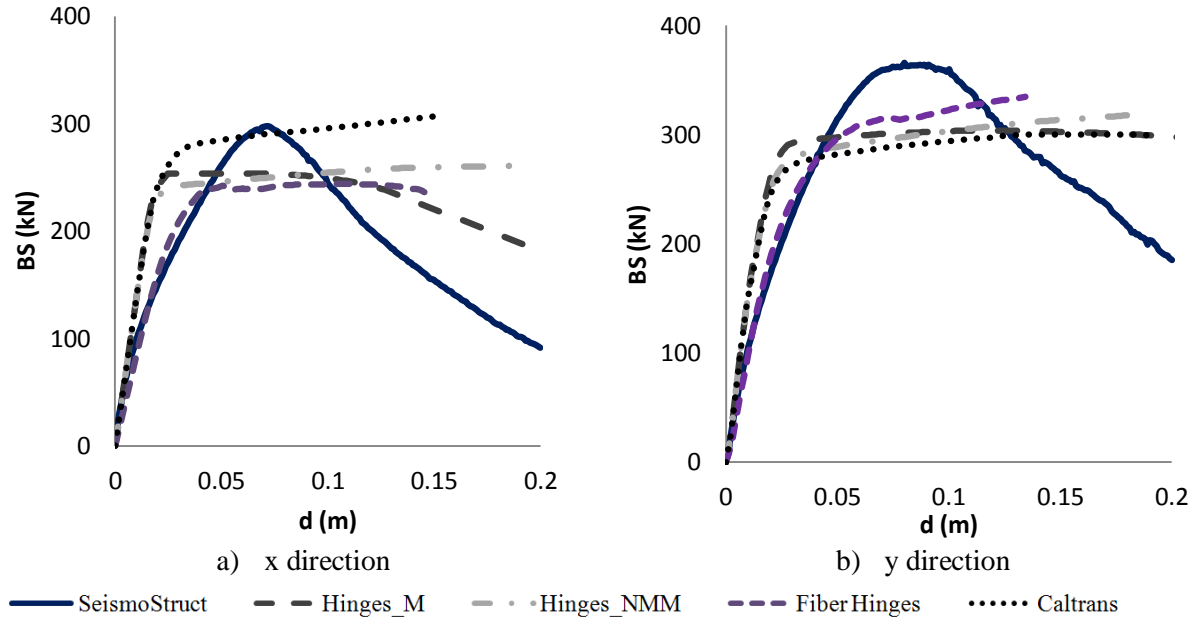


Figure 6 - Pushover Curves obtained with SeismoStruct and with SAP2000 defining Hinges according to Caltrans, Uncoupled M Hinges, Interacted NMM Hinges and Fibre Hinges

Only the pushover curve obtained considering fibre Hinges using SAP2000 shows a similar behaviour until the maximum Base Shear is reached. Although, it underestimates this value when compared with the Pushover curve obtained with SeismoStruct. The other approaches overestimate the pre-peak stiffness, underestimating in general the maximum base shear. None of the SAP2000 modelling approaches was able to reproduce the softening effect of the curve (well exhibited by the SeismoStruct curve). In fact, all these models reproduce the post peak behaviour as a constant line with practically zero stiffness (almost elastic - perfect plastic behaviour). The only exception is the fibre Hinges model in the X direction. These lumped models can also exhibit some convergence problems particularly when a degradation slope is taken into account in the curve.

Despite the results, one should refer that these types of hinge models represent an interesting alternative to more complex fibre hinge models, since they need a minor effort of computation time.

5.3.Target Displacements

Herein the results obtained with SeismoStruct are the same presented in publications of Bento and Bhatt (2009, 2010).

In Table 2 are shown all the target displacements obtained from all modelling approaches and the corresponding Base Shear for all seismic intensities considered in both x and y directions.

