EVALUATION OF DIFFERENT TYPES OF PUSHOVER ANALYSES FOR R\C FRAME STRUCTURES

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ABSTRACT

The objective of this study is to evaluate the different types of push over analyses for R\C frame structures with variety of natural periods. To realize the analyses, four different type of R/C frame structures are chosen. The evaluation is done by performing pushover and nonlinear dynamic time history analyses. Even though the nonlinear dynamic time history analysis is known as the best way to compute seismic demands, FEMA-273 and ATC-40 proposes to use of nonlinear static procedure or pushover analysis. Three different pushover methodologies are taken into consideration. These are classical, modal and energy based pushover analyses. The push over analyses are evaluated with nonlinear dynamic time history analyses. The load distributions for classical pushover analysis are chosen as triangular (IBC, k=1), IBC (k=2) and rectangular. In modal push over analysis, higher mode effects are taken into account, in energy based push over analysis, energy parameters are considered. The time history analysis is realized by 30 different earthquake data from all over the world. This paper is intended to compare the results of pushover analyses regarding with nonlinear dynamic time history analyses. In the present study, it is aimed to evaluate accuracy of the pushover analyses according to fitting of the curves to time history analyses results for each type of frame structures.

KEY WORDS

Classical, Modal and Energy based Pushover Analyses, Nonlinear Time History Analysis, R\C frame structures

INTRODUCTION

The objective of this study is to evaluate the performance of pushover analyses in different methodology for frame structures with variety of natural periods by performing pushover and nonlinear dynamic time history analyses. 3, 5, 8 and 15 story R\C frame structures are used in the analyses. Three different pushover analyses are realized as classical, modal and energy based. For the classical pushover analysis, the load distributions are chosen as triangular (IBC, k=1), IBC (k=2) and rectangular, where k is the an exponent related to the structure period to define vertical distribution factor (IBC, 2000). The four frame structures have been

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analyzed using nonlinear program DRAIN-2D (Prakash, V., Powell, G., Campbell, S., 1993) and the results have been compared by recorded response data. Both nonlinear static pushover analyses and nonlinear dynamic time history analyses are performed. The correlations between these nonlinear analyses are studied. For this study, it is considered as totally 30 different data used in the nonlinear dynamic time history analyses, in the Table 1.

DESCRIPTION OF THE FRAME STRUCTURES

3, 5, 8 and 15-story R\C frame structures with typical cross-sections and steel reinforcements are shown in Figure 1. The reinforced concrete frame structures have been designed according to the rules of the Turkish Code. The structures have been considered as an important class 1 with subsoil type of Z1 and in seismic region 1 . The dead, live and seismic loads have been taken account during design. All reinforced concrete frame structures consist three-bay frame, spaced at 800 cm. The normal story height is 300 cm; the first story height is 400 cm. The columns are assumed as fixed on the ground. Yield strength of the steel reinforcements is 22 kN/cm^2 and compressive strength of concrete is 1.6 kN/cm^2. The first natural period of the 3-story frame structure is computed 0.54 s. The cross section of all beams in this frame is rectangular-shapes with 30 cm width and 60 cm height. The cross section of all columns is 40cmx40cm. The first natural period of 5-story frame structure is 0.72 s and the cross section of beams is 30 cm width and 60 cm height similar to 3-story frame. Cross section of columns at the first three stories is 50cmx50cm and at the last two stories, it is 40cmx40cm. For the eight-story frame, the cross section of all beams is rectangular-shapes with 30 cm width and 60 cm height. The 8-story frame structure has 70cmx70cm columns for the first five stories and 50cmx50cm for the last three stories The natural period of this structure is 0.90 s. For 15-story frame structure, has natural period as 1.20 s. The cross section of beams for 15-story frame structure is 40cmx70 cm. The cross section of columns for first 8 stories in the 15-story frame structures is 90cmx90cm and at the last seven stories, it is 70cmx70cm. These dimensions are shown in the figure 1.

