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EXAMPLES OF EXPECTED SEISMIC BEHAVIOR OF EXISTING BUILDINGS

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ABSTRACT

Three reinforced concrete frame buildings were designed twice according to an older building code (Puerto Rico 1968) and according to a modern code (Puerto Rico 1987). Their expected behavior to seismic excitations were compared, and failure modes determined. The older buildings showed less displacement capacity, but were also initially stiffer than newer buildings. The older buildings might be able to resist an earthquake similar to TAFT, but would fail for stronger earthquakes.

KEYWORDS

Nonlinear Analysis, Monotonic, Existing Buildings, Anchorage Length, Seismic Evaluation

INTRODUCTION

Recently developed tools for nonlinear analysis of reinforced concrete buildings subjected to strong ground motions allow comparisons of calculated behavior of buildings designed using different concepts. This paper summarizes the results of one study in which buildings with the same geometry were designed according to the Puerto Rico building codes of 1968 and 1987. The buildings were analyzed using a computer program that permits modeling of nonlinear behavior of reinforced concrete elements, including approximate provisions for shear failure and for loss of anchorage.

DESCRIPTION OF BUILDINGS

Figure 1 indicates the general layout of buildings. Six regular buildings were designed (two 5-story, two 10-story, and two 15-story buildings), applying the 1968 and 1987 Puerto Rico codes (ARPE 1968, 1987) . The first code does not have special provisions to guarantee ductile behavior of buildings and seismic detailing considerations. The second one is an updated code based on the Uniform Building Code (UBC 1985).

The material strength was considered equal for all buildings, f'c = 3000 psi. and fy = 60,000 psi. Other parameters like importance factor, occupancy category, soil profile, etc., were kept equal for all buildings. However, the design procedure for buildings designed under the 1968 code was the working stress method, and for the 1987 code the strength design method was used. The element dimensions and reinforcement were changed every 2 stories for columns, and every 4 levels for beams.

Table 1 contains the final design of the 1st story exterior column and 1st story beam for the six buildings. Note the differences in sizes, longitudinal amount of reinforcement and confinement index. The confinement index was calculated as the ratio between the volume of stirrups and the volume of the confined concrete core per unit length. Total weight per level was 467 Kips for the typical interior frame.