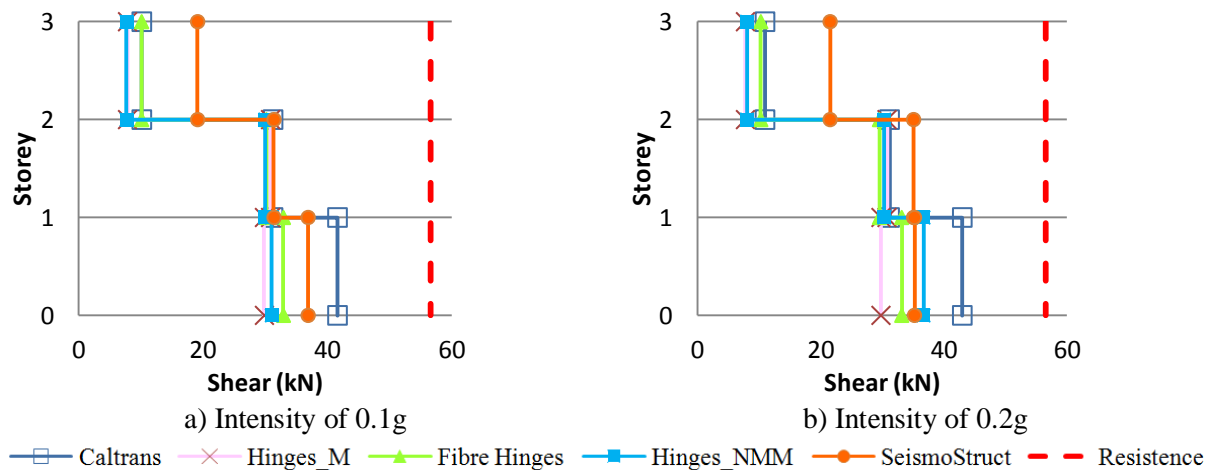
Table 2 – Target displacements obtained with all modelling approaches studied for x and y directions

x direction										
PGA	SAP2000								SeismoStruct	
	Caltrans		Hinges_M		Hinges_NMM		Fiber Hinges			
	d (m)	BS (kN)	d (m)	BS (kN)	d (m)	BS (kN)	d (m)	BS (kN)	d (m)	BS (kN)
0.05g	0.022	248.68	0.017	226.02	0.021	233.45	0.024	180.53	0.02	149.83
0.1g	0.043	282.68	0.034	253.4	0.042	244.49	0.047	240.12	0.04	226.82
0.2g	0.086	293.08	0.069	253.4	0.084	252.04	0.094	243.38	0.09	270.26
0.3g	0.13	302.23	0.103	248.43	0.127	258.6	0.141	239.18	0.13	183.23
y direction										
PGA	SAP2000								SeismoStruct	
	Caltrans		Hinges_M		Hinges_NMM		Fiber Hinges			
	d (m)	BS (kN)	d (m)	BS (kN)	d (m)	BS (kN)	d (m)	BS (kN)	d (m)	BS (kN)
0.05g	0.02	245.43	0.017	238.81	0.022	259.23	0.024	211.24	0.02	173.27
0.1g	0.04	278.58	0.035	295.09	0.043	286.99	0.047	292	0.04	273.14
0.2g	0.08	290.17	0.069	300.77	0.087	300.13	0.095	320.46	0.07	359.32
0.3g	0.12	297.79	0.104	303.55	0.13	310.39			0.11	339.46

For 0.3g all approaches present overestimated Base Shear values in x direction. In y direction the base shear values are not overestimated but in all approaches the base shear is still increasing, opposed to what happens with SeismoStruct. This was expected since in all these approaches, the pushover curves do not reproduce the softening in the post-peak range. However, for lower intensities the results obtained are in general satisfactory. The results obtained with Fibre Hinges using SAP2000 and the ones using SeismoStruct are quite close in terms of target displacements and corresponding Shear Forces (differences of 18% and 16% respectively in terms of mean values) in both directions for lower intensities of peak ground acceleration (PGA). On the other hand the approach considering uncoupled M2/M3 hinges is the only that always leads to underestimated results of target displacements in x and y directions. The reason for these results may lie in the fact of this approach be the only that does not consider the PM interaction during the incremental loading.

5.4. Shear vs. Resistance

In this section, the shear profiles in columns are discussed for all the modelling approaches analysed. In the following figures the results for the column C (C3) are displayed for 0.1g and 0.2g and compared with the shear resistance of the column calculated through the dispositions of EC8.

