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Study regarding the stiffness influence of slab to beams for a plan structural reinforced concrete frame system in seismic zones

To cite this article: I Sococol *et al* 2019 *IOP Conf. Ser.: Mater. Sci. Eng.* **586** 012013

Study regarding the stiffness influence of slab to beams for a plan structural reinforced concrete frame system in seismic zones

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Abstract. For monolithic reinforced concrete structures, it is known that beams and slabs form a common body, so that the stiffness of the dissipative elements (beams) increases significantly. Also, it is known the fact that the beams are the principal structural dissipative elements. In these circumstances, it will try through numerical simulations (nonlinear calculation) a theoretical reproduction of a recently executed structure, so as to take into consideration the effect of excess rigidity brought to the horizontal dissipative structural elements (beams). It will be pursued the dissipation mode of seismic energy through plastic deformations (formation the punctual plastic hinges at the end zones of the beams and especially at the end regions of the columns).

1. Introduction

The design of reinforced concrete frame systems concerns to use the ductile design concept, for which it is necessary to activate the plastic energy dissipation zones by deformations in the nonlinear domain. These areas are considered to be the end regions of the beams and the end zones of the columns at the base of the structures. As beams become through this design concept the essential dissipative elements, the interest of their seismic response is primordial.

Because of the fact that in the last decade have occurred a significant number of severe earthquakes, it can be observed the real response of the designed and executed reinforced concrete frame structures in the corresponding seismic zones. In the considerable majority of cases it was observed the difference between the real structural seismic response and the critical areas of the structural elements considered by the designers. In these conditions, the beams are not seismic energy dissipators and the columns work intensively in the nonlinear domain with important deformations in the critical zones not only at the base of the structure.

Thus, through by this study it is desired a more realistic knowledge of the seismic response for the monolithic reinforced concrete frame systems, following the stiffness influence of slab to beams and structure. It also follows the dissipation mode of seismic energy through plastic deformations.

2. General data

In this case study is presented biographical calculation (nonlinear static) and time-history method for a planar reinforced concrete structure: Axis 2, figure 1(a), figure 1(b) from a hotel with the P+3E height regime located in Bucharest, performed with SAP2000 program [14].

Description of component parts from structural system:

- Reinforced concrete frame structure;
- Frame opening from structural system: 5.1 (m);
- The height of the floors: 3 (m);
- Marginal columns reinforcement (C_{01}): figure 2(a);
- Central columns reinforcement (C_{02}): figure 2(b);
- Beams reinforcement (B_{01}): figure 2(c);

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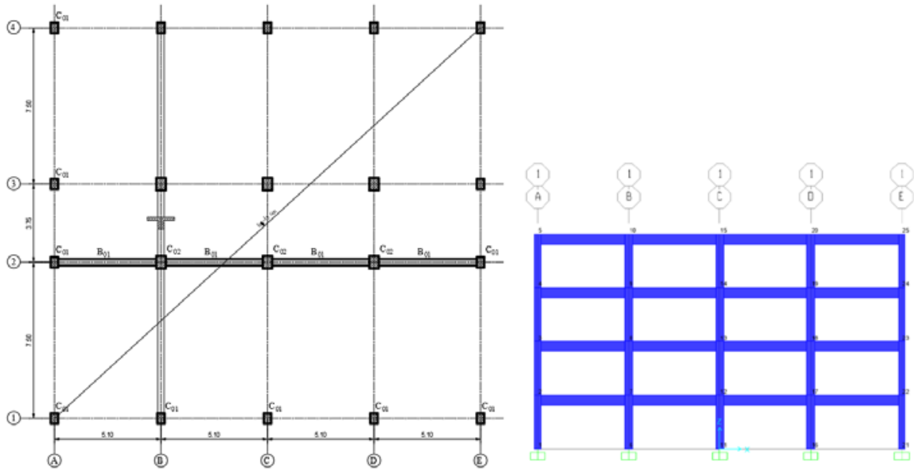
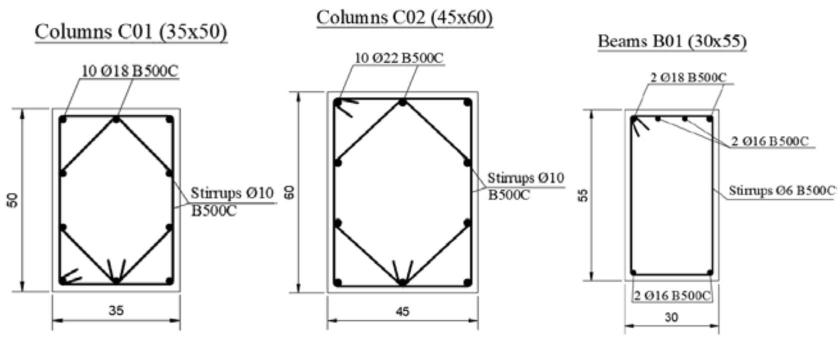
Materials used: Concrete C20/25, Steel S500 (trade name: B500C) [12].