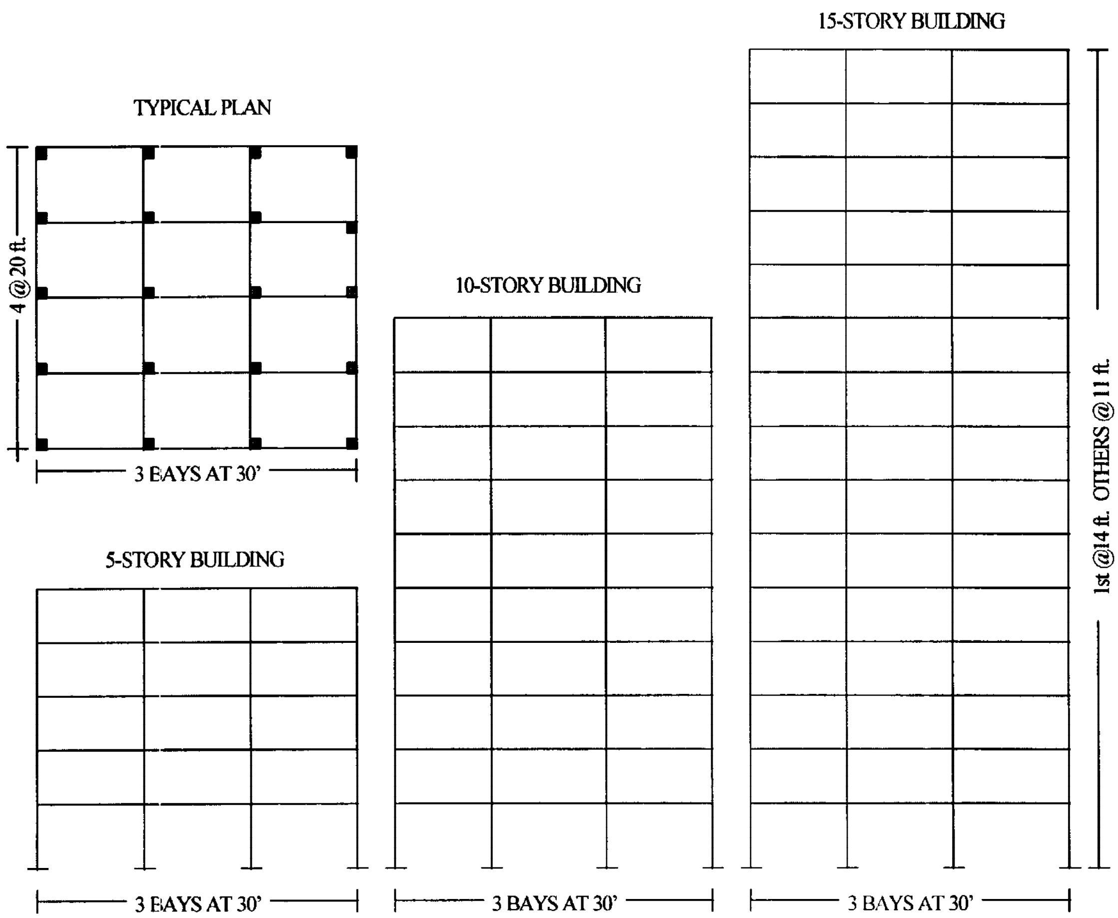

Fig. 1. General Layout of Buildings

Table 1. Final Sections and Reinforcement for Exterior Column and Beam at 1st Story

Column / Beam Description	1968's Code			1987's Code		
	b x h	Steel [%]	Conf. Index	b x h	Steel [%]	Conf. Index
Ext. Column 5-Story Building	26x26	1.8	0.0016	24x24	3.1	0.0079
Ext. Column 10-Story Building	32x32	1.6	0.0069	30x30	2.4	0.022
Ext. Column 15-Story Building	39x39	1.9	0.0074	32x32	2.7	0.021
1st Story Beam 5-Story Building	22x30	1.1	0.0021	18x26	1.6	0.006
1st Story Beam 10-Story Building	30x42	1.04	0.0034	22x30	1.9	0.007
1st Story Beam 15-Story Building	30x42	1.16	0.0025	22x32	1.6	0.0058

COMPUTATIONAL PROGRAM

LARZWS and LARZWD are the latest static and dynamic versions of the nonlinear analysis program developed by Saiidi and Sozen (1979) at the University of Illinois. These programs are applicable to reinforced concrete frame structures and shear wall buildings. The major features of these programs are the following:
- Analyze 2D structures under lateral loads including P-Delta effect
- Use the Takeda hysteretic routine
- Considers perfectly fixed supports at the base
- Considers a horizontal DOF per level and condenses the rotational DOFS in terms of the horizontal.
- The nonlinear properties of the elements are modeled with the "one component model" (see Figure 2). This model considers two inelastic rotational springs at the ends of each member and an elastic portion along the clear span. The rotational springs are defined based on the moment-curvature diagram for each end cross section.

The programs have been modified in order to evaluate possible failure mechanisms (Daza 1996). These new features are the following:
- Includes the effect of the gravitational loads,
- The static analysis can be carried out under displacement or force control,
- Detects structural failures due to flexure, shear, insufficient development length of bars, excessive interstory drift ratio and excessive horizontal displacement,
- Calculates the hysteretic energy absorbed by every joint of the structure,
- Computes local and global damage indexes due to flexure and shear.

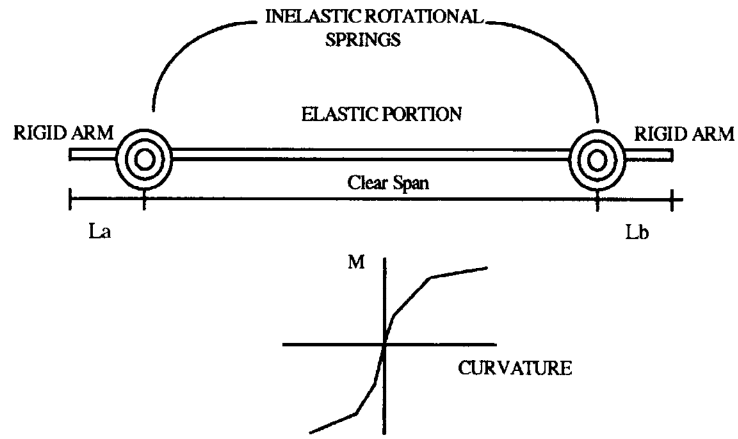

Fig. 2. The "One Component Model"

RESULTS

Static Analysis

The six buildings described above were subjected to an incremental lateral load pattern, following the equivalent static loading procedure defined in the P.R. 68 and P.R. 87 codes. The Load distribution in both codes is similar to the 1985 Uniform Building Code.

