Influence of Shear Flexibility in Structural Shear Walls on Pushover Analysis

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1. Introduction

The analysis of seismic performance of reinforced concrete structures, has the possibility to use different design calculation approaches, since it is possible to consider the deformations beyond the elastic domain. Therefore, it is important to know the collapse mechanisms to occur, as well the capacity of dissipate energy within the structure [1, 2], in order to correctly estimate the correct ductility factor. In order to perform an accurate seismic analysis, the most correct would be to use a non-linear dynamic analysis. However, the use of this kind of analysis involves quite complex calculation method, becoming less suitable for its practical application on the seismic design calculation [3]. Despite the fact, that it takes high time-consuming (in terms of computational cost), when using this method, it is also necessary to calculate various accelerograms and keep huge amounts of the structural response data. Therefore, an alternative non-linear static analysis method could be used, which is also designated as pushover analysis. Using this method, it is possible to identify the collapse mechanisms of the structure, on the simpler way than using the non-linear dynamic analysis [1]. The main objective of this work is to study the application of this type of method in general commercial software, particularly the software SAP2000.

2. State of the art

Historically, the design for seismic load was preformed using equivalent static loads during the 80´s. After the 90's, the progress of computational tools, lead to the development of a method, based on Response Spectrum as proposed by [4], and developed by [3]. This method is still largely used in today's codes [2, 5], with special focus in Portugal. This methodology was being criticized by [6] and [7], in terms of adopted spectrum, and internal non-equilibrium modal combination. Also the stiffness reduction is not correctly accountable, being the adopted behaviour factor only dependent of the structure, and not the type of element. Also, this method provides no information, regarding the type of collapse of the structure. In general, it is accepted by the scientific community, that the non-linear analyses methods produce better results than the standard ones describe above [8]. In any case, the non-linear analyses are almost unfeasible, when taking into account the size of the outputs. To overcome these difficulties, many recommend the use of static non-linear analyses (aka pushover analysis) [9], which provide the same reliability as the previous ones, and with a feasible practical application for the industry. The first pushover analysis in 2D were carried and presented with success by [10], and improved later in the works of [11] and [12], due to significant advances with computational tools. These first studies were only valid for simple and rectangular frame structures. The success and reliability of these results, promoted several new codes, with direct application in terms of pushover analysis: ATC40 [13], FEMA273 [14] and FEMA440 [15]. Through the report of FEMA440 [15], it was concluded that high vibration modes could influence the structural response. Therefore, several authors [16,17] suggest some modification in the methodology of the pushover analysis. The main modifications were performed by [18], in which the "Multi-Modal Inelastic Procedures" method, was successfully tested and validated. Recently in 2011 Fajfar proposed in his work [19], an adjustment of his own N2, in order to account high vibration modes. Another suggestion is related to the elastic calculation of the vibration modes, regarding this subject, several authors [20] suggested adaptive methods, in which the stiffness of the structure depends on the load level. The existing methods use different approximations, for the calculation of the target displacement, depending on the energy dissipation mechanism. It is accepted by the scientific community that these methods rely on the type of structure [21–25]. Another subject rely in the correct application of the pushover, during the transformation of multiple to single degree of freedom (DOF), in which Hernandez-Montes [26] reported some criterion to be obey, in order to correctly simulate the non-linearity of the structure in the participation vibration modes. Following this work, several authors tested pushover in 3D symmetric [27], and non-symmetric buildings [28]. In EC8, the N2 method proposed by [19] is recommend. This method is quite simple and presents a formulation based on the ADRS format (Acceleration Displacement Response Spectrum), combined with pushover analysis of multi DOF, with response spectrum of 1 equivalent DOF [3].

