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# SEISMIC BEHAVIOR OF REINFORCED CONCRETE FRAMES WITH SETBACKS

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## SUMMARY

The purpose of the presented work is the behavior assessment of reinforced concrete frame structures with irregularities in elevation. Three six-storey frame structures with different degrees of irregularity in elevation were selected and their responses were compared with the ones of a corresponding regular structure. In this study, the effects of the variation of the axial force in the columns and of different contributions of slab width to the beams' flexural strength were investigated. In addition, a structure similar in configuration with one of the previously selected was defined and designed using current Portuguese codes to perform a comparison between different design approaches. Structural performance was assessed by comparing local ductility levels, displacements, inter-storey drifts and damage indices of the irregular frames and the regular one.

## INTRODUCTION

It is widely acknowledged that the structural behavior of buildings during high intensity earthquakes depends on mass, stiffness and strength distributions both in plan and in elevation. By analyzing the current state of development of seismic design methods, the general agreement seems to be that the degree of confidence they provide is sufficient when applied to regular structures or in cases in which the mass, stiffness and strength distributions obey certain regularity criteria. However, when dealing with irregular structures, substantial doubts still exist.

For some years, several studies have tried to evaluate the influence of the referred structural parameters on the dynamic behavior [1-6]. Based on these studies, it has been found that even considerable variations in the mass distribution in elevation cause minor effects on the ductility demand or on the maximum displacements [7]. However, such outcome cannot be expected when speaking of variations in the stiffness or strength distribution in elevation. Although current design codes exhibit an important state of development, it is important to stress that, when looking at the damage levels endured by structures under severe earthquakes, structures with irregular profiles in elevation can be seen to constantly exhibit inadequate behavior though they were designed by appropriate codes.

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Due to the multitude of factors involved in the structural conception process, most of the common building structures end up to be structurally irregular. Since it is known that structural irregularity may lead to increased deformations and damage concentration under earthquake loading, the present study analyses the behavior of a class of irregular structures and compares it with the behavior of a regular one.

## PROPOSED STUDY

In general terms, the proposed study aims to determine the influence of factors such as the variation of the axial force in the columns and the different contributions of the floor slabs to the beams' flexural strength on the structural behavior of irregular frame structures.

### Selected structures

As recent literature references to experimental results regarding the behavior of irregular structures is very scarce, a one-quarter scale model of a two-bay by two-bay six-storey reinforced concrete (RC) frame having a 50% setback at mid-height tested at the shaking table of the University of California at Berkeley was selected [8]. From this structure, Fig. 1, the frames of the *X* direction were chosen.

Based on the structural detailing of the building, [8], it was seen that for the selected direction the interior frame is different from the exterior ones. Therefore, the numerical model defined to simulate the real structure in the *X* direction, which was called "Experimental", is the one represented in Fig. 2, in which the frames are grouped in series and linked by strut beams at each storey in order to guarantee the same displacement for each storey of all the frames. The left frame in Fig. 2 represents both exterior frames of the real building while the right frame represents the interior one.

Frames exhibit a constant bay length of 7.62 m and an also constant inter-storey height of 3.66 m. Slab thickness was 0.18 m while member cross sections were 0.51*0.71 $m^2$ for all the beams and 0.66*0.51 $m^2$ for all the columns. Longitudinal reinforcement ratio varies between 0.41 % and 0.66 % for the beams. Total steel areas are equal to 1.5% and 2.3% of gross section area of the exterior and interior columns, respectively.

By analyzing the design of this structure, it can be see that column cross sections and reinforcement areas are considerably large and constant throughout the building height. Such characteristics can be related to the search for a weak beam/strong column behavior, which is common nowadays in light of concepts such as capacity design. Regarding the cross sections dimensions of both columns and beams, they appear to be somehow excessive in light of the current design practice. Due to the considerable bay lengths of the structure, beam cross sections are quite large. This aspect works against the weak beam/strong column behavior but does not restrain adequate structural behavior. Based on [8], it can be seen that even for structures that would be graded as "irregular" regarding the regularity in elevation and according to criteria present in modern codes such as the Eurocode 8 [9], adequate behavior under earthquake loading is achievable.

With the purpose of analyzing the behavior of structures that could equally be graded as "irregular" by the referred criteria, two additional structures were defined based on the "Experimental" structure. These were denominated "Irreg1" and "Irreg2". In addition, a regular structure, called "Regular", was also defined to work as a comparison structure. Figs. 3 to 5 present the corresponding typical frame of the "Irreg1", "Irreg2" and "Regular" structures, respectively. The structural characteristics of the "Experimental" structure form the base for the definition of these new structures, meaning that both exterior and interior frames of the new structures have the same characteristics as the "Experimental" ones and that their global numerical model is similar to the one presented in Fig. 2.

In addition to these structures, another one similar to the "Experimental" structure was defined and designed according to current Portuguese codes [10, 11]. This structure, called "PT-Design", was designed for normal ductility class requirements [10] which is the current most common design practice does not account for capacity design principles, and considering the two different earthquake types that must be considered in Portugal:

> Seismic action type I – earthquakes with moderate magnitude, small focal distance, with a peak ground acceleration (PGA) of 0.28g and a code defined duration of 10sec.
>
> Seismic action type II – earthquakes with high magnitude, high focal distance, with a PGA of 0.16g and a code defined duration of 30sec.

The concrete design strength was set to be 13.3 MPa and the steel yield design strength was considered to be 348 MPa. Beam cross sections were 0.30*0.70 $m^2$ for all the beams while column cross sections were the ones presented in Table 1. Longitudinal reinforcement ratio varied between 0.09% and 0.3% for the beams of the exterior frames and between 0.13% and 0.79% for the beams of the interior frame. Total steel areas varied between 1.16% and 2.16% for the outer columns of the exterior frames and between 1.14% and 1.65% for the interior columns of those frames. For the interior frame, they varied between 1.41% and 2.46% for the outer columns and between 0.72% and 1.82% for the interior ones.

