INVESTIGATION OF DETERIORATION BEHAVIOR OF HYSTERETIC LOOPS IN NONLINEAR STATICANALYSIS FOR SCMR FRAMES WITH SHEAR WALL

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ABSTRACT

One of the shortages of pushover analysis is that it approximately considers the effects of deterioration pertained to hysteretic loops for structural elements. To evaluate this effect, it is necessary to perform nonlinear static and dynamic analyses and compare the results. To this end, six different planar frames, each one being a part of three-dimensional designed structures, have been modeled in OPENSEES software. All of the structures are the same in plan and different in height. Deterioration behavior as well as non-degrading behavior is considered in time history analyses. Also twelve ground motion records which are scaled to 0.35g hazard level have been used. For modeling the structural elements, stresses and strains for each designed section are considered with respect to confinement effects. The required backbone curve includes cracking, yielding and ultimate stress points which are all derived from USC-RC software, and of course with considering elements cross section, arrangement, number and size of reinforcing bars. For accurate calculation of the target displacement and bilinear idealization of capacity curve, a computer program was developed in MATLAB environment to determine the target displacement, strength ratio (R) and etc.
Finally maximum displacement amounts derived from inelastic dynamic analyses for 0.35g hazard level are compared with those obtained from nonlinear static analyses. Results show that with increasing the height of frame, the variance between frame displacements with both deteriorating and non-degrading behavior will be decreased, so that the effect of deterioration behavior could be neglected in target displacement calculation for high-rise frames.

INTRODUCTION

As most of the buildings which have been built in the past years are designed according to the previous codes and/or even the effects of earthquake have not been considered in their design, proceeding to evaluate these structures is necessary. Quantitative evaluation is one of the methods of evaluating structures. It is usually done by analytical methods which include linear and nonlinear static and dynamic procedures. Considering the behavior of structure in the nonlinear zone, linear methods cannot provide us with an accurate evaluation of it against the effects of earthquake that leads it to the nonlinear stage. Nonlinear methods present more realistic results. Nonlinear dynamic analysis (NDA) is more accurate than nonlinear static analysis (NSA). Since this method is so time-consuming and needs many experts to interpret the results and also sensitivity of results to selection of earthquake records, it cannot be vastly used.Therefore due to its simplicity, the structural engineering profession has been using the nonlinear static procedure (NSP) or pushover analysis, described in FEMA-356 and ATC-40.Nonlinear static method is not only simple but also presents fairly accurate results. Although it has these advantages, it has some shortages such as it approximately considers the effects of deterioration behavior of structural elements under earthquake cycles. So that, many studies have been done in order to evaluation and improvement of this method considering cyclic loading and deterioration parameters. Among all, Akkar and Metin (2007), Goel and Chadwell (2007), Lin and Miranda (2009), Amiri*et al.* (2010) and Jaiyong and Chintanapakdee (2011) can be referred to.

Song and Pincheira (2000) started studying the effects of strength and stiffness degradation on SDOF systems. In a part of their study, they evaluate the displacement coefficients method suggested by FEMA 273. The results have shown that the amounts of displacement coefficients C_1, C_2 and C_3 in FEMA 273, for the systems with periods equal or greater than 0.3 second are more conservative under earthquake excitations on hard soil. Of course it is not the same for the systems which their periods are smaller than 0.3 second and the structure may experience a larger displacement. Furthermore, in order to improve the inelastic seismic analysis procedures, extensive studies on different soil types have been done by Miranda (2002) and presented in ATC55 that leaded to publishing FEMA 440.

One of the features of this research is the applied models which include six SCMR-frames with shear walls that have been designed by ETABS software. It is necessary to perform nonlinear dynamic analyses and compare the results with those of nonlinear static analyses in order to study the deteriorating effects of hysteretic loops. Because OPENSEES software benefits from various behavioral characteristics for steel and concrete, and also has the ability to suitably model the structural elements, it has been used for performingnonlinear analyses. The employed material models are REINFORCING STEEL MATERIAL, CONCRETE02 and HYSTERETIC MATERIAL. Also, twelve ground motion records which are scaled to 0.35g hazard level have been used in time history analyses.