3. Adopted finite elements

3.1. Frame element

The Frame elements, use a three-dimensional general formulation, beam-column, which includes the effects of the bi-axial bending and axial deformation [29]. With the Section Design module, it is possible to define the reinforced concrete Frame section, defining the concrete as base material and disposing the steel bars manually [30]. The nonlinear behaviour on this type of elements is taken into account using the definition of plastic hinges. To define this kind of hinges it is necessary to use different constitutive laws, like the moment-rotation or moment-curvature, as well interactions between forces (axial and bending), in order to achieve better understanding of the results [31]. In SAP2000, in the frame element, the non-linear due to shear stress is never taken into account, therefore, the need to use shell elements, in shear resistant walls. It is possible to define plastic hinges by two ways: first, by manual definition of the plastic relation moment-rotation, with null length hinges; and the second, nonlinear connections with multilinear plasticity, wherein is defined a moment-curvature relation as follows:

1. Multilinear uncoupled M2 or M3 hinges: The moment-curvature relation can be defined either automatically, based on the recommendations of Caltrans and Fema356 [30], or manually, defining five principal points for it: by convention the first point represents the load point, the second and third points are the yielding point and the ultimate load point, respectively, and the fourth and fifth points represent the residual stiffness and the collapse point, respectively. Once the non-linear behaviour of the element is independently characterized in both directions 2-2 (transversal) and 3-3 (longitudinal), this type of models should only be used in 2D analysis. On the other hand, once this hinge model allows the analysis of the elements cyclic behaviour, selecting isotropic hysteretic models, Kinematic, Takeda or Pivot, despite of some numerical instability, it can be adopted to carry out the dynamic time analysis in 2D models.

2. Multilinear interaction PM or PMM hinges: The definition of hinges with forces interaction is similar to the previous one, except that the behaviour considers bending in both orthogonal axes, for the case of PMM hinges, and considers the interaction between axial force and bending moment in the case of PM hinges. For the last, the program requires a previous definition of the diagrams which describe the relation PMM and PM for the cross section. Another important aspect, relatively to the PMM hinges, is the fact of being necessary to define the number of curves moment-curvature, according to the type of cross section in analysis. Therefore, in the case of circular symmetry of the column cross section, it is necessary to define only one curve, whereas for asymmetrical configurations it is recommended to use at least three curves (longitudinal, transversal and 45o direction). The main advantage of PMM hinges is that it can be used for the Pushover 3D analysis. However, like the PM hinges, it does not allow the use of all kind of hysteretic models, therefore, it should not be used in the case of nonlinear dynamic analysis.

3. Fibre PMM hinges: This type of hinges is used when it is pretended to define the interaction between the axial force and the deviated bending along the Frame. The hinges can be defined manually or automatically for some cross sections, including the ones defined in the Section Design SAP2000 module. In this type of hinges, for each cross section fibre, it is used stress-longitudinal deformation nonlinear curve of the material to define the relation $\sigma11$ - $\varepsilon11$. By adding up the behaviour of all fibres multiplied by the hinge length, it is obtained the relation between the axial force and deformation, and the moment-rotation in both directions. The relation $\sigma11$ - $\varepsilon11$ is always the same, either for uniaxial, isotropic, orthotropic or anisotropic material. The shear behaviour is not considered for the fibres, this one is computerized for the Frame section using the linear shear modulus G. This type of model is more complete and steady for nonlinear analysis, and can be used in any in 3D, either to pushover or dynamic analysis).

3.2. Shell element

The Shell element is a type of finite element formulation for areas, usually used to model membranes, plates or slabs. It can be homogeneous, composed by one material only, or heterogeneous, composed with more than one material with the possibility to define the nonlinear property. These elements are constituted by three or four nodes, which formulation combines the membrane behaviour and plate-bending behaviour. In order to model reinforced concrete shell element, and after that carry out a nonlinear analysis, it is necessary to define heterogeneous Shell element. In this type of element, the material is defined by layers, where in each layer it is possible to consider the material behaviour as linear or nonlinear, as also any kind of different behaviour for the defined material. The study of heterogeneous section, for the shell element, uses the thick plate formulation, where it is considered the deformation due to the shear force. To define correctly this type of element, it is necessary to consider eight parameters, which are fully detailed in [32].

4. Nonlinear analysis

The pushover analysis is used in order to observe the numerical differences between the frame elements and the shell elements, for different geometries of reinforced concrete structural walls. The software used was SAP2000 in analysis. Before carrying out this type of analysis, it is important to perform correct definition of the material nonlinear behaviour. The concrete is considered to be class C25/30, which stress-strain relationship in compression is as assumed in EC2 [33]. The steel is considered as class A400NR, which stress-strain relationship is a classical perfect elastoplastic [34]. In all examples, EC8 response spectrum was used, for soil type C, with type 1 earthquake, in Lisbon, for buildings class II.