The study that was carried out consists in the nonlinear dynamic analysis of the four frame structures subjected to increasing intensity earthquakes. The selected ground motion consisted in the North-South waveform of the 1940 El Centro earthquake, Fig. 6. Four intensity ground motions were then defined by scaling the original wave motion. The PGA of these ground motions are 0.077g, 0.166g, 0.319g (original PGA of the earthquake) and 0.493g, where g represents the acceleration of the gravity. The intensities 0.077g, 0.166g and 0.493g are the same that were used in the experimental program and will allow for the correlation between numerical and experimental results of the "Experimental" structure.

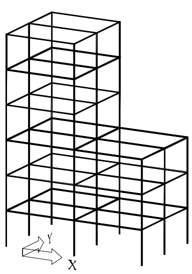

**Figure 1 – Structural configuration of the "Experimental" structure.**

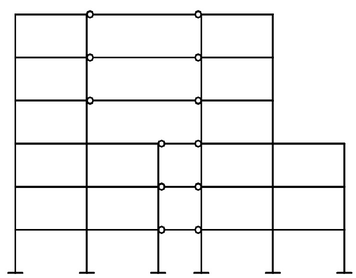

**Figure 2 – Structural configuration of the analyzed direction of the "Experimental" structure.**

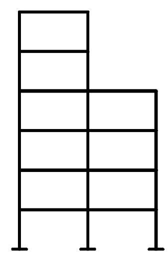

**Figure 3 – "Irreg1" structure.**

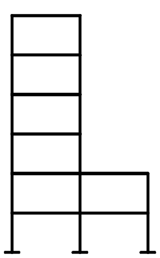

**Figure 4 – "Irreg2" structure.**

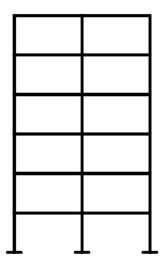

**Figure 5 – "Regular" structure.**

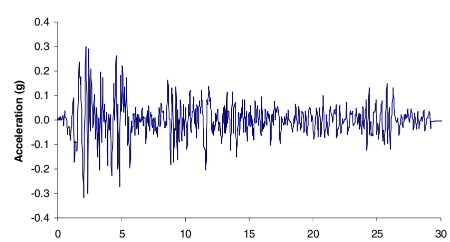


**Figure 6 – Accelerogram of the 1940 El Centro earthquake.**

**Table 1 – Column cross sections of the "PT-Design" structure.**

| Storey | Exterior Frame | | | Interior Frame | | |
|---|---|---|---|---|---|---|
| | Left Columns | Central Columns | Rigth Columns | Left Columns | Central Columns | Rigth Columns |
| 6 | 0.30*0.30 $m^2$ | 0.30*0.30 $m^2$ | - | 0.30*0.30 $m^2$ | 0.30*0.30 $m^2$ | - |
| 5 | 0.30*0.30 $m^2$ | 0.30*0.30 $m^2$ | - | 0.30*0.30 $m^2$ | 0.30*0.30 $m^2$ | - |
| 4 | 0.30*0.30 $m^2$ | 0.35*0.30 $m^2$ | - | 0.35*0.30 $m^2$ | 0.40*0.40 $m^2$ | - |
| 3 | 0.30*0.30 $m^2$ | 0.35*0.30 $m^2$ | 0.30*0.30 $m^2$ | 0.35*0.30 $m^2$ | 0.40*0.40 $m^2$ | 0.30*0.30 $m^2$ |
| 2 | 0.30*0.30 $m^2$ | 0.40*0.30 $m^2$ | 0.30*0.30 $m^2$ | 0.40*0.30 $m^2$ | 0.50*0.50 $m^2$ | 0.30*0.30 $m^2$ |
| 1 | 0.30*0.30 $m^2$ | 0.40*0.30 $m^2$ | 0.30*0.30 $m^2$ | 0.40*0.30 $m^2$ | 0.50*0.50 $m^2$ | 0.35*0.30 $m^2$ |

**Analytical modeling of the structures**

As previously referred, the dynamic behavior of the several structures was assessed through the results of a number of nonlinear dynamic analyses. With the exception of the strut beams linking the frames that define the global model and are considered elastic, Fig. 2, the remaining structural members were modeled using member-type non linear macro-models with three zones: one internal zone having linear behavior and the plastic hinges, located at the members' extremities, which have inelastic behavior.

The skeleton moment-curvature curves that define the behavior of the nonlinear zones were calculated using experimentally measured values of the material properties, [8], by matching the monotonic behavior of a RC section to a trilinear envelope. This trilinear envelope is obtained through the theory proposed in [12] which enables the calculation of cracking, yielding and ultimate resistance points of the trilinear behavior envelope curve based on equilibrium conditions, considering axial loads in columns and asymmetric bending in beams. Hysteretic behavior of the structural members was defined by the Costa-Costa hysteresis model rules [13]. In the inelastic zones, stiffness and strength degradation with increasing deformations were considered effects. Accounting for the modeling indications provided in [8], the considered plastic hinge length corresponds to the depth of the structural members and the viscous damping was set as 2.3%.

Regarding the modeling of beams, three different situations were considered when defining their monotonic trilinear skeleton envelopes. In the first case, the nonlinear behavior of these elements was considered without any slab width contributing to their flexural strength. In the second and third cases, slab width was considered to contribute to the beams' flexural strength. In the second case, called case "T1", slab width was considered to be 1.7 m for the outside beams and 2.9 m for the inside ones. In the third case, called case "T2", slab width was considered to be 2.29 m for the outside beams and 4.57 m for the inside ones. These latter values correspond to the maximum possible slab contribution.