RESEARCH METHOD

In this part we briefly review the method which has been applied in this paper. Six SCMR-frames with shear walls and equal number of spans and various stories (three to twenty-stories) that had been designed and analyzed by ETABS software, have been used in the present study. Nonlinear static analyses have been performed for the designed frames by OPENSEES software. The pushover analyses for each frame was done in accordance with the displacement coefficients method in FEMA 356 and Publication No. 360, and also, considering the lateral load distribution proportional to the story shear distribution including sufficient modes to capture 90% of the total building mass. In order to specify the capacity curve of the structure, the displacement of the control node and the shear force of the base level were computed by this software. For accurate calculation of the target displacement and bilinear idealization of capacity curve, a computer program was developed in MATLAB environment to determine the target displacement, δ_t, strength ratio, R, and etc.

In nonlinear static analysis the deterioration behavior of the structural elements due to ground shaking is approximately considered by coefficient c_2. Of course for accurate estimating of the deterioration effects, time history analyses which reflect the real behavior of the structure should be conducted. According to the research purpose, studying the deterioration effects of hysteretic loops, it is necessary to apply at least two behavioral models in nonlinear dynamic analyses with and without considering the deterioration behavior. OPENSEES software benefits from various behavioral models for modeling the real behavior of concrete and steel. For instance, CONCRETE02 behavioral model has the ability of modeling the behavior of concrete with considering the degrading effects, and REINFORCING STEEL MATERIAL behavioral model is not only able to consider the deterioration effects but it is also able to consider the effects of buckling and fatigue of bars. It is also possible to model the behavior of concrete and steel with or without the deterioration in bilinear or three linear forms with the help of HYSTERETIC MATERIAL. In this way, after conducting the pushover analyses for defining the target displacement, dynamic analyses are also performed considering the two different employed behavioral models. Then the maximum amounts of derived displacements are compared with the amounts of the target displacements, δ_t, which express the maximum displacement of the structure under design earthquake, and consider the effects of different parameters such as deterioration with the coefficient c_2. It is worth mentioning that nonlinear dynamic analyses have been done for 0.35g hazard level. This hazard level is derived by scaling the applied ground motions according to the Iranian Code of Practice for Seismic Resistant Design of Buildings, No. 2800.

The employed material models

OPENSEES software has been used for performing nonlinear static and dynamic analyses. This software is able to produce different hysteretic models using various behavioral models. As it is shown in Fig. 1, backbone curve includes cracking point, yielding point and ultimate stress point which are all derived from USC-RC software, and of course with considering elements cross section, arrangement, number and the size of reinforcing bars. Also, hysteretic stress-strain relation of CONCRETE 02 model in tension-compression is shown in Figs.2(a)-(b).