4.1. Rectangular wall

The main objective of this example is the analysis of results which were obtained using shell elements, and its validation in the modelling of the structure, comparing it with the results obtained using frame elements. The wall is constituted by reinforced concrete with 4 metres height and a cross-section of 0.2x1 metres (Fig. 1), with 30 cm² of steel rebar. The wall is not restrained at the top and is fixed at the

base. Once is referred, it was not taken into account the deformability by shear in the nonlinear analysis of the wall.

In order to perform the pushover analysis, the structure was modelled using frame and shell elements. Initially, the uniform load with light masses (1 ton at each level of the wall, with the level being measured per unit meter) was applied to direct assessment of Frame and Shell models. This was done on purpose to evaluate the nonlinear effects due to the axial force. The uniform load was applied according to the direction of the highest resistance and its structural response is represented in Fig. 2. As it was expected, the yielding load for the Frame model and for the Shell model has similar responses, with the respective constitutive relationships for curvature and rotation. But there is a small difference in the first elastic regime, when comparing frame and shell element in Fig. 2, the first ones present a more rigid behaviour. This phenomenon occurs because of two main reasons: the first one is that the frame element, contrarily to the shell element, does not have behaviour in Stage II (section stiffness after the concrete cracking), it just admits reduction of the flexibility after the plastic hinge is formed; the second reason is that the frame element is not able to simulate accurately the deformability by shear with steel bars considering the nonlinear analysis. Therefore, this type of element just presents deformation by bending, and it is stiffer than the shell element. The application of N2 method has the necessity of positioning concentrated masses of 50 tonnes at each level of the wall, with the level being measured per unit meter. The results obtained from pushover analysis and from N2 method are presented below in Figure 3 and 4, for uniform and modal loading. It allows identifying the numerical differences which exist when different finite element types are used in the structural modelling.
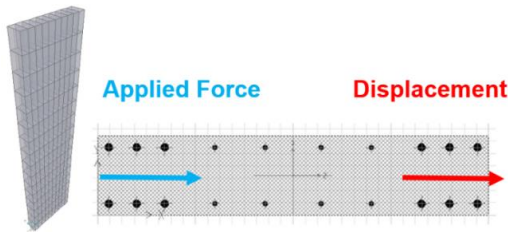

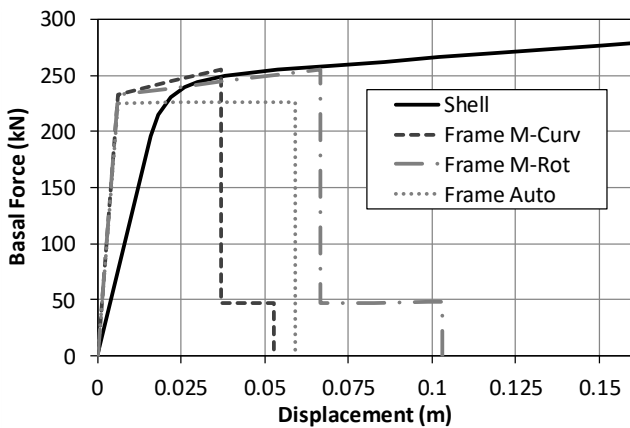Fig. 1 Geometry of the rectangular wall

Fig. 2 Initial test using uniform load with light masses

Although it is possible to verify the stiffness variation of the section in the shell elements when it passes from uncracked to cracked phase, the stiffness is always minor compared to the frame elements, which never consider the appearance of cracks, wherein the elastic phase is always