With respect to the modeling of columns, two different situations were also considered. On the first situation, the monotonic trilinear skeleton envelopes of these elements were kept constant during the analysis. In the second case, the monotonic trilinear skeleton envelopes of the columns were updated throughout the analysis in order to account for the effects of the axial force variation. Experimental tests in this area, [13], show that the axial force value and the way in which it varies associated to the also varying column displacements have a significant effect on the bending response of columns. In order to account for the axial force variations, the numerical procedure used to calculate the trilinear monotonic envelopes [12] was integrated in the computer program used to perform the dynamic analyses, [14]. In order not to update the monotonic envelopes for small and insignificant axial force variations, a limit was set to bound the value from which a specific column should have its monotonic envelope updated. After this update, the new axial force value becomes the value the bounding limit is applied to. A small study was performed to determine the value of the referred limit for the monotonic envelope update. The monotonic envelopes of the columns of the "Experimental" structure were determined for different values of the axial load. The axial force value corresponding to the quasi-permanent loading was selected for the starting point and 5%, 10% and 15% positive and negative variations around that value were then considered. Fig. 7 presents the results of this study for one the columns. It can be seen that the effects of a +/-15% variation in the axial force level are small and correspond to a variation in the yield moment of the column under 3%. Based on this study, the axial force variation limit to update the monotonic envelope of a specific column was set to be 20%.

Regarding the analyses results, maximum horizontal displacements, inter-storey drifts, local curvature ductilities and local damage indices using a modified Park and Ang damage index formulation [15] were used to assess the dynamic responses.

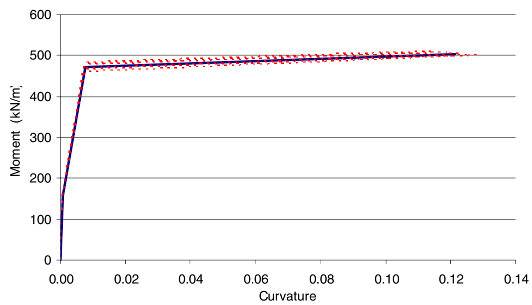


**Figure 7 – Effect of the axial force variation on the monotonic envelope of a column.**

## RESULTS OF THE DYNAMIC ANALYSES

Considering the substantial amount of available results from the analyses that were carried out, only a summarized description of those is presented in the following. Firstly, some comments are presented regarding the correlation between the available experimental results and the numerical simulations. The characteristics of the overall behavior of the structures are then addressed. Afterwards, the results regarding the influence of the variations of the axial force in the columns and of different contributions of slab widths to the beams' bending behavior are presented. Finally, results concerning the behavior of the "PT-Design" structure are addressed and compared with the ones of the "Experimental" structure.

### Correlation between experimental results and numerical simulations

Since the real test structure was subjected to several experimental earthquake simulations and free vibration tests, correlation of its response by numerical analyses was found to be difficult. In addition, the selected El Centro accelerogram, Fig. 6, does not correspond to the exact same ground motion used in the experimental program, a fact that only increased the difficulties of matching the two dynamic responses. Since previous to the first earthquake test, intensity 0.077g, the real structure had already been subjected to free vibration tests, it can be assumed that the structure's stiffness conditions at the beginning of the 0.077g test didn't correspond to uncracked stiffness conditions. This can be the reason why an acceptable correlation between experimental and analytical maximum horizontal displacements for the 0.077g intensity test could only be obtained by using the initial stiffness of the structural members equal to their cracked stiffness, Fig. 8. With respect to the subsequent test, the 0.166g intensity test, the matching difficulties between experimental and numerical results were higher. To consider the initial stiffness of the structural members equal to their cracked stiffness was not enough to reach the high deformation levels exhibited by the experimental results, Fig. 9.

Regarding the structure's frequencies, a good agreement was, nonetheless, found between experimental and analytical values. The experimental first and second longitudinal frequencies were 1.85 Hz and 5.0 Hz while the numerical ones were 1.82 Hz and 4.46 Hz.

Since the accelerograms used in the analyses were not the ones used in the experimental program, the reasons for the encountered matching differences could not be fully identified. However, due to the interaction between the shaking table platform and the test structure, additional rotational accelerations were measured during the experimental tests [8]. This secondary motion which was not included in the numerical simulations could be an additional reason for the poor matching between experimental and analytical results.

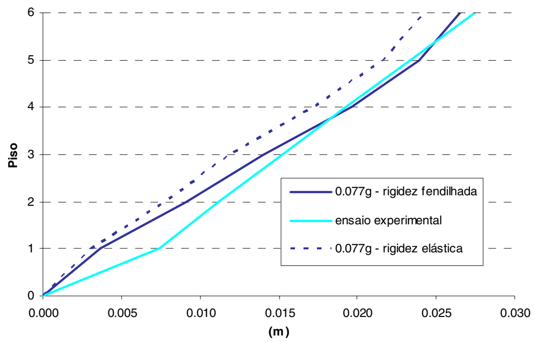


**Figure 8 – Comparison between experimental and numerical maximum horizontal displacements – 0.077g.**

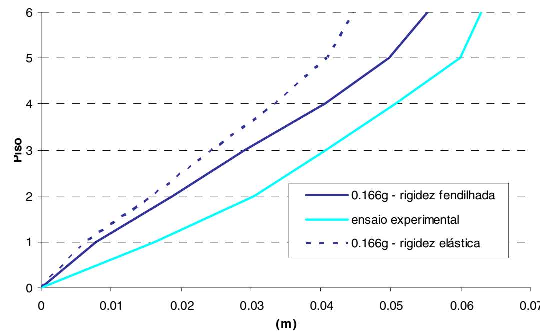


**Figure 9 – Comparison between experimental and numerical maximum horizontal displacements – 0.166g.**

**Overall description of the global structural behavior**

Since the proposed study is mostly academic in nature, the results presented in the following were obtained considering that all the structures have initial uncracked stiffness conditions for all the ground motions intensities.