(a) (b)

Figure 1. (a) Current level plan with representation of axis 2; (b) Structural representation of the frame system in X-Z plan [14].



(a) (b) (c)

Figure 2. Columns reinforcement: (a) marginal, (b) central; (c) beams reinforcement.

Seismic loads:

- Used loads: table 1;
- Class of importance for structure: III;
- The structure was designed for high ductility class: DCH;
- Seismic parameters for Bucharest city: according to P100-1;
- Hysteretic degradation model: Takeda bilinear.

Accelerograms and elastic acceleration spectra corresponding to them:
Six accelerograms were used for non-linear Time-History analysis:
-two synthetic accelerograms generated by a specialized software: QKE_1 figure 3(a), QKE_3 figure 3(b) [2];
-two natural accelerograms (recorded): INCERC Bucharest 1977 N-S figure 3(c) and INCERC Bucharest 1986 E-W figure 3(d), scaled to maximum acceleration (PGA=0.30g) [13];
-the same natural accelerograms but compatibilized figure 3(e), figure 3(f) (scaled such that the elastic acceleration spectrum of the code corresponds to the elastic acceleration spectrum for the fundamental vibration period) [2].

In this order, lower is presented 6 accelerograms and 6 elastic acceleration spectra figure 4(a)-(f) corresponding to them [2],[3],[10],[13].

Table 1. Used loads.

Load Pattern	Story	Values	Units
Dead	GF; S1; S2	1.2	kN/m^2
	S3	2.5	kN/m^2
Live	GF; S1; S2; S3	1.5	kN/m^2
Snow	S3	1.6	kN/m^2
Seismic	-	-	-