**Figure 7** – Shear profiles in column C3 for 0.1g and 0.2g in the x direction

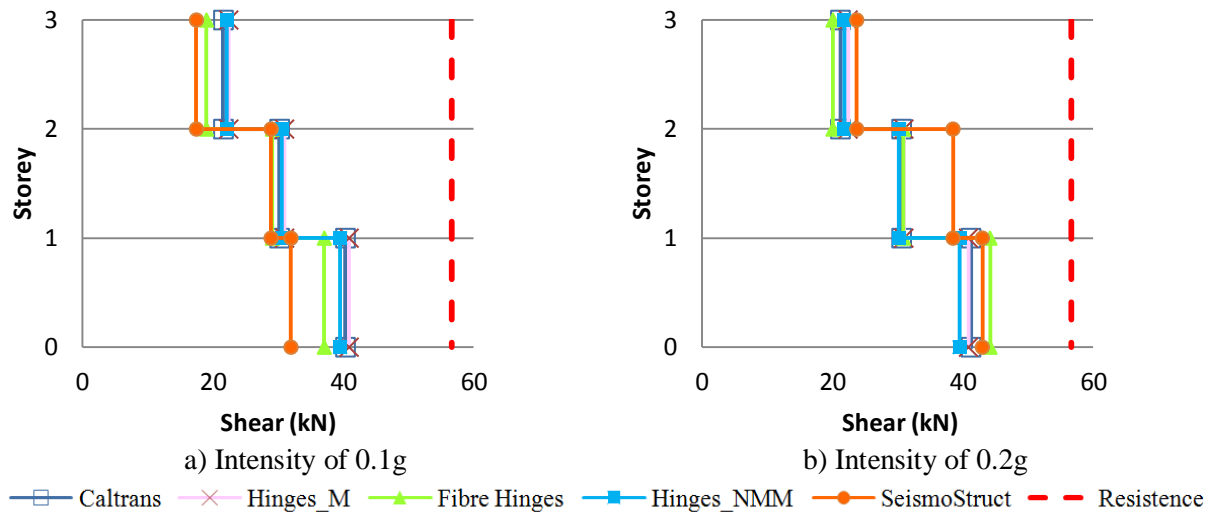


Figure 8 – Shear in the column C3 for intensities of 0.1g and 0.2g in the y direction

From Figures 7 and 8 one can realise that there are no relevant differences between the considered approaches. For this structure there are no problems of brittle failures.

5.5. Interstorey Drifts vs. Damage Limitation EC8

To complete this study, the interstorey drifts were calculated for all approaches and the results displayed in Figures 9 and 10 for the column named FE (C2), for the same intensities as the previous analysis: 0.1g and 0.2g.

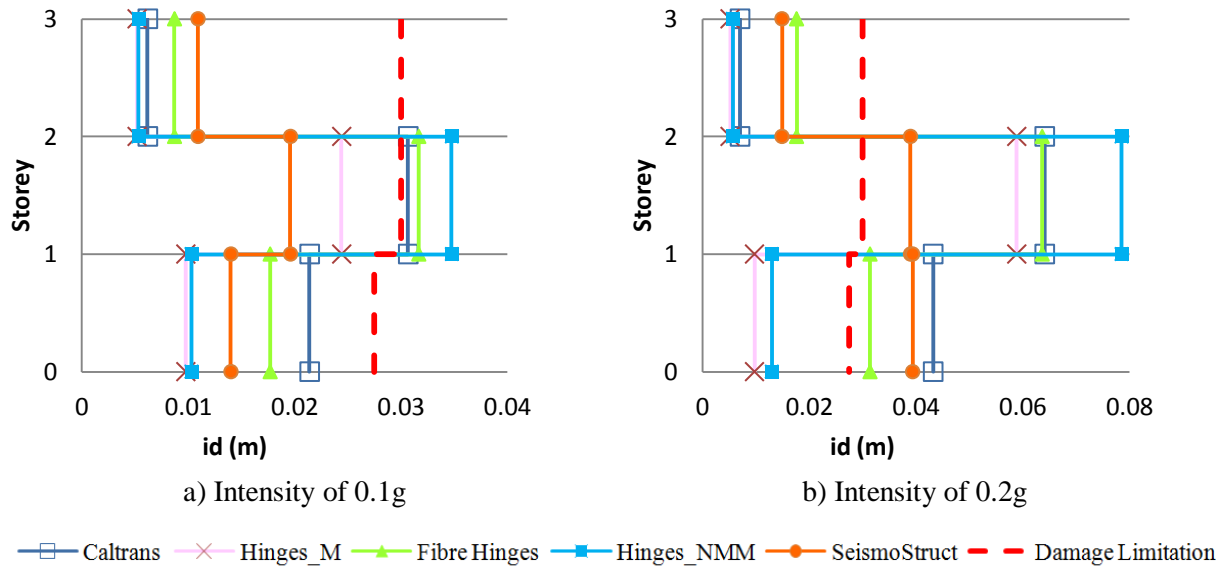


Figure 9 – Interstorey drifts in column C2 for intensities of 0.1g and 0.2g in the x direction

In the second floor, that is the one which presents the most relevant drifts, the approaches with fibre Hinges and interacting Hinges NMM lead to conservative results in both directions, while the other methods lead to conservative results only in x direction. For 0.2g, both approaches are able to reproduce the exceedance of the interstorey drift in the second floor, as well as the approach based in Caltrans, when compared with EC8 damage limitation.