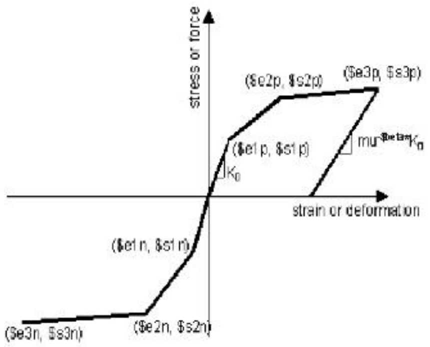


Figure1.Hysteretic Material Parameters

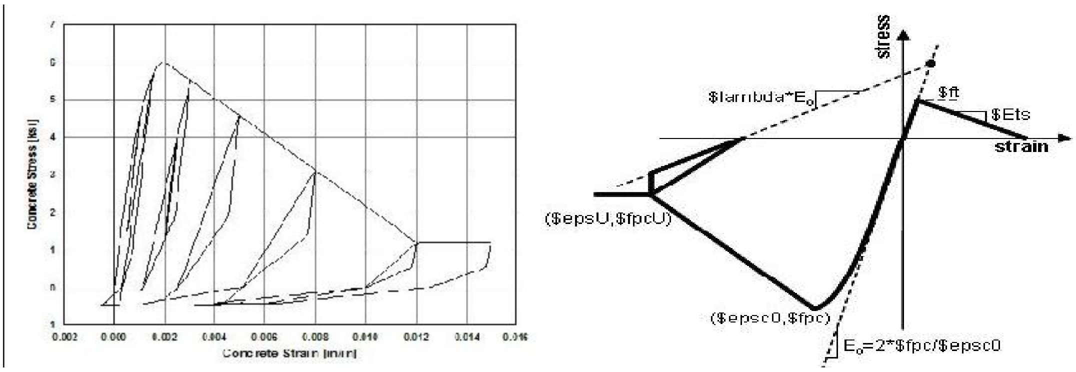


Figure2.CONCRETE 02 material model(left) Hysteretic stress-strain relation (right)

DEFINITION OF STRUCTURAL MODELS

In the present investigation, six planar frames, which are a part of three-dimensional frames, have been used according to Fig.3. Considered lateral load-resisting system of the buildings is SCMR-frame with shear walls in both directions. The buildings were 3,5,7,10,15 and 20-story buildings, so that the role of shear walls in concrete frame will lead to more general results and the responses will include wide range of structural models considering natural periods and seismic performances. All buildings were 25 m by 25 m in plan and comprised five equal bays in each direction. The story heights were equal to 3m for all buildings. They were assumed to be symmetric-plan and vertically regular. Joist system has been used for ceiling, so story and roof dead loads have been considered 650 ($\frac{kg}{m^2}$) and 570 ($\frac{kg}{m^2}$) respectively. Also, floor live load is considered equal to 200 ($\frac{kg}{m^2}$) and boundary walls and parapet loads are considered 600 ($\frac{kg}{m}$) and250 ($\frac{kg}{m}$). Seismic loading of these structures has been done according to the requirements of Iranian Code of Practice forSeismic Resistant Design of Buildings and gravity loading according to Iranian National Building Regulation (INBR-part 6). The buildings are residential which are built on soil type III in an area with high seismicity hazard level.Since all the defined frames in this paper are considered as SCMRF, the coefficient of R is equal to 11. The seismic masses at all floor levels of each building were assumed to be equal andto consist of the dead load plus 20% of the live load. The buildings were designed based on ACI318requirements by ETABS software and satisfied the detail requirements ofmentioned code including strong column-weak beam criterion. It is worth mentioning that walls have been designed in a way to possess boundary columns according to provisions mentioned in ABA-04 code.Frames periods are as follow: 0.251, 0.46, 0.66, 0.996, 1.954 and 2.676 seconds.

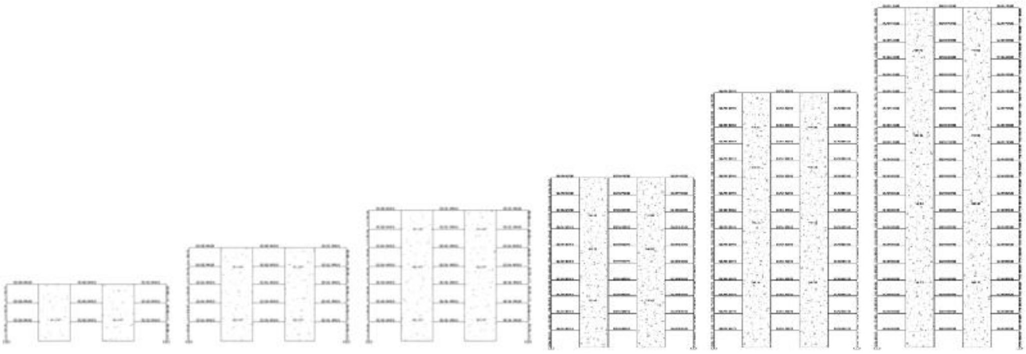


Figure3.Studied Frames

GROUND MOTIONS

According to the main objective of the research, studying the deteriorating behavior of hysteretic loops, which is mainly caused by ground shaking, it is necessary to perform nonlinear dynamic analyses under various ground motions. In the present study, twelve ground motion records were used in time history analyses which are scaled according to the Iranian Code of Practice for Seismic Resistant Design of Buildings, No. 2800 and Publication No. 360. More details of the ground motion records are presented in Table 1.