referred to uncracked section. Even with these small differences, the results are similar, being possible validate the shell elements structural response. The big difference lies on the ductility factor, which is substantially lower/smaller when shell elements are used to model the structural walls. N2 method permits to verify that, although the shell elements are the ones, which present the higher target displacement, they also present the minor ductility factor.
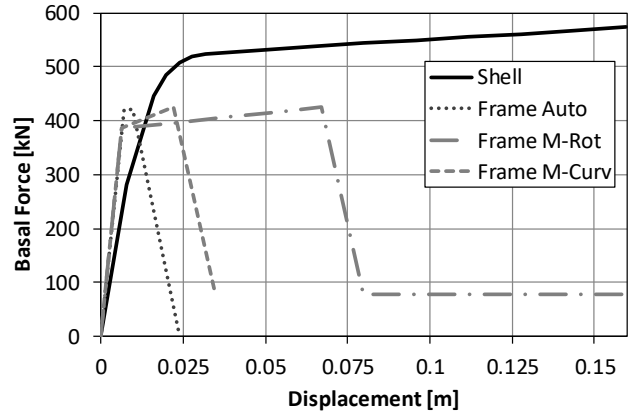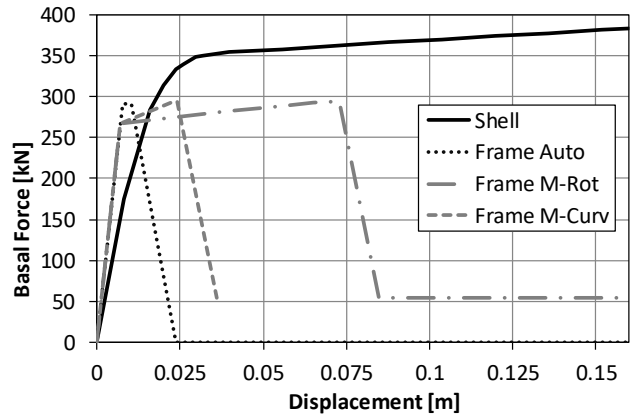

Fig. 3 Uniform loading

Fig. 4 Modal loading

4.1.1. Discussion of the results

Analysing force-displacement curves obtained from the pushover analysis it is possible to define one of the principal limitations of the shell elements: for the frame element formulations exists a failure point, whereas for the shell element it never happens, with a curve always crescent. In this case, it was admitted for the shell element the failure point, when the concrete reach 0.35 % of deformation and the steel 2.0 %.

Results obtained from N2 method are presented in Table 1.

Table 1

Method N2 results for the rectangular wall

Type of modelling	Target displacement, cm	Ductility factor μ
Shell	5.61	2.29
Auto	2.63	4.69
M-Rot.	3.75	3.24
M-Curv	3.23	3.53

It is important to note that in contrast to the frame elements, the structure stiffness is minor for the shell elements.

4.2. L wall

The main objective of the example with L type wall is to understand the difference which exists in the case of the structure where the principal directions of inertia are not coincident with the Cartesian axes. L type wall is constituted by reinforced concrete with 4 metres height, cross-section of 1x1 metres and the thickness of 0.20 metres (Fig. 5), with a total of 50 cm^2 of steel rebar. The wall is not restrained at the top and is fixed at the base.
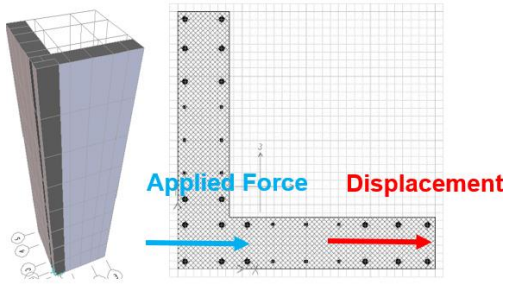

Fig. 5 Geometry of the L wall

The structure is modelled with frame and shell elements in order to perform the pushover analysis. Concerning the application of N2 method, it was necessary to position concentrated masses of 50 tonnes at each level of the wall, with the level being measured per unit meter.

Next, it is displayed the results acquired from the pushover analysis, as well the results computed from the N2 method, in order to detect the numerical differences which, exist when using different finite elements in the structural modelling. The structural response of the basal force is displayed in Figs. 6 and 7 for uniform and modal loading. The level of displacements, are the same as the rectangular wall, due to the analysis being performed in displacement control.

Results from N2 method in Table 2.