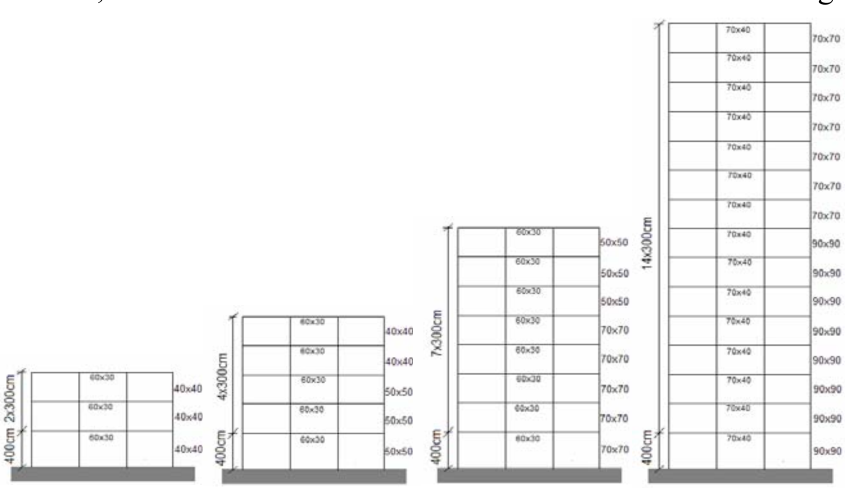

Figure 1. Two-dimensional frame models for the four R/C structures

CLASSICAL NONLINEAR STATIC PUSHOVER ANALYSIS

For performance levels, to estimate the demands, it is required to consider inelastic behavior of the structure. Pushover analysis is used to identify the seismic hazards, selection of the performance levels and design performance objectives. In Pushover analysis, applying lateral loads in patterns that represent approximately the relative inertial forces generated at each floor level and pushing the structure under lateral loads to displacements that are larger than the maximum displacements expected in design earthquakes (Li, Y.R., 1996). The pushover analysis provides a shear vs. displacement relationship and indicates the inelastic limit as well as lateral load capacity of the structure. The changes in slope of this curve give an indication of yielding of various structural elements. The main aim of the pushover analysis is to determine member forces and global and local deformation capacity of a structure.

The classical pushover curves are sketched for three distributions, and for each frame structures. The curves represent base shear-weight ratio versus story level displacements for uniform, triangular and IBC load distribution. Shear V was calculated by summing all applied lateral loads above the ground level, and the weight of the building W is the summation of the weights of all floors. Beside these, these curves represent the lost of lateral load resisting capacity and shear failures of a column at the displacement level. The changes in slope of these curves give an indication of yielding of various structural elements, first yielding of beam, first yielding of column and shear failure in the members. By the increase in the height of the frame structures, first yielding and shear failure of the columns is experienced at a larger roof displacements and rectangular distribution always give the higher base shear-weight ratio comparing to other load distributions for the corresponding story displacement (horizontal displacement).

MODAL NONLINEAR STATIC PUSHOVER ANALYSIS

The Multimode Pushover Analysis (MPA) procedure of Chopra and Goel (2002) combines quantities determined in independent modal pushover analyses. The capacity curve determined for the equivalent" SDOF system for the first mode load pattern of the MPA procedure is identical to that determined in the first mode pushover method of ATC-40. Thus, the following discussion will refer to the nth mode pushover of the MPA procedure, recognizing that the specialization for n = 1 applies equally to the ATC-40 first mode pushover. The MPA procedure proposes to estimate peak dynamic response quantities of inelastic structures based on a combination of possibly nonlinear responses obtained independently for each mode. The structure is subjected to a static force distributed over the height of the building with amplitude increasing until the roof displacement equals or exceeds the maximum displacement, expected in each mode. The peak modal responses, each determined in an independent modal pushover analysis, are combined according to the SRSS method to obtain an estimate of the peak value of the total response. When applied to structures responding in-elastically, the method neglects the influence of the modal forces sn on the response of other modes. That is, superposition is assumed, or alternatively, interaction among the modes is neglected, just as in elastic modal analysis.

ENERGY BASED NONLINEAR STATIC PUSHOVER ANALYSIS

In the energy-based method, the work done by the lateral forces acting through their corresponding floor displacements is used to derive an "energy-based" displacement. This energy-based displacement is plotted on the abscissa of the capacity curve, in contrast to the use of the roof displacement with conventional capacity curves. The energy based displacement is derived in such a manner that the elastic portion of the capacity curve matches the elastic portion of the conventional pushover curve, thus preserving the theoretical equivalence of the elastic portion of the pushover response and the modal response described by Chopra and Goel (2002).

The incremental displacements, $\Delta D_{e,n}$, are accumulated to obtain the displacement, $D_{e,n}$, of the ESDOF system at any given step in the analysis. The capacity curve of the ESDOF system is a plot of $(V_{b,n} / \alpha_n W)$ as a function of $D_{e,n}$, where α_n *is* modal mass coefficient for the nth mode and W *is* weight of the structure. The peak displacement of the ESDOF system then may be estimated using a smoothed design spectra using the capacity spectrum method, the displacement coefficient method, inelastic spectra, or yield point spectra, or by means of a nonlinear dynamic analysis of a SDOF system having properties defined by the capacity curve. In the energy-based displacement approach, the estimated or computed peak displacement corresponds to a particular step in the nonlinear static pushover analysis, and it is at this step that the peak roof displacement and other response quantities can be identified for the MDOF system.