Although structures “Irreg1”, “Irreg2” and “Experimental” can be graded as “irregular” [9], by comparing their behaviors with the one from structure “Regular” those were seen not to differ much. By looking at Figs. 10 and 11 corresponding to the maximum horizontal displacements and inter-storey drifts for the 0.493g intensity, respectively, though structures “Experimental” and “Irreg2“ exhibit higher displacements at the top storey, their vertical distribution is in the overall adequate. Although larger differences between the several structures can be observed in Fig. 11, it must be stressed out that the maximum drift value is less than 1%. It is also interesting to see that structures “Experimental” and “Irreg2“ exhibit a considerable inter-storey drift increase in the setback zones. By opposition, the behavior of structure “Irreg1“ is closer to the one of structure “Regular“ which concentrates larger drifts at stories 1 and 2.

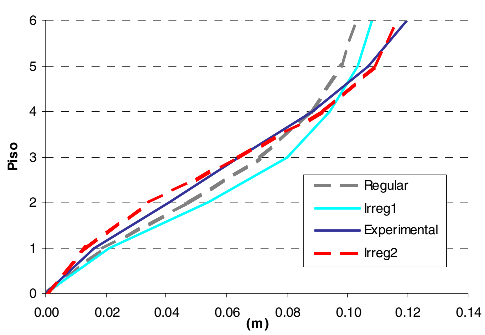


**Figure 10 – Maximum horizontal displacements for the 0.493g intensity.**

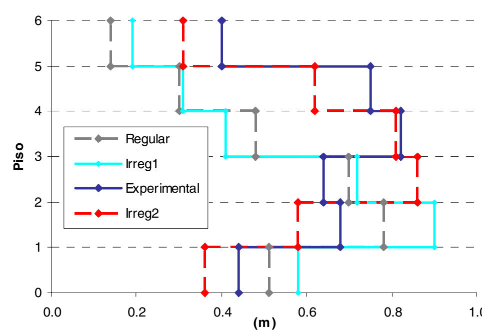


**Figure 11 – Maximum inter-storey drifts for the 0.493g intensity.**

In global assessment terms, the several structures exhibit low displacement values and also small inelastic incursions, even for the highest ground motion intensity. Since this highest intensity can be assume to correspond to a large and very intense earthquake, the low values of the several dynamic responses indicate that the design of the "Experimental" structure, from which all the others were obtained, was essentially controlled by earthquake design measures. This statement can be supported by observing Tables 2 and 3 where values of the maximum top displacements D, maximum inter-storey drifts, maximum damage indices and local curvature ductilities are presented for beams and columns and for the intensities 0.319g and 0.493g. The storey numbers where these maximum values occur are also displayed in those tables.

With respect to the effect of considering different slab widths contributing to the flexural strength of the beams, it was observed that the largest slab width values, corresponding to the "T2" case, have a lower influence in the overall dynamic response than the "T1"case, when comparing results of the case where beams are considered with rectangular cross sections. This effect was observed both at the member and global levels, as can be seen by Figs. 12 and 13 that present the maximum displacements and inter-storey drifts for the 0.493g intensity and for the different beam flexural modeling strategies.

**Table 2 – Maximum response values for the 0.319g intensity.**

| | Experimental | | Irreg1 | | Irreg2 | | Regular | |
|---|---|---|---|---|---|---|---|---|
| | Value | Storey | Value | Storey | Value | Storey | Value | Storey |
| D (cm) | 8.0 | - | 7.1 | - | 8 | - | 6.7 | - |
| Drift (%) | 0.4 - 0.5 | 2 and 3 | 0.4 - 0.6 | 1 to 3 | 0.5 - 0.6 | 3 and 4 | 0.45 - 0.5 | 1 and 2 |
| Column Damage | 0.06 | 1 | 0.08 | 1 | 0.05 | 1 and 2 | 0.08 | 1 |
| Beam Damage | 0.06 | 4 | 0.07 | 1 and 2 | 0.08 | 3 | 0.06 | 1 and 2 |
| Column Ductility | 0.6 | 1 | 0.9 | 1 and 2 | 1 | 2 | 0.8 | 1 and 2 |
| Beam Ductility | 1.2 | 4 | 1.2 | 1 and 2 | 1.5 | 3 | 1.0 - 1.1 | 1 and 2 |

**Table 3 – Maximum response values for the 0.493g intensity.**

| | Experimental | | Irreg1 | | Irreg2 | | Regular | |
|---|---|---|---|---|---|---|---|---|
| | Value | Storey | Value | Storey | Value | Storey | Value | Storey |
| D (cm) | 12.0 | - | 10.8 | - | 11.7 | - | 10.3 | - |
| Drift (%) | 0.6 - 0.8 | 2 to 5 | 0.9 | 2 | 0.8 - 0.9 | 3 | 0.7 - 0.8 | 1 and 2 |
| Column Damage | 0.12 | 1 | 0.12 | 1 | 0.09 | 2 | 0.13 | 1 |
| Beam Damage | 0.17 | 4 | 0.16 - 0.17 | 1 and 2 | 0.2 | 3 | 0.14 - 0.16 | 1 and 2 |
| Column Ductility | 0.9 | 1 | 1.5 - 1.6 | 1 and 2 | 1.6 | 2 | 1.3 | 1 and 2 |
| Beam Ductility | 2.5 | 4 | 2.3 - 2.3 | 1 and 2 | 2.5 | 3 | 1.9 - 2.0 | 1 and 2 |