Table 2

N2 method results for L type wall

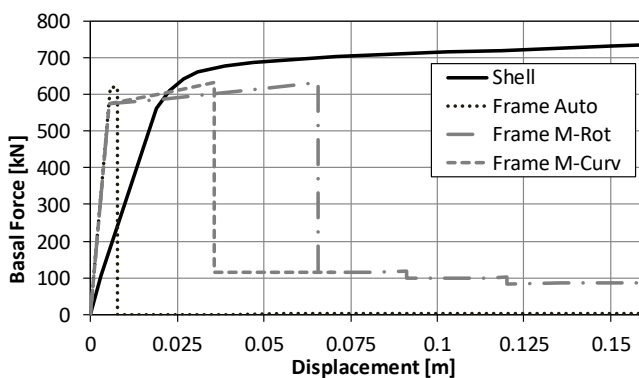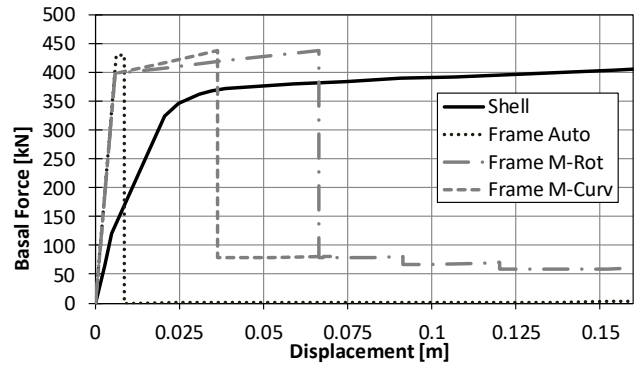| Type of model-ling	Target displacement, cm	Ductility factor μ
Shell	5.24	1.72
Auto	1.71	2.69
M-Rot.	2.32	1.99
M-Curv	2.04	2.09

Fig. 6 Uniform loading

Fig. 7 Modal loading

4.2.1. Discussion of the results

In this example, analysing the results obtained from the pushover method it is possible to observe that shell elements have less load capacity than frame elements. Such difference is more visible in the case of modal loading. This is because principal axes of inertia of shell elements are not coincident with load application axes, which provoke torsion effects, reducing the load capacity of the structure. Using shell elements for modal loading, it is possible to observe that the structure vibration modes present bigger participation in torsional mode than the frame elements. The results obtained from the application of N2 method are quite similar to the results presented in the previous example. The shell elements have less ductility factor and higher target displacement. Although there is no consensus in the scientific community, about which is the best load to apply, for this example in particular, clearly the high vibration modes have strong influence in the value of the basal force.

5. Application to real building

After the case studies previously presented in this work, and after the validation of the results, as well the identification of certain phenomenon which are taken into account differently, according to the type of finite elements used in the modelling, a real building was studied, in order to compare the results obtained when shear resistant walls are modelled with shell elements or frame elements. For the frame element, only the Frame M-Rot is used. The building is constituted by 5 floors, with 3 meters' height each, and with the dimensions of 10x18 meters, measured from the top. The slabs are flat and support 11 structural walls. There are 8 rectangular walls, 2 with L shape and 1 with U shape. All the walls are fixed at the base (Figure 8). The interior shear walls have 0.5x0.15, and the rebar is distributing along its length with 8Ø10. The rectangular that are located in the corners have 1.0x0.15 and a total rebar of 16Ø10. The L shear walls have 1x1x0.15, and a total of 28Ø10. The U shear wall in the centre has 1x1x0.15 with a total of 42Ø10. Each floor has 3.0 metres high, and the concrete slab is 0.15 meters thick. The beams with 0.65x0.2, and are all in linear regime, in order to force the collapse mechanism in the shear walls. In this example, the slabs are always modelled with shell elements; only for the walls the distinction is made between the type of used elements, because the main objective is to evaluate the behaviour of reinforced concrete walls. The structure is then modelled using shell and frame elements, in order to perform the pushover analysis. In this

case, pushover analysis is performed in both directions (x and y). The values of the basal force are displayed in Fig. 9 to Fig. 12, in both directions, and for both type of loads.
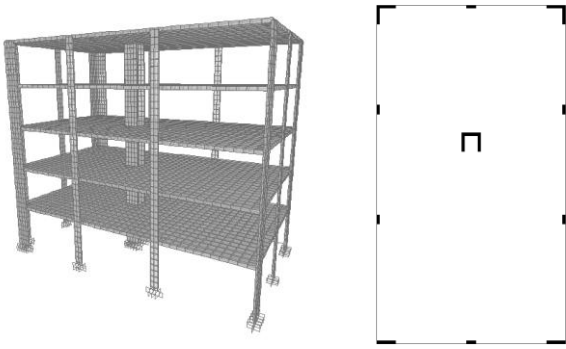