Figures 3 through 5 indicate the monotonic curves for the typical interior frames of these buildings. Table 4 summarizes the major results of the unidirectional monotonic analysis and the calculated failure modes; "shear" means shear failure of any element, "flexure" means excessive rotation at any joint and "excessive drift" means interstory drift exceeding 2%. The insufficient anchorage length is included in the flexural failure mode (affecting the moment-curvature diagram).

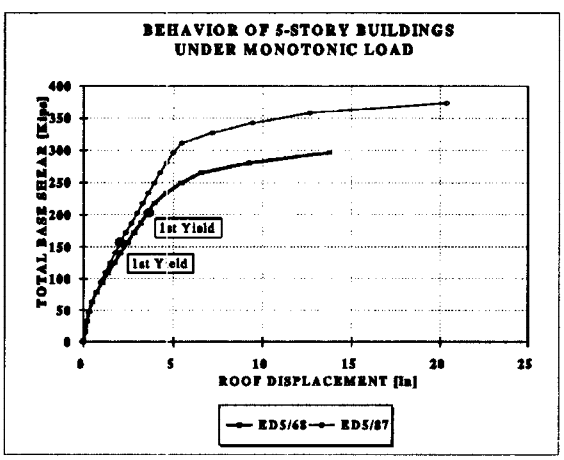

Fig. 3. Monotonic Behavior of 5-Story Buildings Designed under P.R.-68 and P.R.-87 Codes.

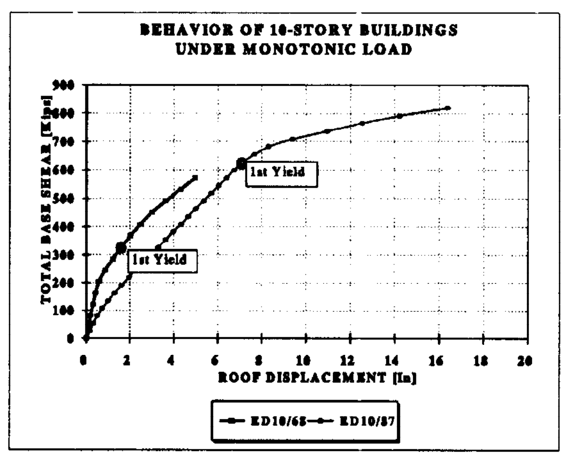

Fig. 4. Monotonic Behavior of 10-Story Buildings Designed under P.R.-68 and P.R.-87 Codes.

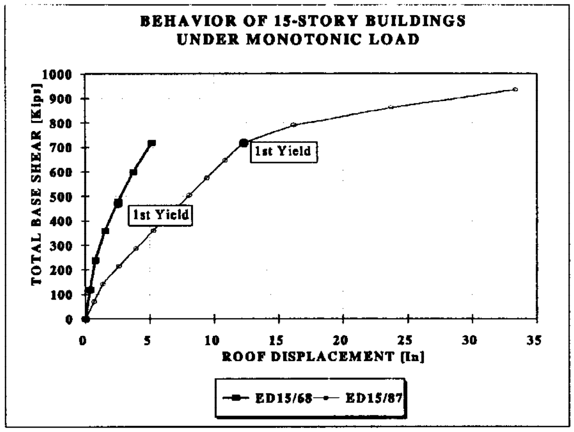

Fig. 5. Monotonic Behavior of 15-Story Buildings
Designed under P.R.-68 and P.R.-87 Codes