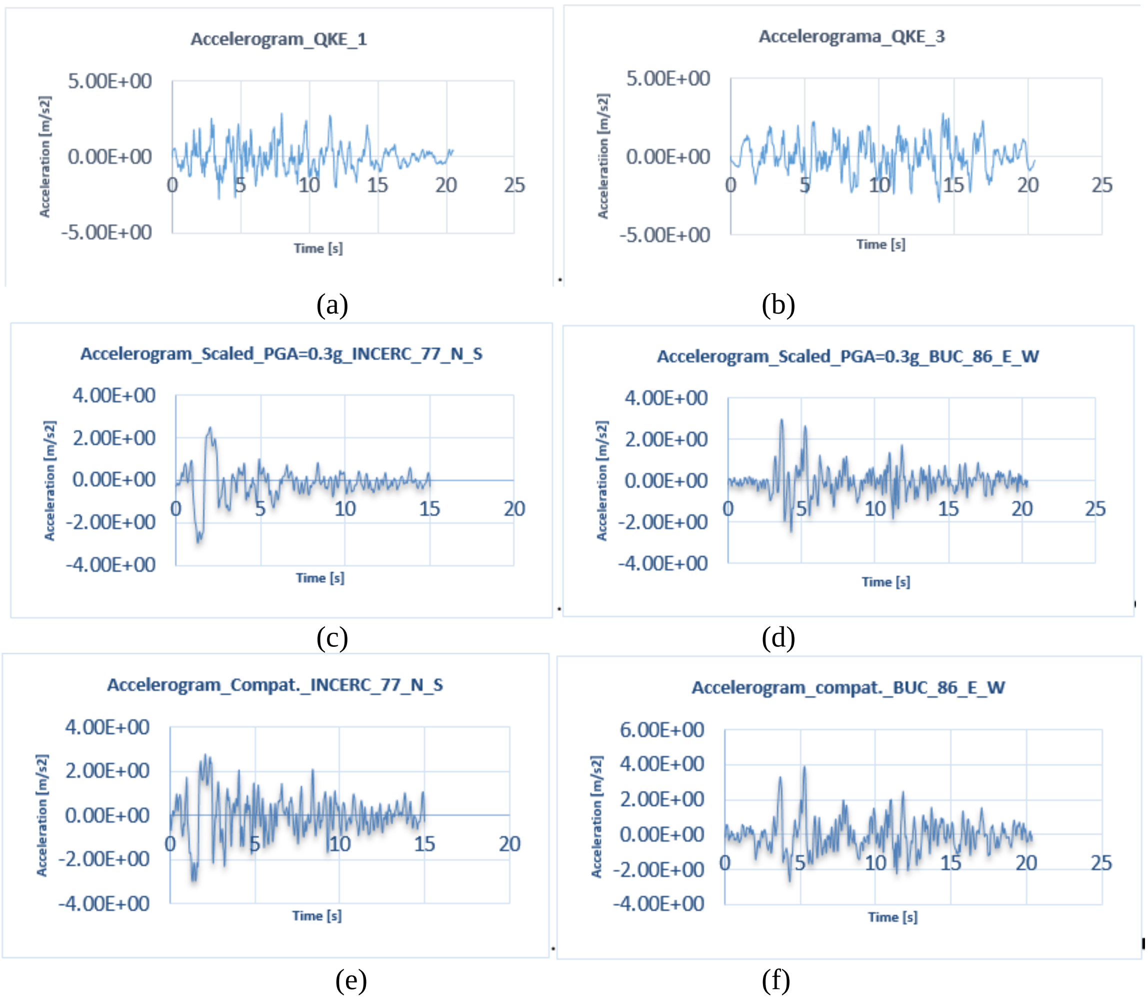


Figure 3. Accelerograms:(a) QKE_1; (b) QKE_3; (c) INCERC_77_N_S_Scaled_PGA=0.30g; (d) BUC_86_E_W_Scaled_PGA=0.30g; (e) INCERC_77_N_S_Compatibilized by spectral design value criteria (PGA=0,304g); (f) BUC_86_E_W_Compatibilized by spectral design value criteria (PGA=0,397g).

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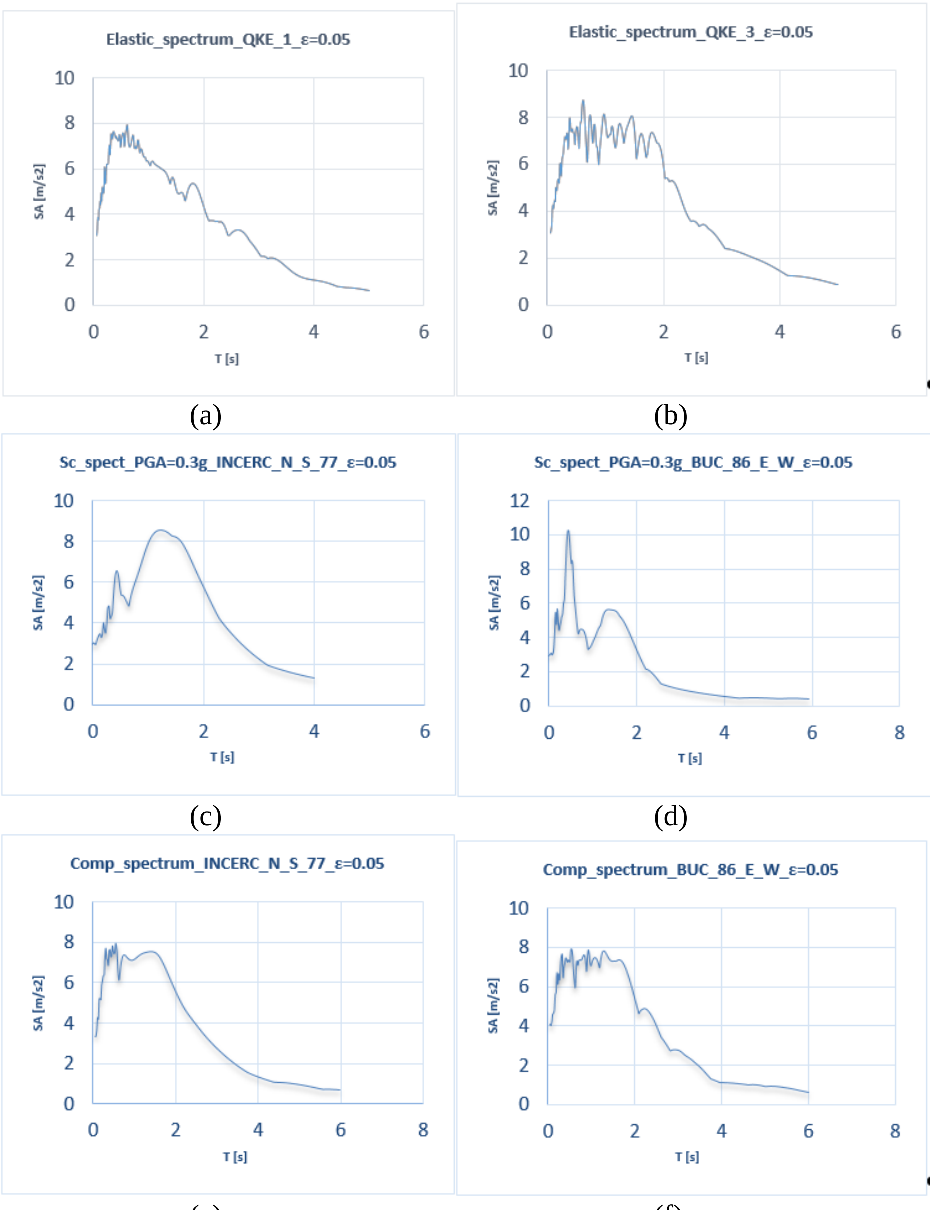