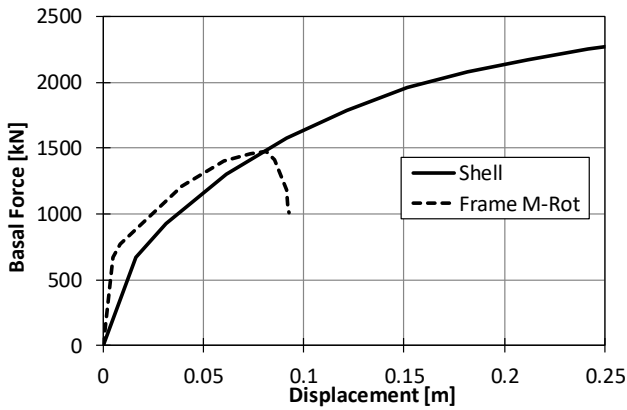Fig. 8 Geometry and adopted mesh for the building
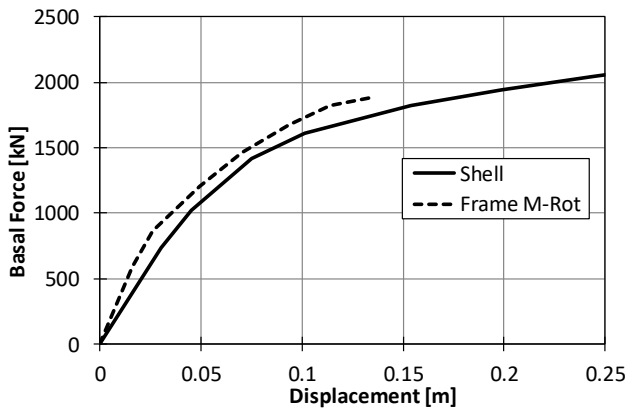

Fig. 9 Uniform loading in direction x

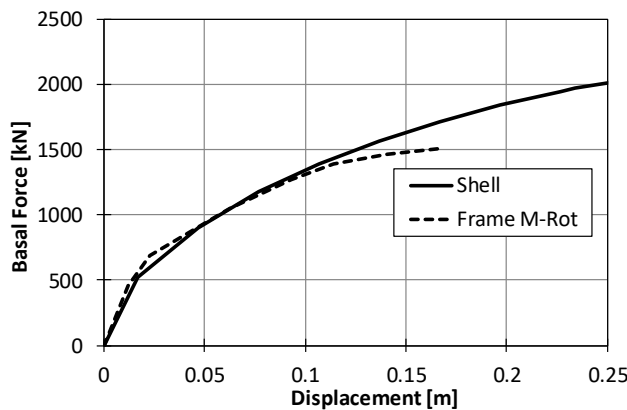Fig. 10 Uniform loading in direction y

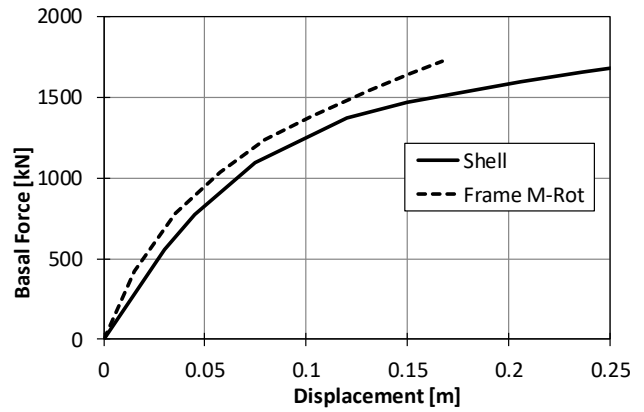Fig. 11 Modal loading in direction x

Fig. 12 Modal loading in direction y

Results from N2 method in Table 3 and Table 4 for the respective x and y direction.