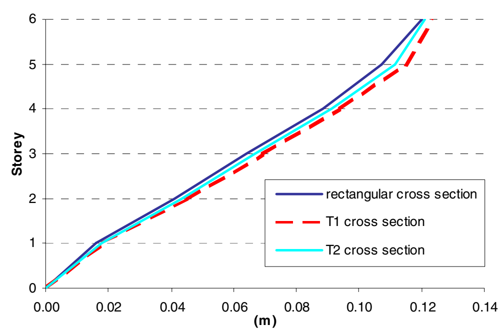


**Figure 12 – Maximum horizontal displacements for the different beam modeling strategies.**

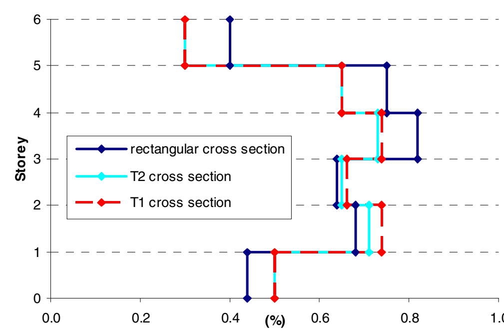


**Figure 13 – Maximum inter-storey drifts for the different beam modeling strategies.**

**Influence of different slab widths contributing to the flexural strength of the beams**

No clear definition was found regarding the influence of this parameter when analyzing the many available results. Its influence was found to change from structure to structure and also between different intensities of the ground motions. Some of the effects of considering the "T1" cross section are presented in the following and compared with the case where beams were modeled by rectangular cross sections. As stated previously, considering the "T2" cross section causes small changes on the structural behavior when comparing it with the "T1" modeling case.

*"Experimental" structure*

Regarding this structure, Figs. 14 and 15 present the variations in the demand when the "T1" cross section is considered. In this case, the structural response was found to exhibit a tendency to increase damage in the columns with considerable variations for the 0.493g intensity which sets the maximum damage index to a value of 0.12 (still a low value). A similar trend was observed for the curvature ductility demand in those elements, though exhibiting lower variations, setting the maximum ductility demand in 2.0 for the 0.493g intensity. This tendency to increase column demand tends, however, to be less important in the upper stories with the third storey acting as a key turning point for the positive/negative demand variations. Regarding beams, the observed tendency was similar to the one of the columns. However, from the third storey up a tendency for a decrease in the demand is observed. With respect to the inter-storey drift demand, the influence of the "T1" cross section changes with the increase of the ground motion intensity. For less intense motions, drift demand is lower for the "T1" cross section case. For the more intense ground motions the lower stories tend to exhibit an increase up to 20% in drift demand while the upper stories maintain the reducing trend up to -25%. A similar trend was observed for the maximum horizontal storey displacements. However, for the highest ground motion intensity all the stories exhibit a displacement increase which is, nonetheless, lower than 15%.

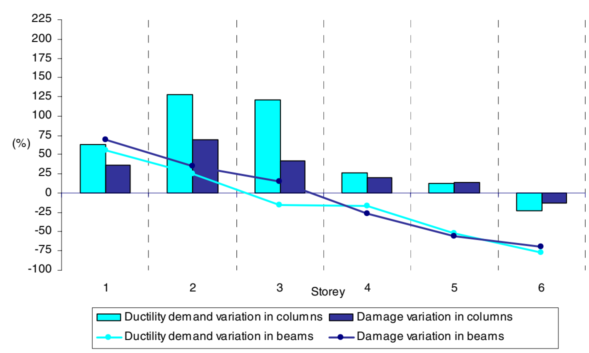


**Figure 14 – Variations in column and beam demand for the "Experimental" structure and for the 0.319g intensity when considering the "T1" cross section.**

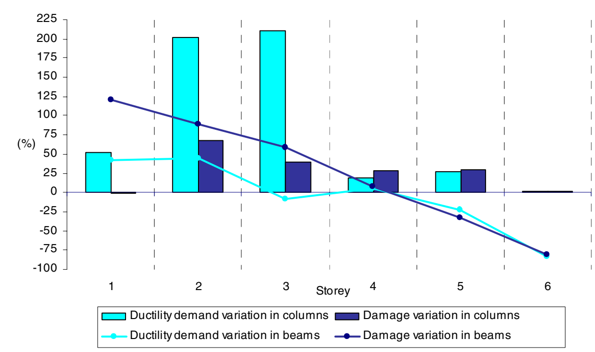


**Figure 15 – Variations in column and beam demand for the "Experimental" structure and for the 0.493g intensity when considering the "T1" cross section.**

*"Irreg1" structure*

With respect to the damage and curvature ductility demand variations in the columns, a trend similar to the one of the previous structure was observed. However, the variations in the demand seem to be more uniformly distributed in elevation, Figs. 16 and 17. For the 0.493g intensity, the maximum damage index was 0.16 and the maximum curvature ductility was 1.9. Also in beams the response tendencies with the "T1" cross section were similar to the ones of the previous structure, though exhibiting lower variation values. Regarding the inter-storey drifts a tendency comparable to the one of the previous structure was detected but with lower variations. For the case of the maximum displacements, the changes are less clear than for the previous structure but in the overall, lower variations were detected than for that structure.