Ground-Motion Scaling

Comparing the effects of different earthquakes, we need to scale acceleration time histories. In this paper the employed earthquakes have been scaled to 0.35g hazard level according to Fig. 4 for doing dynamic analyses. The proceeding procedure explains scaling method: First, two horizontal components of each earthquake are scaled to their maximum level. It means that maximum acceleration should be equaled to gravity acceleration g. Then, acceleration response spectrum of each one of the scaled horizontal components has been determined with considering 5%damping ratio. Next, the SRSS of the two response spectrums for each ground motion will constructed and afterward, the area of these constructed spectrums for all of the earthquakes and also site-specific spectrum curve in a period range of 0.2T seconds to 1.5T seconds is calculated. Finally the acceleration time histories of all earthquakes are multiplied in a corresponding ratio (specific spectrum amounts to constructed spectrum amounts related to the type of soil (in the present study the soil type III has been used)). It is worthwhile noting that, this coefficient should be chosen in a way that none of the average amounts fall below 1.4 times of their correspond amounts in standard spectrum. Now the produced records can be multiplied in any base acceleration which in this study 0.35 has been applied.

Table 1.List of ground motion used.

No.	Earthquake name	Date	Magnitude	Station name	Component(deg)	PGA(g)
1	**Tabas, Iran**	9/16/1978	Ms(7.4)	9101 Tabas	LN TR	0.836 0.852
2	**Kobe**	1/16/1995	M(6.9)	0 KJMA	0 90	0.821 0.599
3	**Chichi, Taiwan**	9/20/1999	Ms(7.6)	CHY080	N W	0.902 0.968
4	**Lomaprieta**	10/18/1989	Ms(7.1)	14 WAHO	0 90	0.370 0.638
5	**Northridge**	1/17/1994	Ms(6.7)	24436 Tarzana, Cedar Hill	90 360	1.779 0.990
6	**Landers**	6/28/1992	Ms(7.4)	24 Lucerne	0 275	0.785 0.721
7	**Duzce, Turkey**	11/12/1999	Ms(7.3)	Bolu	0 90	0.728 0.822
8	**Gazli, USSR**	5/17/1976	Ms(7.3)	9201 Karakyr	0 90	0.608 0.718
9	**Friuli, Italy**	9/15/1976	Ms(5.7)	8014 ForgariaCornino	0 270	0.260 0.212
10	**Bam, Iran**	12/26/2003	Ms(6.7)	Bam	L1 T3	0.745 0.542
11	**Taft, Kern County**	7/21/1952	Ms(7.7)	1095 Taft Lincoln School	021 111	0.156 0.178
12	**Manjil-Abbar, Iran**	6/20/1990	Ms(7.7)	Ab-bar	L T	0.418 0.394