NONLINEAR DYNAMIC TIME HISTORY ANALYSIS

After performing pushover analyses, nonlinear dynamic time history analyses have been employed to the four different story frame structures. These frames are subjected live and dead weights. Also P-Δ effects are under consideration as in pushover analysis. For time history analysis P-Δ effects have been taken into the account. Finite element procedure is employed for the modeling of the structures during the nonlinear dynamic time history analyses. Drain 2D has been used for nonlinear time history analysis and modeling. The model described for pushover analyses has been used for the time history analyses. Mass is assumed to be lumped at the joints. The frames are subjected to 30 earthquake ground motions from Soil type A, which are recorded during Anza (Horse Cany), Parkfield, Morgan Hill, Kocaeli, Coyota Lake, N. Palm Springs, Northridge, Santa Barbara, Imperial Valley, Cape Mendocino, Kobe, Central California, Lytle Creek, Whittier Narrows, Hollister Westmoreland, Landers, Livermor and Cape Mendocino earthquakes, for the nonlinear dynamic time history analyses.

Table 1. Records Used in The Analyses (Soil Type A)

No	EARTHQUAKE	DATE	(Mw)	RECORD	PGV (cm/s)	PGA (g)
1	Anza (Horse Cany)	25/02/1980	4.9	AZF315	2.6	0.066
2	Morgan Hill	24/04/1984	6.2	G01320	2.9	0.098
3	Coyote Lake	06/08/1979	5.7	G01320	8.3	0.132
4	Landers	28/06/1992	7.3	GRN180	14.1	0.041
5	Landers	28/06/1992	7.3	ABY090	20	0.146
6	Landers	28/06/1992	7.3	SIL000	3.8	0.05
7	Landers	28/06/1992	7.3	29P000	3.7	0.08
8	Loma Prieta	18/10/1989	6.9	G01090	33.9	0.473
9	Loma Prieta	18/10/1989	6.9	SGI360	8.4	0.06
10	Loma Prieta	18/10/1989	6.9	MCH000	3.5	0.073
11	Loma Prieta	18/10/1989	6.9	PTB297	12.9	0.072
12	Lytle Creek	12/09/1970	5.9	CSM095	1.8	0.071
13	N. Palm Springs	08/07/1986	6.0	AZF225	5.8	0.099
14	N. Palm Springs	08/07/1986	6.0	ARM360	3.4	0.129
15	N. Palm Springs	08/07/1986	6.0	H02090	1.8	0.093
16	N. Palm Springs	08/07/1986	6.0	H02000	1.9	0.07
17	Whittier Narrow	01/10/1987	5.3	MTW000	40	0.123
18	Anza	25/02/1980	4.9	AZF225	3.3	0.065
19	Anza	25/02/1980	4.9	PTF135	5.1	0.131
20	Anza	25/02/1980	4.9	TVY135	1.7	0.081
21	Coyote Lake	06/08/1980	5.7	G01-UP	2.5	0.072
22	Düzce	12/11/1999	7.1	1060-E	5.3	0.053
23	Düzce	12/11/1999	7.1	1060-N	11	0.028
24	Hollister	28/11/1974	5.2	G01247	4.0	0.132
25	Kocaeli	17/8/1999	7.4	GBZ000	50.3	0.244
26	Kocaeli	17/8/1999	7.4	GBZ270	30	0.137
27	Cape Mendocino	25/4/1992	7.1	CPM-UP	63	0.754
28	Loma Prieta	18/10/1989	6.9	RIN090	10.4	0.092
29	N. Palm Springs	08/07/1986	6.0	WWT180	34.7	0.492
30	Whittier Narrow	01/10/1987	5.3	MTW090	35	0.036