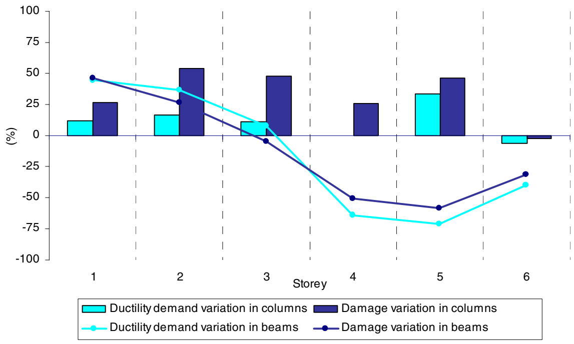


**Figure 16 – Variations in column and beam demand for the "Irreg1" structure and for the 0.319g intensity when considering the "T1" cross section.**

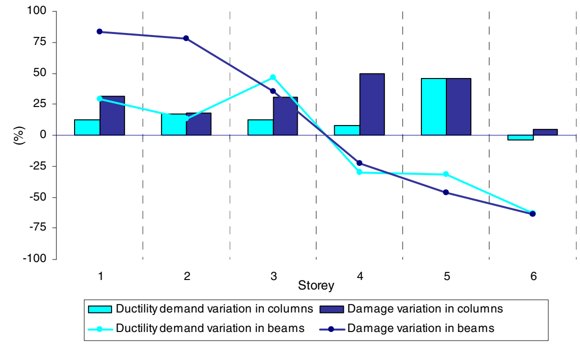


**Figure 17 – Variations in column and beam demand for the "Irreg1" and for the 0.493g intensity structure when considering the "T1" cross section.**

*"Irreg2" structure*

In opposition to what was observed for the previous structures, the variation trends for column and beam damage and ductility demands were in this case less clear. Columns tend to exhibit an increase in damage and ductility demand in the upper stories with the uprising of the ground motion intensity, Figs. 18 and 19. The key turning point for the positive/negative demand variations seems to have shifted to the second storey. In beams the key point effect is less clear, still a tendency similar to the one of the columns can is identifiable but with lower variations. In the overall, the variations are lower than for the previous structures. Regarding the inter-storey drifts, only for the 0.493g intensity an increase of this parameter was obtained, though lower than 15%. For the remaining intensities, drift values are smaller than those obtained by considering beams with rectangular cross sections. Also in the case of the maximum displacements, a clear influence of the consideration of the "T1" cross section was unable to be defined. Small variations around +/-10% were, nonetheless, detected.

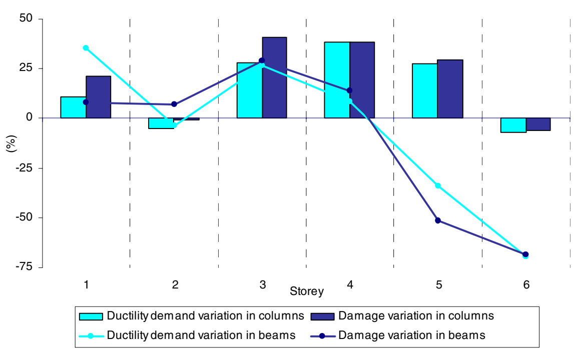


**Figure 18 – Variations in column and beam demand for the "Irreg2" structure and for the 0.319g intensity when considering the "T1" cross section.**

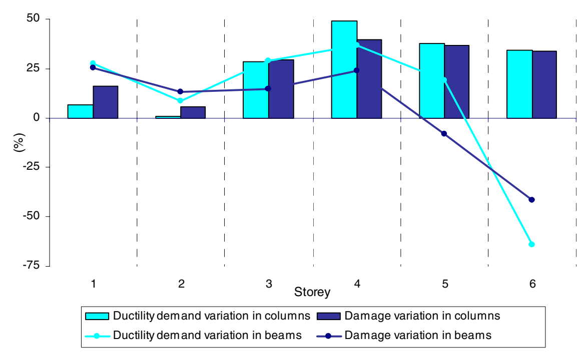


**Figure 19 – Variations in column and beam demand for the "Irreg2" and for the 0.493g intensity structure when considering the "T1" cross section.**

*"Regular" structure*

For this structure, columns display a tendency to increase the damage level and the ductility demand when the "T1" cross section is considered, Figs. 20 and 21. For the 0.493g intensity, damage variations are comparable to the ones of the "Irreg1" structure and set the damage index to a value of 0.1. Ductility demand increases up to 40% but exhibits a maximum value of 2.1. With the exception of the top storey response, beams exhibit demand variations similar to the ones of the "Experimental" structure. Regarding drift demand, variations are under 20%. As the ground motion intensity increases, drift demand tends to increase up to the third floor and to decrease from that storey up. With respect to the maximum displacements a similar trend was detected but exhibiting variations not higher than 15%.

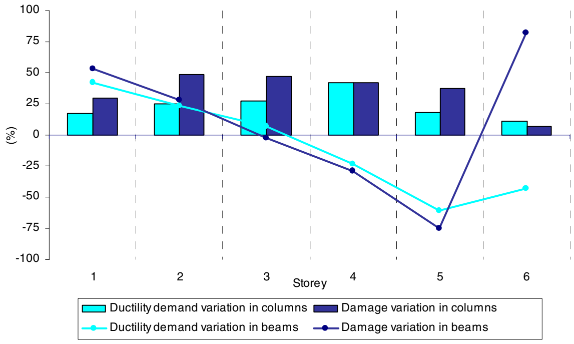


**Figure 20 – Variations in column and beam demand for the "Regular" structure and for the 0.319g intensity when considering the "T1" cross section.**

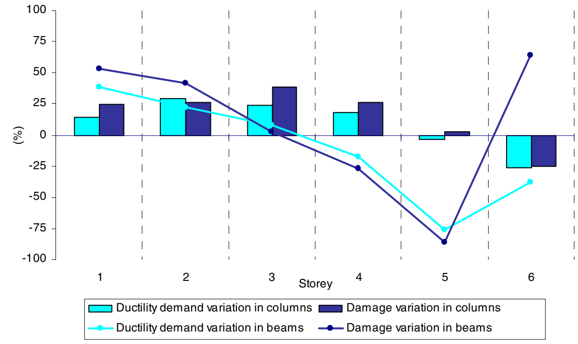


**Figure 21 – Variations in column and beam demand for the "Regular" and for the 0.493g intensity structure when considering the "T1" cross section.**

**Influence of the axial force variations in the columns**

The consideration of the axial force variation in the previously described way did not seem to significantly influence the structural behavior, both locally and globally. Figs. 22 and 23 display the maximum horizontal displacements and inter-storey drifts with and without the consideration of the axial force variations, for the "Experimental" structure with beams modeled has rectangular cross section and for the 0.493g intensity.

