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Cyclic loading deterioration effect in RC moment frames in pushover analysis

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ABSTRACT: The aim of this research is to investigate effect of hysteresis loops in static nonlinear analysis. One of the inefficiencies of static nonlinear analysis is that nonlinear behavior of structural elements due to cyclic deformations is approximately considered in the analysis, and only one quarter of a full hysteretic loop is considered. For investigating the effect of this inefficiency in analysis results, three intermediate concrete moment frames are selected. These models are selected from regular RC structures. The notified procedures in FEMA-356 and proposed plastic hinges in this guideline are utilized for performing static nonlinear analysis. A coefficient for consideration of stiffness degradation and strength deterioration is proposed by FEMA-356 in nonlinear static analysis. This coefficient for intermediate RC moment frames is equal to unity. For calculation of this coefficient, in this paper, the nonlinear dynamic analysis is used. Clough and Takeda Hysteretic loops and a hysteretic loop that considers effects of severe stiffness degradation, strength deterioration and pinching are assumed in nonlinear dynamic analysis. By comparison of results the value of this coefficient is obtained 25% more than the value proposed by FEMA-356.

1 INTRODUCTION

Since inelastic behavior is intended in most structures subjected to infrequent earthquake loading, the use of nonlinear analyses is essential to capture behavior of structures under seismic effects. A set of earthquake records which have been selected accurately, can give a detailed evaluation of expected behavior and seismic performance of structures. With accepting this fact that the suitability and accuracy of calculation instruments are being increased, but there are some restriction for using nonlinear dynamic analysis. Some of these restrictions are the difficulty of interpreting the results and sensitivity of results to selection of earthquake records so that this problems, make using this kind of analysis very difficult for practical purposes. Therefore due to its simplicity, the structural engineering profession has been using the nonlinear static procedure (NSP) or pushover analysis, described in FEMA-356 (FEMA 2000) and ATC-40 (ATC 1996). It is widely accepted that, when pushover analysis is used carefully, it provides useful information that cannot be obtained by linear static or dynamic analysis procedures. In performance assessment and design verification of building structures, approximate nonlinear static procedures (NSPs) are becoming commonplace in engineering practice to estimate seismic demands. In fact, some seismic codes have begun to include them to aid in performance assessment of structural systems (IIEES 2002). Although seismic demands are best estimated using nonlinear time-history (NTH) analyses, NSPs are frequently used in ordinary engineering applications to avoid the intrinsic complexity and additional computational effort required by the former. As a result, simplified NSPs recommended in ATC-40 (ATC 1996) and FEMA-356 (FEMA 2000) have become popular. These procedures are based on monotonically increasing predefined load patterns until some target displacement is achieved. In the implementation of pushover analysis, modeling is one of the important steps. The model must consider nonlinear behavior of structural elements. Such a model requires the determination of the nonlinear properties of each component in the structure that are quantified by strength and deformation capacities. Lumped plasticity idealization is a commonly used approach in models for deformation capacity estimates. The real plastic behavior of a member during an earthquake is a cyclic behavior and after one cycle of loading and unloading, properties of the curve such as strength and stiffness is changed. Because in the nonlinear static analysis the applied load is not cyclic and is an incremental load and it is in the specified direction, therefore some kind of modeling is needed, that can give the appropriate estimation of cyclic behavior of member. Also it should consider the effects of strength deterioration and stiffness degradation. For achieving this purpose, as shown in Figure 1, the backbone curve of actual hysteretic behavior of a member is used and it has been idealized to curve shown in Figure 2. The specifications of these idealized curves have been explained in some standards and pre-standards like FEMA356 (FEMA 2000).

After performing the analysis, it is needed to evaluate the demand of structure. This evaluation can be done by using capacity spectrum method or constant coefficients method as described in ATC40 (ATC 1996) and FEMA356 (FEMA 2000) respectively. In the second method (constant coefficients) the demand of structure is defined by target displacement parameter. For calculating this parameter some coefficients are used and one of these coefficients is for taking into account the amount of strength deterioration and stiffness degradation. This coefficient in FEMA356 (FEMA 2000) for intermediate RC frames is equal to one. But this coefficient can be different depend on the members specifications and their hysteretic behavior. In this paper, it has been tried to calculate amount of effect of hysteresis loops and strength deterioration and stiffness degradation in estimating target displacement by using nonlinear dynamic analysis with different hysteresis loops for members and nonlinear static method with standard plastic hinges.

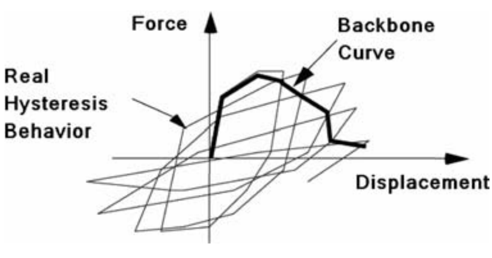

Figure 1. Backbone curve of hysteresis behavior.