Figure 4. The elastic acceleration spectrum corresponding to the accelerogram: (a) QKE_1; (b) QKE_3; (c) INCERC_77_N_S_Scaled; (d) BUC_86_E_W_Scaled; (e) INCERC_77_N_S_Compatibilized; (f) BUC_86_E_W_Compatibilized.

The study mode and the influence parameter of the seismic response for structural system
Four study cases were considered with the flexural stiffness of the beams as variable:1EI; 1.5EI; 1.7EI; 2EI. The mode of considering the beams stiffness is idealized, reducing the stiffness of the beams-slab assembly to the stiffness of the “pure” section of the beams.

Hypotheses regarding the seismic response of the structural system:

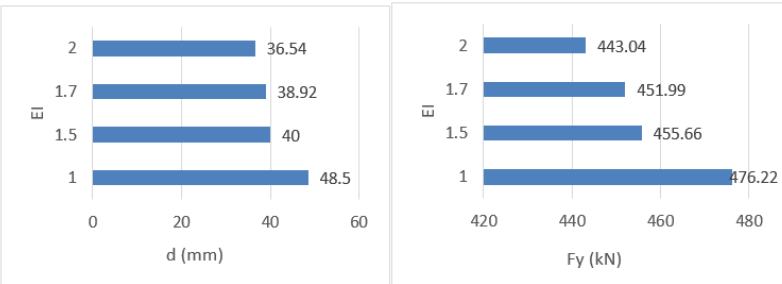
- The rigidity of the beams-slab assembly influences the seismic response of the structural system with incursions in the inelastic domain through degradation and formation of plastic hinges in columns. This phenomenon can produce a floor (storey) mechanism [8],[9];

- The rigidity of the beams-slab assembly influences the seismic response of the structural system with incursions in the inelastic domain through degradation and formation of plastic hinges in beams [4],[5]. This structural response in the nonlinear domain corresponds to the design concept of "weak beams-strong columns": according to the current norms [10],[11] etc.

The veracity of these hypotheses can be demonstrated (interpreted) by the results obtained from nonlinear analysis (Push-Over and Time-History).

3. Push-Over analysis

For the analyzed case, two load distributions were used: (1) uniform distribution of lateral forces; (2) distribution corresponding to the fundamental vibration mode, [3],[10]. Following the bilinearization of the lateral force-displacement curve, have resulted the variation of the displacement requirement represented in figure 5(a), the variation of the lateral force corresponding to the structural system yielding figure 5(b), the variation of the fundamental period figure 6, having as a parameter the bending stiffness of the beams.



(a) (b)

Figure 5. (a) Displacement requirement variation for structural systems according to the stiffness variation of the beams; (b) The lateral force variation corresponding to the yield of the structural systems, depending on the bending stiffness of the beams.

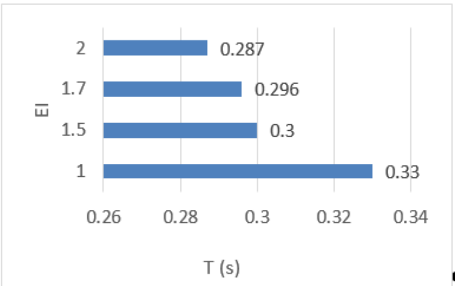
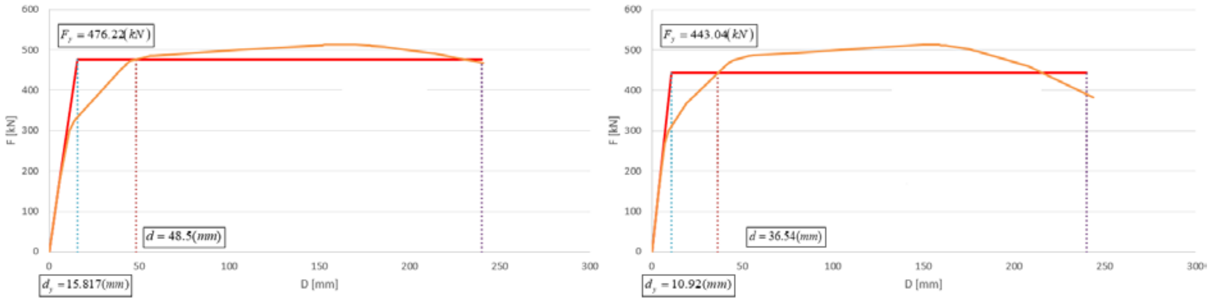


Figure 6. Variation of the fundamental period for structural systems according to the variation of bending stiffness of the beams.