Table 4. Summary of Results under Static Loading

Building	Code	At 1st Yielding		At Failure		Transla-tional Ductility	Failure Mode
		Load [Kip]	Displ. [In]	Load [Kip]	Displ. [In]		
5-Story	P.R.-68	203.1	3.54	296.0	13.8	3.9	Shear & excessive drift
	P.R.-87	156.6	2.03	373.5	20.4	10.0	Flexure & excessive drift
10-Story	P.R.-68	327.6	1.60	573.3	4.97	3.1	Flexure
	P.R.-87	627.9	7.09	819.0	16.4	2.3	Flexure & excessive drift
15-Story	P.R.-68	479.6	2.54	719.4	5.2	2.0	Flexure
	P.R.-87	720.6	12.2	936.8	33.3	2.7	Flexure & excessive drift

Dynamic Analysis

Five seismic records were used to study the response of these buildings: Taft (1952, S-69-E, Tehachapi, Ca), El Centro (N00E 1940, Imperial Valley), Mexico (Sept 19/85, EW component, SCT record), Northridge (Jan 17/94, Castaic NS record) and San Salvador (Oct 10/86, CIG record). The Taft record has been proposed as representative of a major earthquake for Puerto Rico (ARPE 1987).

Figures 6 through 11 show the maximum interstory drift ratios for each building and Figures 12 through 17 contain the maximum response of each building in terms of base shear and roof displacement, against the original Load .vs. Displacement curve.