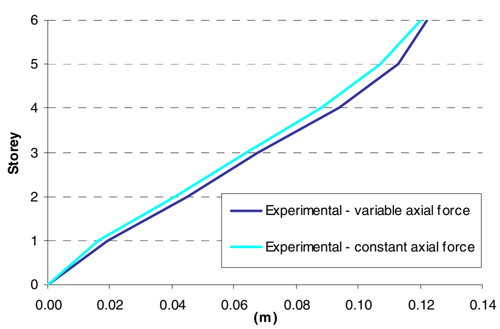


**Figure 22 – Maximum horizontal displacements with and without consideration of the axial force variations.**

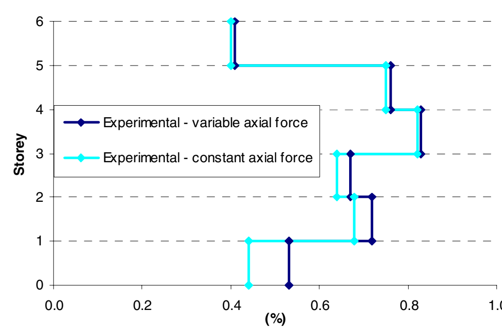


**Figure 23 – Maximum inter-storey drifts with and without consideration of the axial force variations**.

The response of the "Experimental" structure where the axial force variations were accounted for tends to exhibit damage levels and ductility demands, in both beams and columns, with globally slightly higher values than the ones obtained when keeping the monotonic envelopes constant during the analysis. Regarding the response of the "Irreg1" structure, a similar trend was observed except in the beams of the first three stories where an opposite tendency was found. However, the increase in column demand is quite large and causes inelastic behavior in these elements to spread throughout the structure's height. In the case of the "Irreg2" structure beams exhibit slightly lower values of the demand up to the fourth storey

when accounting for the axial force variations during the analysis. In the other hand columns exhibit mores significant lower values of the demand in the first two stories. Finally for the "Regular" structure, a mixed trend was found, columns exhibit lower values of the demand while beams exhibit higher values. In the overall, the changes were not very significant, with the exception of some values of the "Irreg1" structure for the 0.493g intensity where ductility levels in the columns are increased to values close to 2.0. With respect to these findings, it must be pointed out that they were expected since the values of $\nu$ at the bottom storey, Eq. (1), where $N$ represents the axial force, $A_c$ is the cross section area and $f_c$ is the uniaxial concrete compressive strength, were around 0.11.
Globally, an increase of the lateral deformability was detected when accounting for the axial force variations during the analysis but regarding the ductility and damage demands, an overall trend could not be identified for this effect.

$$\nu = \frac{N}{A_c f_c} \tag{1}$$

**Behavior of the "PT-Design" structure**
Based on the member dimensions of the "PT-Design" structure previously presented in Table 1, this structure can be expected to be much more flexible then the others. Considerable differences between the response of this structure and of the "Experimental" one can be observed in Figs. 24 to 27 where the maximum horizontal displacements and inter-storey drifts of both structures and for the 0.319g and 0.493g ground motion intensities are presented.