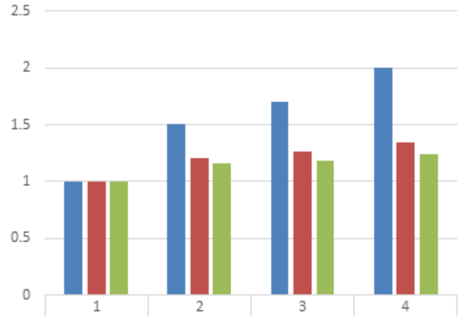
The lateral force-displacement curves bilinearizated according with calculation based on corrected elastic spectra from P100-1 [10], function of bending stiffness *EI* for beams are shown in figure 7(a) and figure 7(b).

With all this data, it is possible to easily determine the stiffness influence of the beams to rigidity of the structure (figure 8), the total seismic load reduction factor (q), the seismic force reduction factor appropriate to structure ductility (q_μ), structure over-strength (q_S), design over-strength (q_{Sd}), redundancy or capacity for plastic redistribution of efforts (q_R) in figure 9, variation of seismic design force (F_d), elastic force (F_e), force corresponding to formation of the first plastic hinge (F_1), yielding force (F_y), depending on the bending stiffness *EI* of the beams (figure 10).



(a) (b)

Figure 7. Lateral force-peak displacement curves: (a) EI; (b) 2EI.



	1	2	3	4
Rigidity_beams	1	1.5	1.7	2
Struct._elastic_rigidity	1	1.204946461	1.266855146	1.347524853
Struct._elast._equiv._rigidity	1	1.160152346	1.182742228	1.234833785

Figure 8. The bending stiffness influence to global rigidity of the structure.

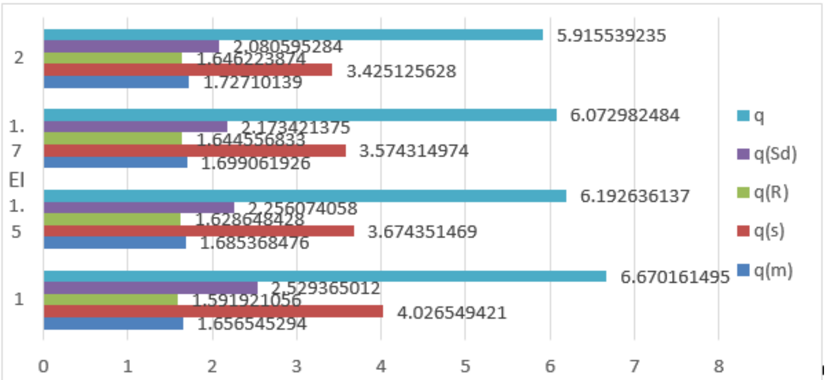


Figure 9. Variation of seismic load reduction factors.

Structure response to static actions according to the Push-Over mode of action can also be represented graphically through color mode of plastic hinge formation for each loading step in all four cases of bending stiffness. In this way, it is presented lower figure 11(a),(b) structural response for triangular loading, step 5 for *EI* and *2EI* rigidity. It was chosen step 5 of the load because in this step is touched the yield force for structural system and the displacement requirement. Thus, it can be seen what happens with the structure for maximum horizontal load.