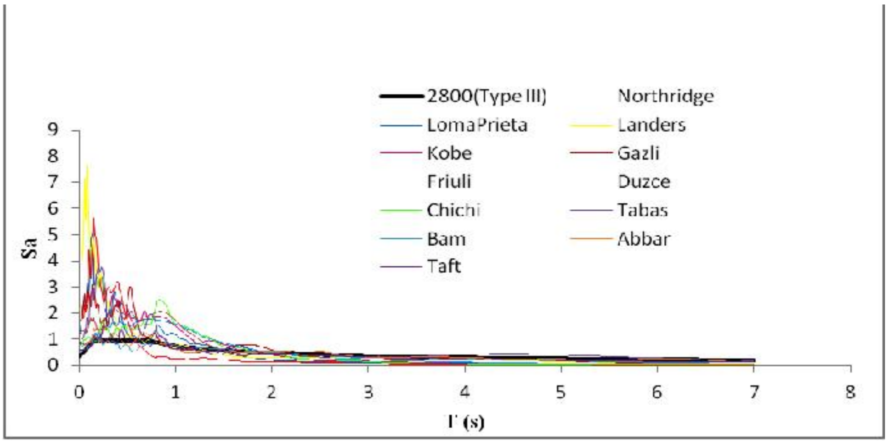


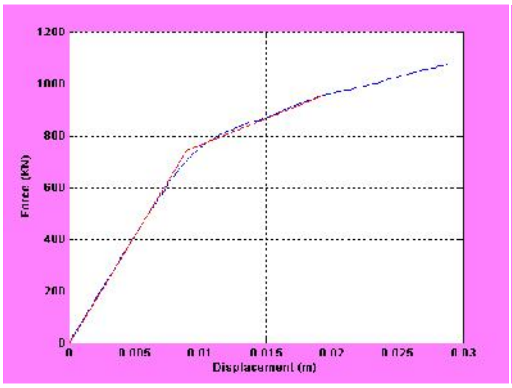
Figure 4.Acceleration response spectrums for 0.35g hazard level

NONLINEAR STATIC AND DINAMIC ANALYSIS

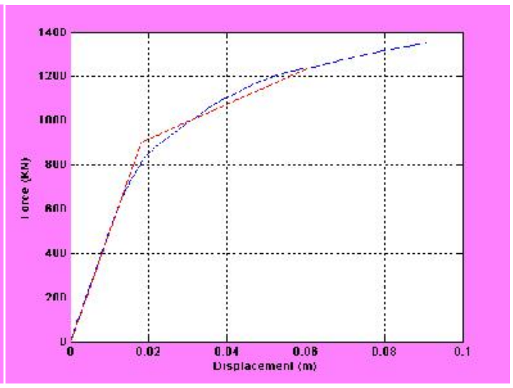
Nonlinear static analysis has been done by the use of displacement coefficients procedure in OPENSEES software. Lateral load distribution is the distribution proportional to the lateral forces caused by linear dynamic analysis. Therefore the number of vibration modes should be chosen in a way that at least 90% of the total structural mass participates in the analysis. Also, steel and concrete behavior modeling has been done with the help of HYSTERETIC MATERIAL and REINFORCING STEEL MATERIAL, without considering the deteriorating parameters. Nonlinear dynamic analyses have also been done by OPENSEES software. At this stage in addition to the mentioned behavioral models as a non-deteriorating pattern, CONCRETE 02 and REINFORCING STEEL MATERIAL have been used for the sake of considering deterioration. Stresses and strains of each element cross section, considering the effects of confinement, have been calculated by USC-RC software. For estimating the accurate amounts of target displacements, δ_t, the derived force-displacement curve(capacity curve) from the analysis should be idealized in bilinear relationship. Achieving this goal, a computer program was developed in MATLAB environment which its derived parameters have been presented in Table 2. The produced capacity curves for each one of the frames are also shown in Figs. 5(a)-(f). Since the effects of deterioration behavior in nonlinear static analyses and calculation of target displacements which represents the maximum displacements of the structure, have been considered by coefficient C_2; so maximum displacement has been calculated in nonlinear dynamic analyses to be compared with the corresponding amounts. Maximum amounts of displacements from nonlinear dynamic analyses, considering ground motions and deferent behavioral models are shown in Table 3 and at the end of each column average amounts of the column have been calculated. Moreover in Table 4 the correlation between the maximum displacement derived from nonlinear dynamic analyses and target displacements is shown, and also in Fig. 6, the derived C_2 coefficient of the analysis has been compared with the recommended amounts in FEMA356.

Table 2.Derived parameters from MATLAB.

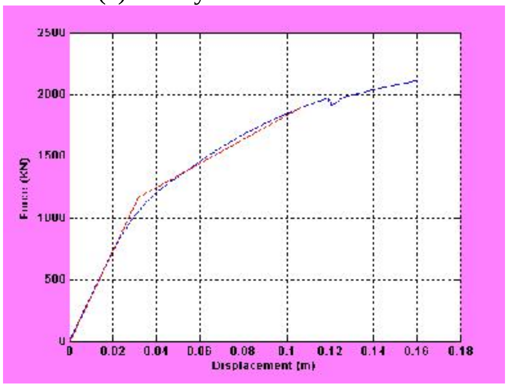
Frame	period T (s)	Effective period T_e (s)	Target displacementδ_t(mm)	Final Target displacement δ_{tfinal}(mm)	δ_y (mm)	Strength ratio (R)	δ_{tf}/δ_t	δ_{tf}/δ_y
3-story	0.251	0.252	20.72	20.88	8.80	1.48	1.008	2.373
5-story	0.460	0.462	60.65	61.20	18.10	2.05	1.009	3.381
7-story	0.660	0.663	107.83	108.94	31.90	2.22	1.010	3.415
10-story	0.996	1.006	187.66	190.15	62.80	1.95	1.013	3.028
15-story	1.954	1.990	460.90	472.25	178.40	1.19	1.025	2.647
20-story	2.676	2.714	700.95	714.25	269.80	1.11	1.019	2.647