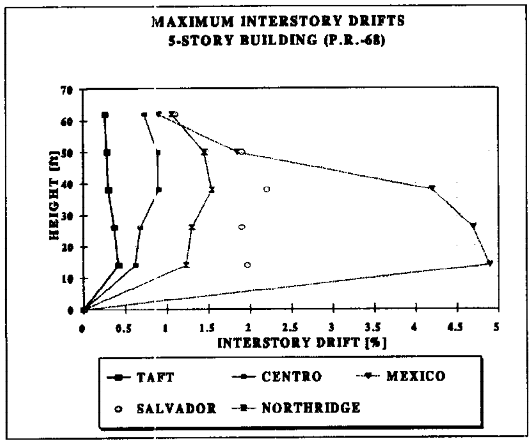

Fig. 6. Interstory Drift Ratios for ED/5/68

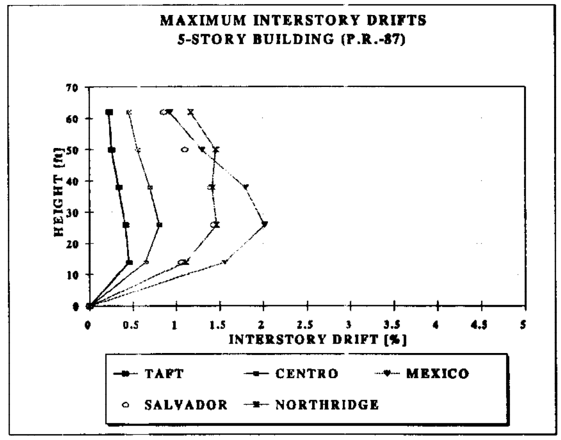

Fig. 7. Interstory Drift Ratios for ED/5/87

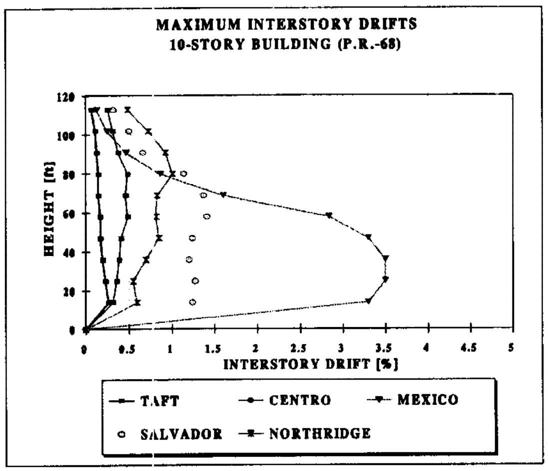

Fig. 8. Interstory Drift Ratios for ED/10/68

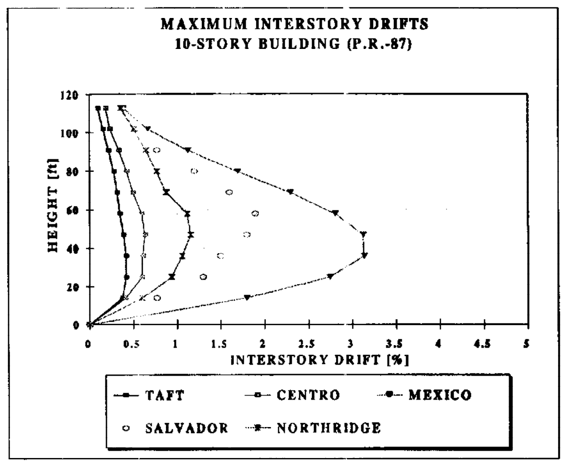

Fig. 9. Interstory Drift Ratios for ED/10/87

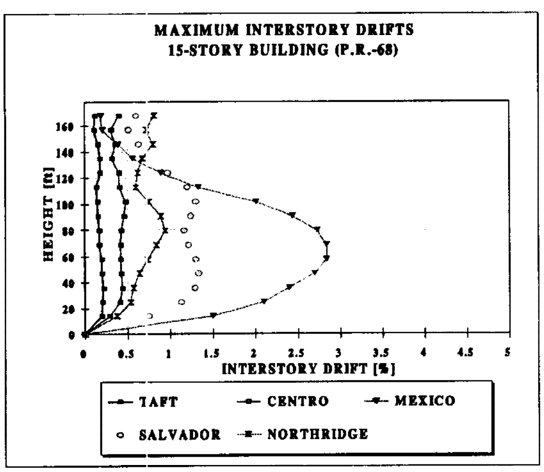

Fig. 10. Interstory Drift Ratios for ED/15/68

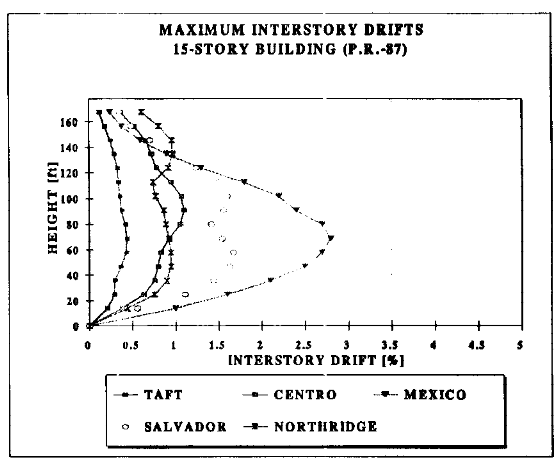

Fig. 11. Interstory Drift Ratios for ED/15/87

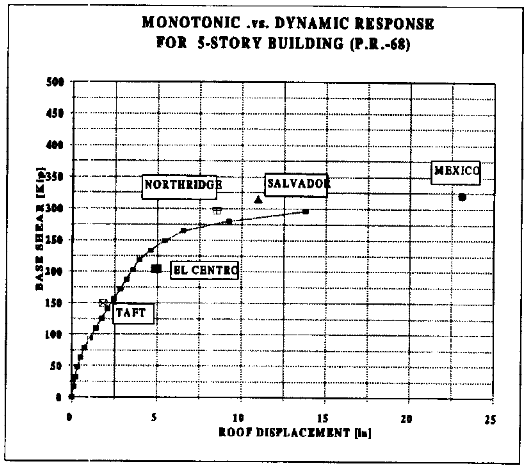

Fig. 12. Dynamic Response for ED/5/68

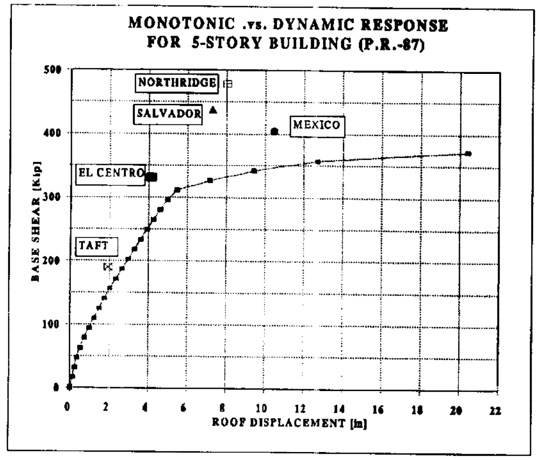

Fig. 13. Dynamic Response for ED/5/87

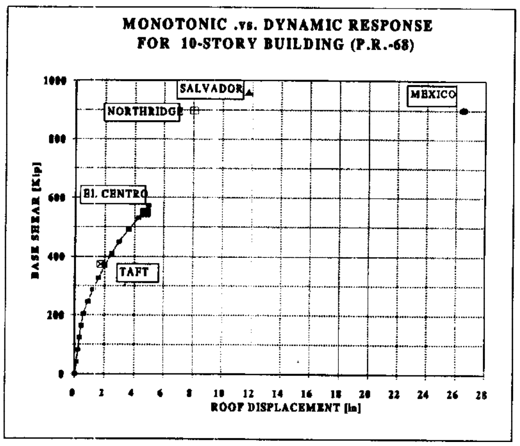

Fig. 14. Dynamic Response for ED/10/68

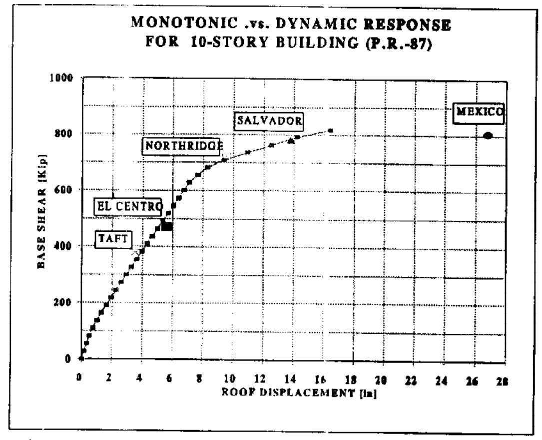

Fig. 15. Dynamic Response for ED/10/87

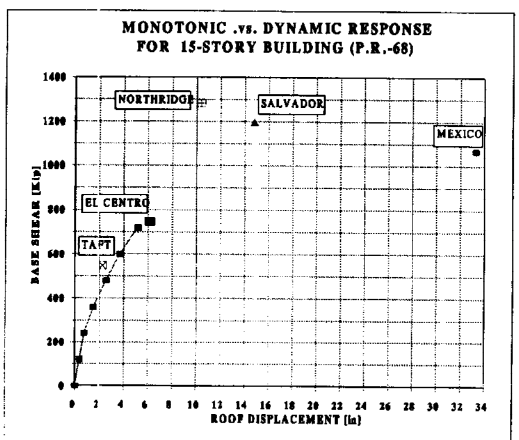

Fig. 16. Dynamic Response for ED/15/68

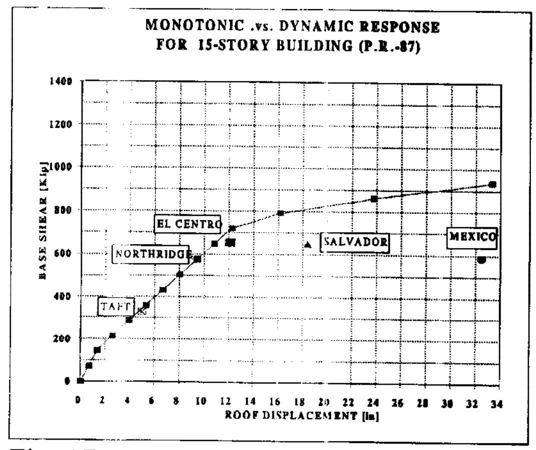

Fig. 17. Dynamic Response for ED/15/87

The calculated fundamental period of the buildings (uncracked section) were: 0.4, 0.53, 0.60, 0.87, 0.89 and 1.17 seconds for 5-story/68, 5-story/87, 10-story/68, 10-story/87, 15-story/68 and for 15-story/87, respectively.

CONCLUSIONS

- In general, PR 87 buildings are more flexible and stronger than PR 68 buildings.
- PR 87 buildings have substantially more deformation capacity than 68 buildings.
- All buildings seemed capable of withstand the Taft ground motion. Interstory drifts did not exceed 0.5%, and calculated base shears were at or below yield.
- The El Centro record was close or exceed the maximum capacity of PR 68 buildings. However, PR 87 buildings seemed capable of resisting El Centro earthquake with moderate or little damage.
- PR 87 buildings might sustain excessive damage in Northridge or San Salvador earthquakes. They might collapse in Mexico earthquake.
- The monotonic curves for these buildings reflects a more realistic behavior because have been obtained considering all the possible structural failures, and not only the traditional flexural concept.

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