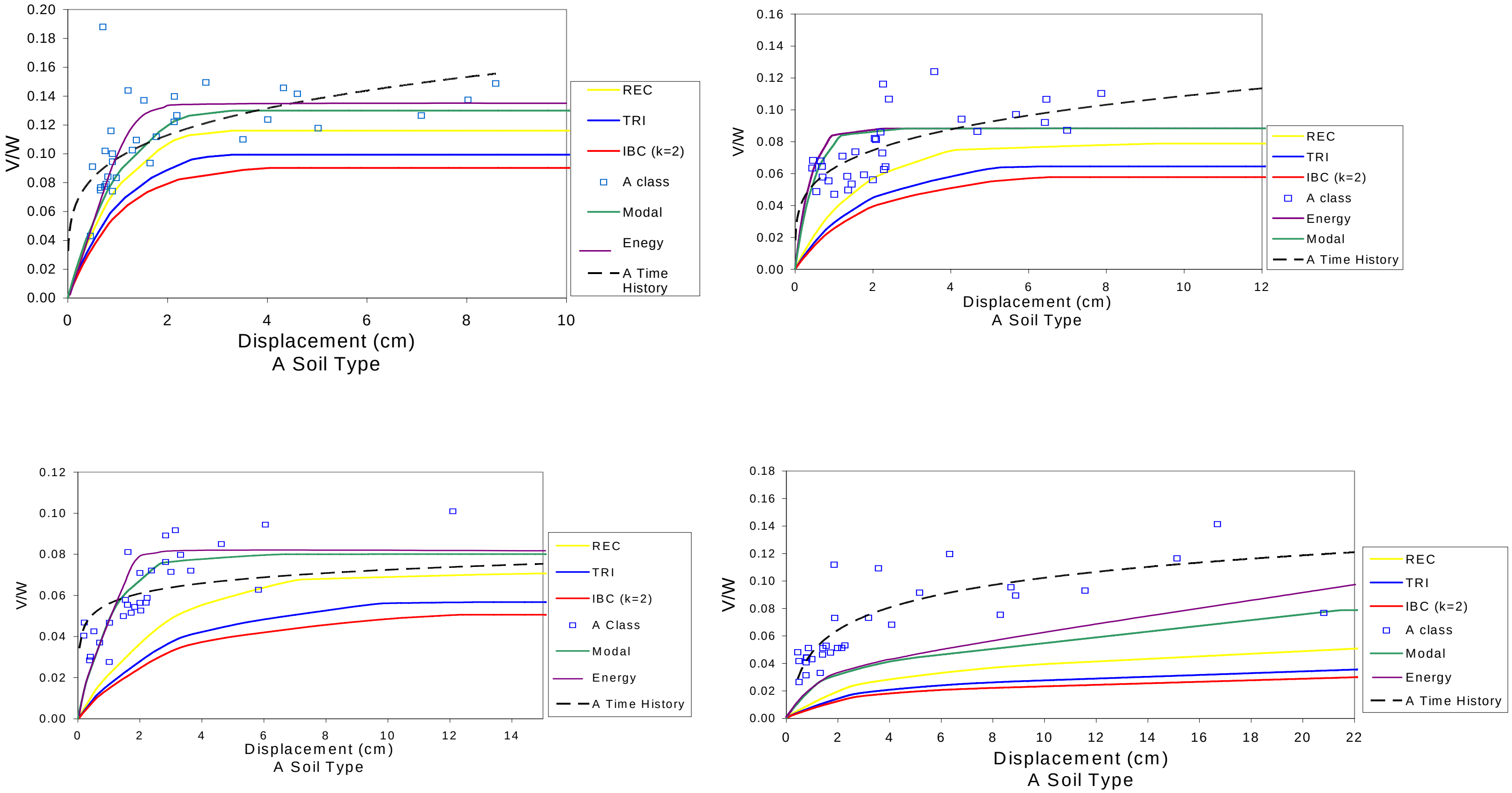

Figure 2. Nonlinear Static Pushover and Dynamic Time History Analyses Results

CONCLUSIONS

After designing and detailing the reinforced R/C frame structures, nonlinear pushover analyses and nonlinear dynamic time history analyses are carried out for evaluating the structural seismic response for the acceptance of load distribution for inelastic behavior. It is assumed for pushover analysis that seismic demands at the target displacement are approximately maximum seismic demands during the earthquake.

For higher story frame structures, first yielding and shear failure of the columns are experienced at the larger story displacements and rectangular distribution always give the higher base shear-weight ratio comparing to other load distributions for the corresponding story displacement. As it is presented in Figure 2, nonlinear static pushover analyses for classical as IBC (k=2), rectangular, and triangular load distribution; modal and energy based and nonlinear time history analyses results for the ground motion data (all of them are near-field data) are compared. Pushover curves do not match with nonlinear dynamic time history analysis results especially for higher story reinforced concrete frame structures (8 and 15-story frame structures). The classical pushover analyses results for rectangular load distribution estimate very close seismic demands to modal and energy based results.

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