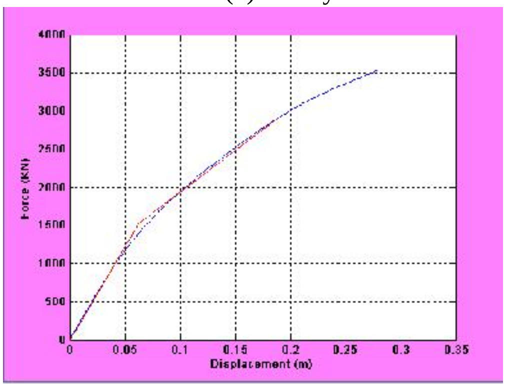
(a) 3 story frame



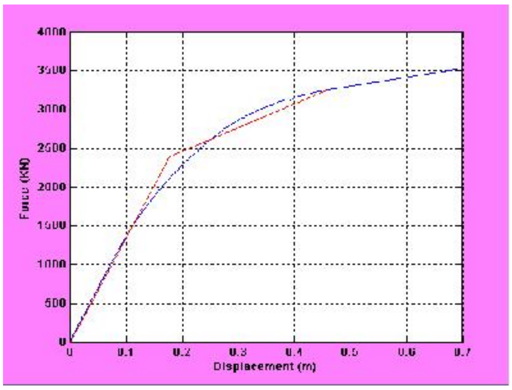
(b) 5 story frame



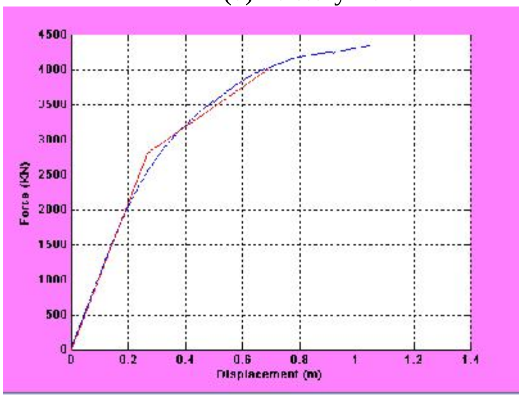
(c) 7 story frame



(d) 10 story frame



(e) 15 story frame



(f) 20 story frame

Figure5.Capacity curve and idealized bilinear relationship for studiedframes.

Table 3.Maximum displacement amounts from nonlinear dynamic analyses.

Ground Motion	3-Story Frame		5-Story Frame		7-Story Frame	
	Deterioration Behavior	Non-deterioration Behavior	Deterioration Behavior	Non-deterioration Behavior	Deterioration Behavior	Non-deterioration Behavior
Tabas	20.9413	18.0512	46.8936	30.2271	111.1640	95.1013
Kobe	36.3708	18.4009	123.8626	87.6746	225.4266	166.7653
Chichi	25.7838	12.0714	116.4460	62.6750	217.6920	150.3926
Lomaprieta	37.4026	21.1408	62.6080	30.0113	93.3158	64.1537
Northridge	23.8064	8.6906	113.5925	64.5280	167.6083	156.8658
Landers	26.5319	18.2408	39.0790	25.6050	53.6014	36.9928
Duzce	39.3509	22.0246	151.5970	125.0205	208.2520	174.9185
Gazli	21.8257	15.6834	78.5080	63.2494	97.4653	68.7565
Friuli	35.0061	34.6906	70.6319	55.0278	68.6517	78.4271
Bam	12.7360	9.9388	31.2744	24.3076	37.7469	31.0032
Taft	14.6908	7.5662	51.2118	36.4445	85.0771	58.3133
Abbar	11.9528	11.2241	50.5306	35.7546	101.6380	72.1204
Average	25.5333	16.4770	78.0196	53.3771	122.3033	96.1509