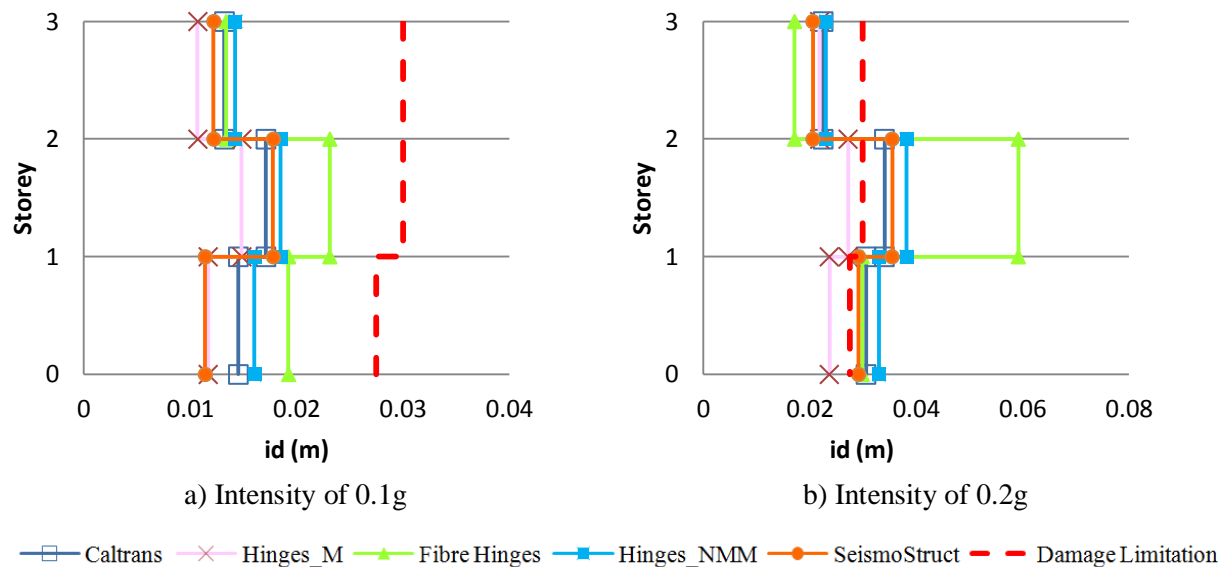


Figure 10 - Interstorey drifts in column C2 for intensities of 0.1g and 0.2g in the y direction

The interstorey drifts in the y direction are also underestimated by the approach considering uncoupled M2/M3 hinges.

6. CONCLUSIONS

In this work, several modelling approaches available in SAP2000 were compared with the “reference” distributed plasticity model of SeismoStruct, using the well known SPEAR building.

From the results presented herein, one could conclude that the definition of plastic hinges with hysteretic relationships automatically based on FEMA356 does not lead to good results for concrete 3D structures in this version of SAP2000.

There are no approaches in SAP2000 that give reasonable results in terms of Base Shear for very high intensities, since they are not able to correctly reproduce the post-peak softening of the structure.

Uncoupled M2/M3 hinges lead to unsatisfactory results for this building, but more tests with 2D structures and plan-regular structures should be performed.

The definition of the plastic hinges through fibre elements is the approach that leads to the best pushover results, if the intensities are not so high. On the other hand, the definition of plastic hinges with hysteretic relationships automatically based on CALTRANS or manually with Interaction NMM Hinges (despite presenting some convergence problems in this version of SAP2000) should represent an interesting alternative for more complex models where fiber hinge models could take a long time to be computed. The conclusions drawn in this paper are valid for the building analysed. Further studies, using a larger set of case studies and more complex typologies, should be developed in order to reach definitive conclusions.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support of the Portuguese Foundation for Science and Technology through the research project PTDC/ECM/100299/2008.

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