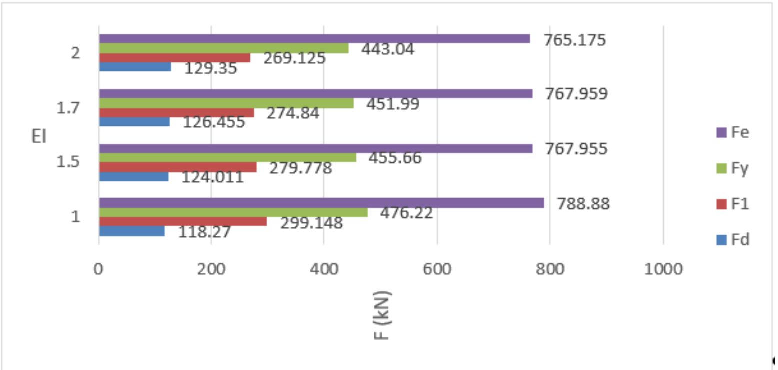
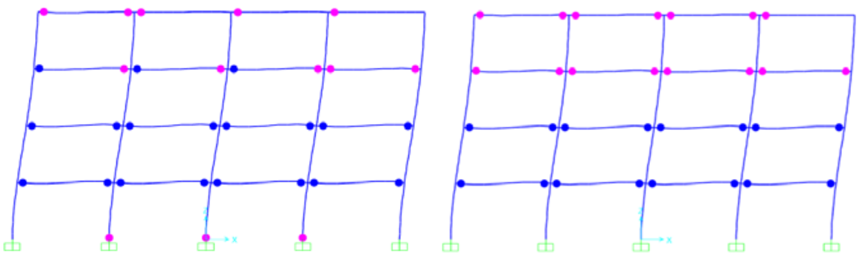


Figure 10. Variation of elastic forces, yielding forces, forces corresponding to formation of the first plastic hinge, designing forces.



(a) (b)

Figure 11. The seismic response of the structural system through formation the plastic hinges (Step 5, Push_Triang.): (a) EI; (b) 2EI, [14].

The colors of the plastic hinges correspond to the American codes [6],[7] who take in consideration four levels of performance in seismic design figure12, figure 13.

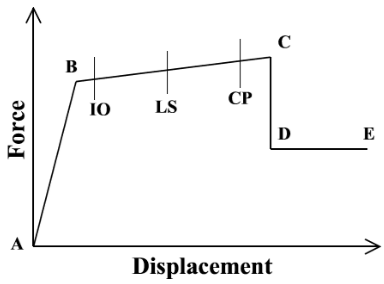


Figure 12. Lateral force-displacement curve according to FEMA 356 [6].

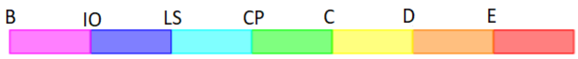


Figure 13. The colors representing each level of performance [14].

Thus, according to FEMA 356 [6] the translation of the performance levels in figure 12 and figure 13 are:

O - Operational Level;
IO - Immediate Occupancy Structural Performance;
LS - Life Safety Structural Performance Level;
CP - Collapse Prevention Structural Performance Level.

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4. Non-linear Time-History analysis

To perform the most efficient evaluation methods for seismic response of structures, it is necessary to establish the accelerogram, hysteretic degradation laws of the material, the damping properties of the structure etc. [3]. Compared with linear time-history analysis, the nonlinear analysis takes into account rigidity parameters and degradation effects. Thus, the real-time response of the structure can be monitored and can be known a multitude of requirements (displacements, rotations, accelerations etc.) on a structural assembly and element [1],[11].

Further, will be presented the hysteretic acceleration-displacement curves for last level (node 5) only for beams with *EI* (figure 14) and *2EI* stiffness (figure 15). Thus, it is possible to accentuate the structural response in displacements:

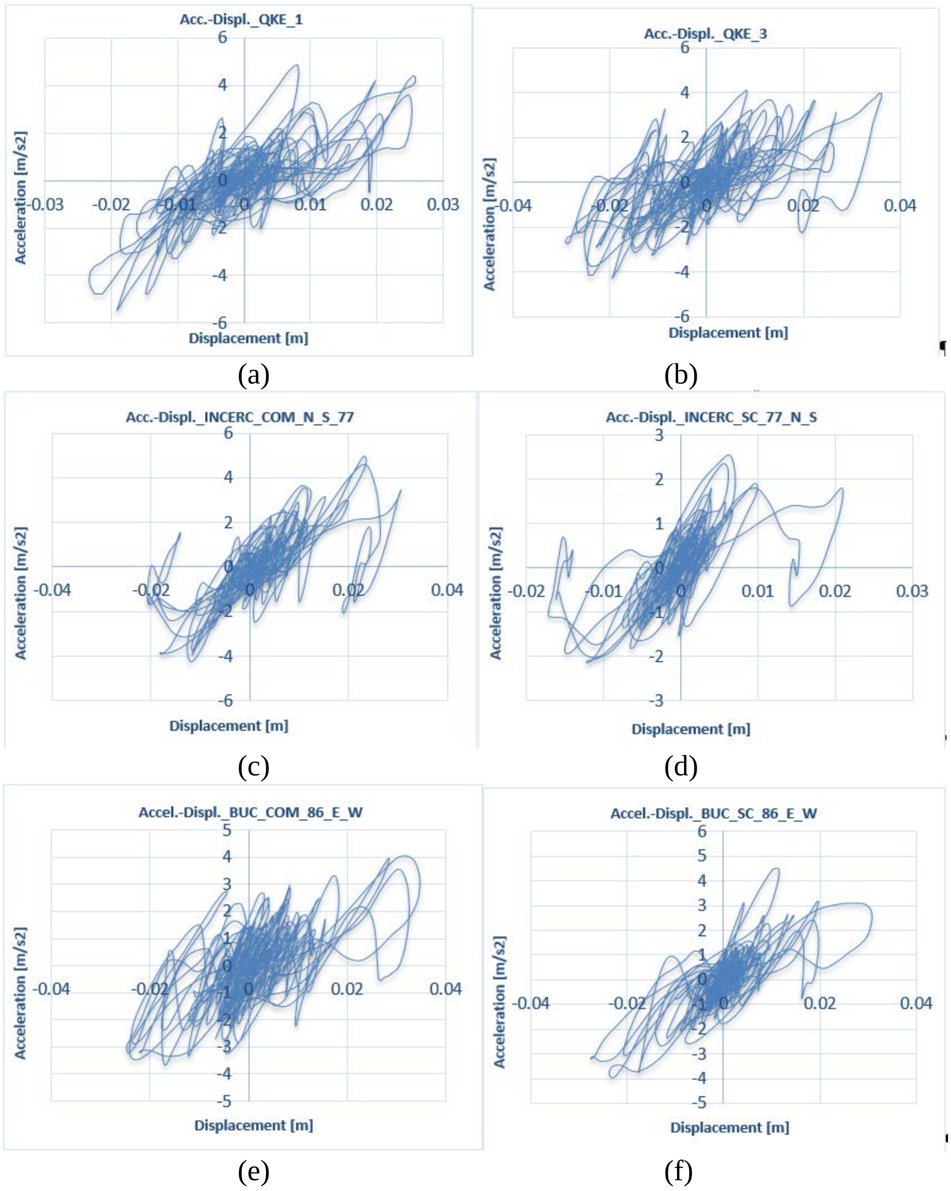

Figure 14. Hysteresis curves corresponding to Non-linear Time-History analysis for beams with EI stiffness: (a) QKE_1; (b) QKE_3; (c) INCERC_77_N_S_Compatib.; (d) INCERC_77_N_S_Scaled; (e) BUC_86_E_W_Compatib.; (f) BUC_86_E_W_Scaled.