Table 3

N2 method results for the building in direction x

Type of model-ling	Target displacement, cm	Ductility factor μ
Shell	10.11	1.23
M-Rot.	7.49	2.00

Table 4

N2 method results for the building in direction y

Type of model-ling	Target displacement, cm	Ductility factor μ
Shell	10.65	1.3
M-Rot.	10.08	1.15

5.1. Discussion of the results

Within this example, the results withdrawn from the analysis with shell and frame elements (M-Rot) are presented. The results of M-Curv are not presented here, once they are identical to the Frame M-Rot. The model with frame Auto is not presented because there was no convergence in the calculation.

Once again it is possible to observe that the effects due to the torsion are taken into account only for the shell elements and not to the frame elements. For the uniform loading it is possible to observe that the curves obtained from the analysis with frame elements are above the curves obtained with shell elements, and the same happens for the modal loading in y direction, where the effects due to torsion are more present because of the asymmetry of the structure.

After analysing the results obtained from the pushover analysis and from the application of N2 method, it is important to refer that problems may exist in its collapse mechanism, because it is assumed linear elastic behaviour for the beams.

For this example, the structure load capacity may be overestimated, because it is considered only the yielding of the walls and that the beams always respond in linear regime. It is possible that in real collapse mechanism could occur the yielding of both elements (walls and beams), by other words, the collapse mechanism studied in this example might not correspond to the real collapse mechanism of the building.

6. Conclusions

The study developed within this work, permitted to observe the main differences between the modelling using shell elements or frame elements, obtaining from it three major conclusions.

Observing all the results, it is possible to verify that the load capacity for the frame elements is always overestimated when there is a torsional component due to the eccentricity between the applied load and the shear centre. This is not verified in the results for the shell elements. In the example of the rectangular wall, it is important to refer that results obtained with shell elements are valid and represent the correct behaviour of the structure.

In the practical example, applied to the building, it can be observed another important conclusion within this work. It is the fact that the modelling with frame elements or shell elements have influence on vibration modes. In the case of the structure modelled with shell elements, all the vibration modes present high torsional component, which is expectable in this type of structural geometry. On the other hand, the frame elements overestimate the load capacity of the structure when presenting an asymmetrical geometry, as it was mentioned before. This addition to the load capacity, is more relevant when the modal load is applied instead of the uniform load, because the vibration modes of the frame present the smaller torsional component. Also in case of load capacity, the shell element presents an average extra difference of 20%, if high torsional component is present.

One of more important conclusions taken from this study is that the frame elements tend to increase the ductility coefficients. Therefore, it is recommended new investigations involving pushover analysis concerning the use of shell elements in structural walls.

To conclude, it is important to refer that using shell elements it is possible to observe the transition from Phase I to Phase II (uncracked cross-section and cracked cross-section, respectively), when the concrete tension capacity is defined. The same is not observed for the frame elements, once the concrete tension capacity does not change the obtained result.

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INFLUENCE OF SHEAR FLEXIBILITY IN STRUCTURAL SHEAR WALLS FOR PUSHOVER ANALYSIS

S u m m a r y

The aim of this work is to show the main differences which exist, taking in to account the influence of the type of finite element used, when performing pushover analysis of reinforced concrete structures. The non-linear analysis was performed using FE software SAP2000, and the results were extracted from models including Frame and Shell elements, respectively.

Several reinforced concrete structures were modelled with Frame elements and Shell elements, which will be further presented. Therefore, it was possible to validate the results obtained from the analysis, also to identify certain restrictions according to the type of finite element used in the modelling of the resistant walls.

In the first phase, three isolated structural walls were modelled with distinct geometries. The first one presents a rectangular shape, the second – "L" shape and the third one "U" shape. The application of pushover analysis through the different examples presented in this document, intends to validate the results obtained for the Shell elements.

Subsequently, the same kind of analysis was performed on a building. These examples intend to show that the performance of ductility is strongly dependent from the type of element, which is not taken into account in the pushover analysis nowadays.

N2 method was applied to all examples, in order to understand the differences in the structures seismic design, according to the type of element used in the modelling. The results are compared, and the differences are identified. As well as, the limitations of applicability of Shell elements in the modelling of structural walls were determined.