Table 3.Maximum displacement amounts from nonlinear dynamic analyses.(Continued)

Ground Motion	10-Story Frame		15-Story Frame		20-Story Frame	
	Deterioration Behavior	Non-deterioration Behavior	Deterioration Behavior	Non-deterioration Behavior	Deterioration Behavior	Non-deterioration Behavior
Tabas	126.4660	111.3333	166.2101	113.9600	254.0264	167.8133
Kobe	328.8453	294.2340	275.0555	278.1730	246.2336	301.0051
Chichi	303.6180	274.1175	344.3834	366.0740	306.5289	339.2744
Lomaprieta	209.7390	184.5128	169.8619	169.0600	128.5028	153.0918
Northridge	317.7734	280.7354	342.3194	343.1366	307.2130	345.3311
Landers	90.4403	67.3285	108.9189	119.0011	151.7210	113.3680
Duzce	306.4280	291.8525	205.3460	214.5210	232.0650	211.0920
Gazli	109.3740	107.5710	369.7590	351.8210	330.6140	372.3360
Friuli	54.4257	66.7912	87.4108	90.4181	112.9570	96.4586
Bam	156.6460	89.3527	310.0020	249.1570	404.4060	302.9556
Taft	139.9760	106.1740	268.7000	189.5260	382.7666	378.5060
Abbar	154.6160	134.5224	280.6230	193.8078	525.0030	471.0060
Average	191.5290	167.3771	244.0492	223.2213	281.8364	271.0198

Table 4.Calculation of coefficient C_2

Structure	Calculation of C_2 According to Nonlinear Dynamic Analyses responses	Calculation of C_2 According to Pushover Analyses responses
3-Story Frame	1.550	1.223
5-Story Frame	1.462	1.275
7-Story Frame	1.272	1.123
10-Story Frame	1.144	1.007
15-Story Frame	1.093	0.517
20-Story Frame	1.040	0.395

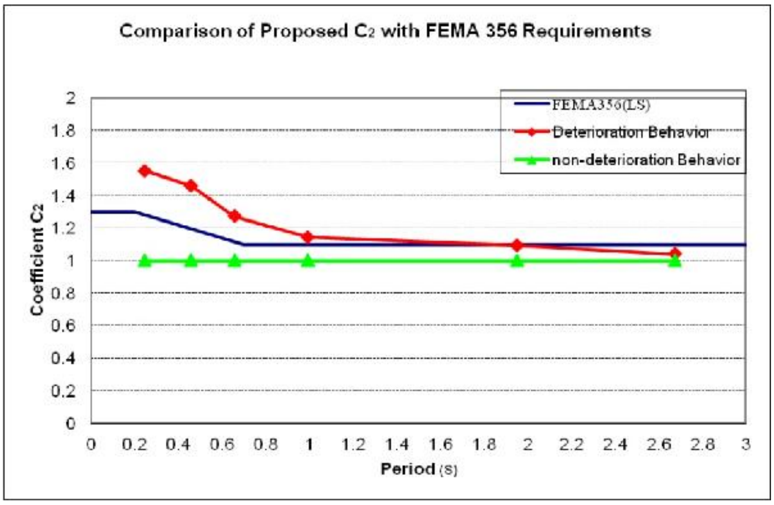


Figure6.Comparison of proposed C_2 by FEMA 356 requirements

CONCLUSIONS

- Findings of the analysis reveal that, as the structural period increases, the amount of c_2 decreases, so that in higher periods, c_2 could equal 1 and could even be ignored in target displacement calculation. Generally, the result expresses that in FEMA 356 and Publication No. 360, for periods of less than 1 sec., c_2 shows an underestimate and for the ones of more than that, a more conservative amount than what you see in the present paper.
- The ratio of target displacement to yielding point displacement, by the increase of period, has an

ascending trend in frames which their periods are smaller than 0.7 sec., and a descending trend for the other three frames.

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