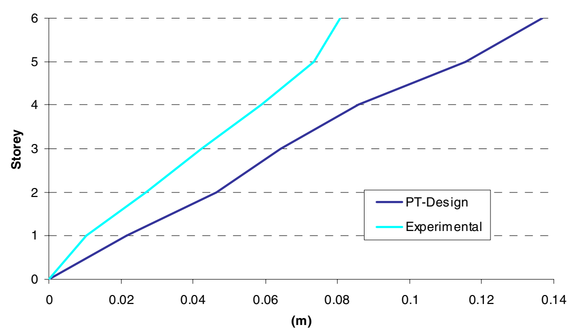


**Figure 24 – Maximum horizontal displacements for the 0.319g intensity.**

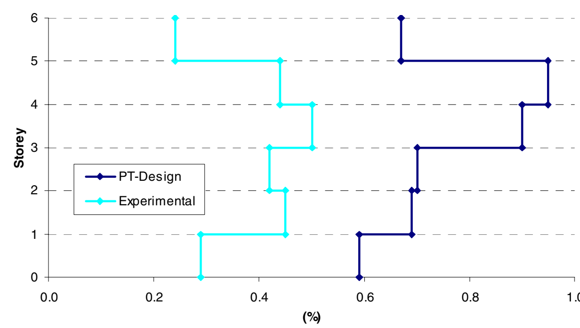


**Figure 25 – Maximum inter-storey drifts for the 0.319g intensity.**

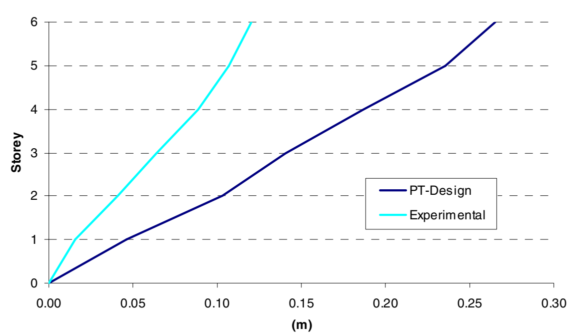


**Figure 26 – Maximum horizontal displacements for the 0.493g intensity.**

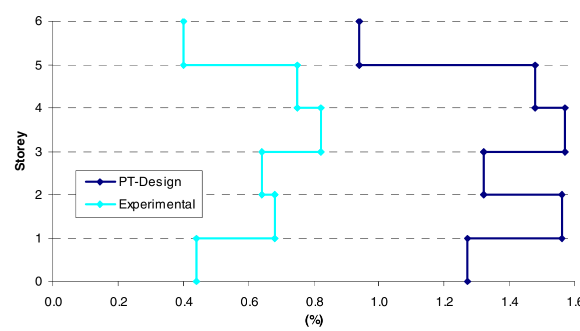


**Figure 27 – Maximum inter-storey drifts for the 0.493g intensity.**

Besides the previously mentioned higher flexibility, there are other less adequate aspects about the response of the “PT-Design” structure. With the exception of the 0.493g intensity, a tendency to concentrate lateral deformations at the setback level was found.
With respect to the damage and ductility levels, it was also found that larger demands were now concentrated in the first three stories and especially in beams. However, yielding of the columns already occurs for the 0.166g intensity. For the 0.493g ground motion intensity, columns exhibit a maximum ductility of 4.5 while beams exhibit maximum values over 15.0 up to the setback level and maximum values of 4.0 above it. Regarding the damage levels, they are very close to 1.0 for beams up to the setback level and not higher that 0.3 for the remaining elements.
In the overall, the “PT-Design” structural behavior cannot be considered as adequate as the one from the other analyzed structures since ductility and damage demands exhibited up to the setback level are much higher than for the rest of the structure. By comparing its behavior with the one of the “Experimental“ structure, the “PT-Design” structure can be seen to behave more like an irregular structure. Although much more flexible than the other structures, its behavior for a very intense earthquake is still acceptable, bearing in mind that the design PGA values are lower than the 0.319g intensity ground motion and that a lower global overstrength level exists since capacity design principles were not considered.

## CONCLUSIONS

The results of a study dealing with the assessment of the dynamic behavior of RC frame structures with irregularities in elevation were presented in this paper. The effects of the variations of the axial force in the columns and of different contributions of slab widths to the beams’ flexural strength were also investigated for increasing ground motion intensities.

When trying to simulate the experimental behavior of the structure “Experimental”, it was found that its mechanical properties at the beginning of the shaking table tests did not correspond to uncracked stiffness conditions. In order to match the experimental results for the 0.077g intensity, it was necessary to consider that the members possessed an initial stiffness equal to their cracked stiffness. However, this assumption was not sufficient to obtain an adequate agreement for the 0.166g intensity. Since the accelerograms used in the analyses were not the ones used in the experimental program, the reasons for the encountered differences could not be fully identified. However, due to the interaction between the shaking table platform and the test structure, additional rotational accelerations were measured during the experimental tests. This secondary motion that was not included in the numerical simulations could be an additional reason for the poor matching between experimental and analytical results.
Regarding the structure’s frequencies, a good agreement was, nonetheless, found between experimental and analytical values.

For the ground motion intensities and structural configurations that were considered in this study, it was seen that, with the exception of the “PT-Design” structure, the several irregular structures exhibit, in global terms, an adequate structural performance when compared to the one of the regular structure. Higher demands of the structural parameters that were used to assess structural behavior at the setback level were only noticed for the “Experimental” and “Irreg2” structures. In the overall, the ductility demand was low, less than 3.0, both in beams and in columns. However, except for the “Experimental” structure, columns exhibiting inelastic behavior were found. Except in the case of the “PT-Design,” these values were lower than 2.0 and restricted to the first two stories.

Regarding the cases where the effect of different contributions of slab widths to the beams’ flexural strength were analyzed, a clear identification of the full nature of the influence of this parameter was found to be very difficult. This influence changes considerably from structure to structure and also according to the ground motion intensity. In global terms, when considering the slab width contribution

the obtained results suggest that an increase in column ductility demand and damage can be expected. In some cases, this increasing demand can also be detected in beams, especially for higher ground motion intensities. With respect to maximum displacements and inter-storey drifts, the influence of the slab width contribution is less clear. These quantities seemed, however, to increase in the lower stories and to decrease in the upper ones. Therefore, a concentration of the lateral deformations in the lower stories can be generally expected when taking into account the slab width.
It was also observed that large slab widths did not have more significant effects when compared to the ones obtained using smaller slab widths. However, this conclusion is limited to the range of the analyses that were carried out.

Regarding the effects of taking into account the axial force variation during the analyses, it was also seen that its influence is not significant when comparing the dynamic responses of cases where this effect was not accounted for. An increase of the lateral deformability was detected but regarding the ductility and damage demands, a global trend could not be identified for its influence.

With respect to the "PT-Design" structure, its behavior was found to be more characteristic of an irregular structure with large inelastic demand concentration up to the setback level. Although following current design codes, additional measures should have been considered to reduce inelastic demand and lateral deformability. Nonetheless, though much more flexible than the other structures, its behavior for a very intense earthquake is still acceptable, bearing in mind that the design PGA values are lower than the 0.319g intensity ground motion and that a lower global overstrength level exists since capacity design principles were not considered.

As a final observation, with the exception of the "PT-Design" structure, the structural behavior of the different buildings was seen not to differ much with respect to inelastic demand. In addition, accounting for the axial force variations and for different contributions of slab widths to the beams' bending behavior was found not to be very relevant for these structures. This fact seems to be closely related to the rather excessive member dimensions of the experimentally tested structure that was selected. Nonetheless, the use of this example shows that an adequate dynamic response can be obtained in irregular structures. On the other hand, the use of the "PT-Design" structure demonstrates that the design of this kind of structures based solely in code requirements may not be sufficient and can lead to structures with inadequate dynamic behavior.

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