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Effect of Concrete Class Variation on Dynamic Response of Framed Structures

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ABSTRACT -- *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

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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. 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 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 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 m2 for all the beams and 0.66*0.51 m2 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.

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Such characteristics can be related to the search for a weakbeam/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 structurepresent 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 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 m2 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

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 CostaCosta 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

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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 wasconsidered 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

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

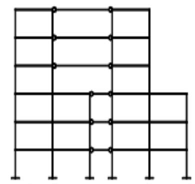

Figure 2 – Structural configuration of the analyzed direction of the "Experimental" structure.

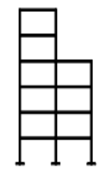

Figure 3 – "Irreg1" structure.

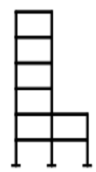

Figure 4 – "Irreg2" structure.

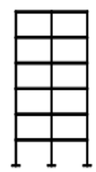

Figure 5 – "Regular" structure.

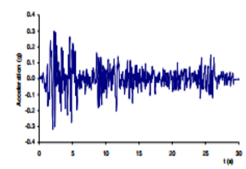

Figure 6 – Accelerogram of the 1940 El Centro earthquake.

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 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

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.

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REFERENCES

1. Moehle, J. P. “Seismic response of vertically irregular structures,” J. of Struct. Eng. ASCE, 1984, 110, 2002-2014.

2. Costa, A., “Influence of vertical irregularities on seismic response of buildings,” Proc. 9th WCEE, 1988, vol V, Tokyo, Japan.

3. Cassis, J. H. and Cornejo, E. “Influence of vertical irregularities in the response of earthquake resistant structures,” Proc. 11th WCEE, 1996, Acapulco, Mexico.

4. Cruz, E. F. e Cominetti, S., “Influence of irregularities in height and different design criteria on the inelastic response of building models”, Proc. 11th WCEE, 1996, Acapulco, Mexico.

5. Valmundsson, E. V. and Nau, J. M. “Seismic response of building frames with vertical structural irregularities,” J. of Struct. Eng., 1997, 123, 30-41.

6. Magliulo, G. et al “Seismic behaviour of irregular in elevation plane frames,” Proc. 12th ECEE, 2002, London, England.

7. Magliulo, G. et al “A critical review of seismic codes provisions for vertically irregular frames,” Proc. 3rd European Workshop on the Seismic Behaviour of Irregular and Complex Structures, 2002, Florence, Italy.