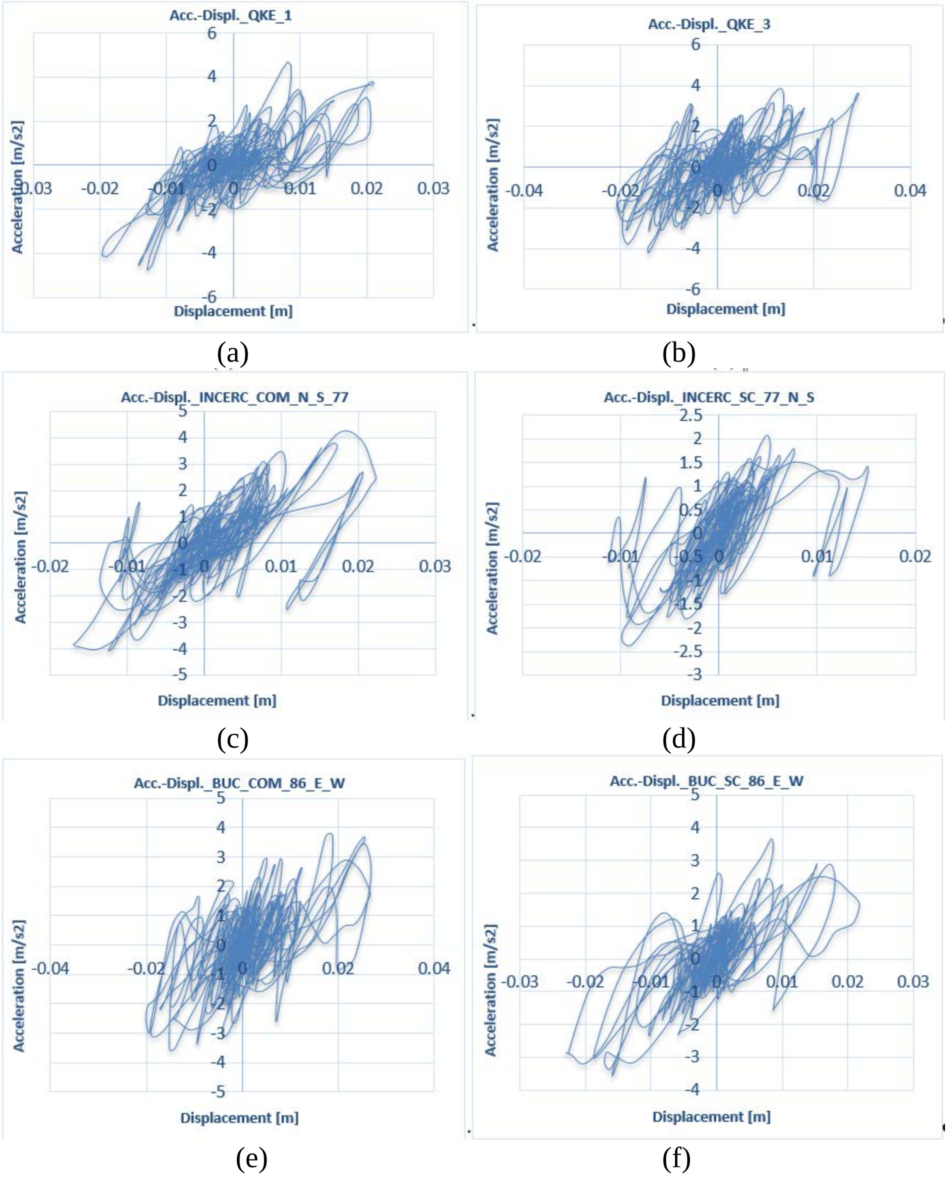

Figure 15. Hysteresis curves corresponding to Non-linear Time-History analysis for beams with 2EI stiffness: (a) QKE_1; (b) QKE_3; (c) INCERC_77_N_S_Compatib.; (d) INCERC_77_N_S_Sc.; (e) BUC_86_E_W_Compatib.; (f) BUC_86_E_W_Sc.

5. Conclusions

Push-Over analysis

(1) Increasing the bending stiffness of the beams, results a decrease with 25% of the displacement requirements for structural system (in the case if the maximum rigidity is considered to be 2EI) and a small decrease of the lateral force corresponding to the yield of the system. These phenomena are explained by a development of the structural rigidity imposed by the stiffness of the beams. Thus, for 2EI bending stiffness of the beams, the elastic rigidity of the structure increase with 35%. This effect can be verified by gradually decreasing of the fundamental vibration period for structural system.

(2) With increasing stiffness of the beams (structural dissipative elements) the global seismic load reduction factor (the behavior factor *q*) decreases (from 6.67 for EI to 5.9 for 2EI). In addition to this process, can be noted the decrease of structural over-strength (from 4.02 for EI to 3.42

for 2EI). The same phenomenon is also produced for elastic seismic forces, yielding forces, forces corresponding to formation of the first plastic hinge or designing seismic forces, but the differences are very small (which implies not considering them).

(3) This information helps us to understand the significant contribution of slab rigidity to structural stiffness. This can be better understood through graphical representation of the structure's response by forming plastic hinges in the considered dissipative areas (end zones for beams and columns). Thus, it is visible how the number of beams who touch the LS (Life Safety) performance decreases at the same time with developing bending stiffness. Of course, in this case the number of beams affected by plastic deformations decreases, but unfortunately the problem must be seen differently, because the seismic designing forces, yielding forces or elastic forces do not vary much, their values are close. In these conditions, it is evident how the structure is loaded approximately with the same level of horizontal excitation (grace to its maximum capacity to load with seismic forces). Thus, if plastic deformations do not occur in the dissipative elements (beams), then these deformations occur in vertical elements (columns) who works theoretically in elastic domain. In these circumstances, some design principles need to be revised.

Non-linear Time-History analysis

(1) For non-linear time-history analysis, it can be observed through hysteretic curves a close response of the structure following synthetic and compatibilized accelerograms. Thus, the compatibility method based on spectral design values is more efficient than the accelerogram scaling method for maximum acceleration (our PGA case =0.30g).

(2) Also, according to the acceleration-top displacement curves (response lines) of the structure, it is visible an extensive range of their activity, due to deformations in the considered dissipative elements. In addition to this, displacement requirements decrease in the same time with developing bending stiffness.

(3) Through non-linear dynamic analysis it was possible to confirm the final conclusion from the Push-Over analysis with regard to the influence of the plastic deformation capacity of the beams.

(4) Non-linear time-history analysis demonstrate how the structure possesses an important redistribution capacity and more over-strength than was considered to be. Proof is the values of the base shear forces, which are much higher compared to the design seismic forces. In this way, it was possible to appreciate the structure's capacity to deform in potentially plastic areas.

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