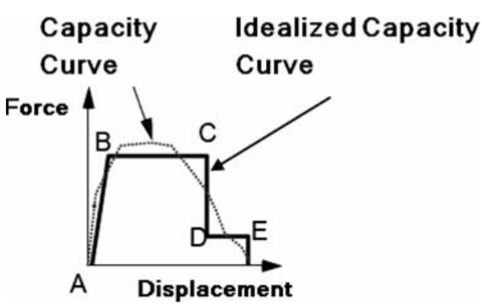

Figure 2. Idealized capacity curve.

For performing nonlinear dynamic analyses computer program, IDARC (Kunnath et al. 1992) has been used. Also for performing nonlinear static analyses computer program ETABS (Computers & Structures 2005) has been used.

2 NONLINEAR STATIC METHOD

Nonlinear static analysis is based on a principle which says structure's response can be simulated with a system which has one degree of freedom with equivalent characteristics. Based on this theory, structure's response is just related to first vibration mode and its shape doesn't change during analysis period. Although both of these theories seems to be wrong, but accurate estimation from system's maximum reflection can be obtained for those structures which their first vibration mode is dominant. The aim of nonlinear static analysis is to evaluate expected behaviors of structural system by estimating the resistance and displacement demand under designed earthquake and comparing the requirements with existing capacities in selected performance level. Nonlinear static analysis (pushover) is a method to estimate the force and displacement demand which it simply does load redistribution for internal forces in members which tolerate more than their elastic limited forces. Specifying force-displacement curve (capacity curve) is one of its most important results. Amount of base shear against lateral displacement of the reference point which is in the roof, are used to draw the curve. It's used for specifying the target displacement.

2.1 *Lateral load distribution*

Using adequate load distribution figure in evaluation of behavior of a building is one of important steps. In fact, the shape of lateral loading presents the distribution of inertia forces in designed earthquake. It is clear that lateral forces change because of intensity of earthquake and duration of earth's excitation time. Contribution of inertia forces during earthquake is constant and maximum displacement can be compared with what is occurred in designed earthquake as if one loading shape is used. These assumptions are sometimes accurate or not. They are accurate when structure's responses are not mostly affected by upper vibration modes and it has only one kind of yielding mechanism.

Some case of lateral loading shapes gives accurate approximates for displacement demands. Using two kind of lateral loading shapes are recommended by researchers in some situations. Applying uniform loads which floor's forces are proportionate with its mass and the shape of load contribution similar to different modes is widely accepted by them.

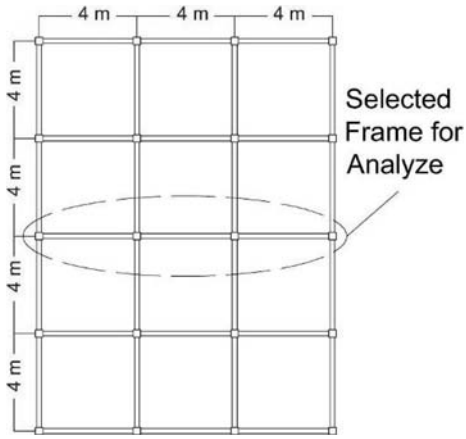

Figure 3. Building plan and selected frame for analyze.

3 DESCRIPTION OF SAMPLES

Three RC frames with 4, 8 and 12 floors are used for this research. These frames have been designed based on intermediate ductility according ACI318-05. (ACI 2005) They are in high seismic risk areas with PGA = 0.35 g (BHRC 2005) and soil is in type II, similar to group C in FEMA-356. Structures' plan and the separated frame are shown in Figure 3. It is supposed that concrete compressive strength is 24 Mpa and steel's yield stress is 400 Mpa. Floor's dead load is 700 Kg/m^2 and live load is 200 Kg/m^2. Live load participating percentage in earthquake force calculation is 20% for residential using (BHRC 2005).

4 HYSTERETIC BEHAVIOR MODELING

Hysteresis model which is used for showing nonlinear behavior of members is in company with three parametric models for mentioning stiffness degradation, strength deterioration and pinching (Park et al. 1987). Push curve of force-displacement is shown as a trifurcation curve in Figure 4. As it is shown, points consist of cracking point, yielding point and ultimate strength point. For showing the effect of stiffness degradation, strength deterioration and pinching behavior, α, β and γ parameters are used. Parameter's concepts and their effects have been shown in Figure 5. Stiffness degradation which is represented by α is introduced by selecting a common point on protraction of first skeletal curve which is aiming the unloading lines to the horizontal axis. This parameter specified the stiffness degradation degree and mentioning the reduction of confined area of hysteresis loops. Strength deterioration is represented by β, which shows reduction rate of strength. This parameter is defined in a ratio of

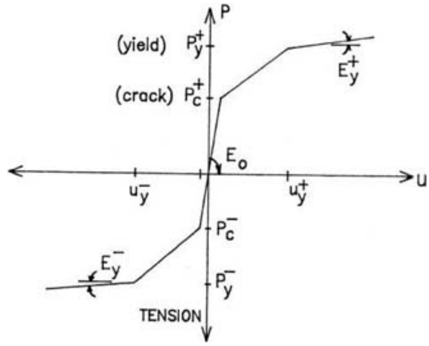

Figure 4. Trilinear curve (Park et al. 1987).

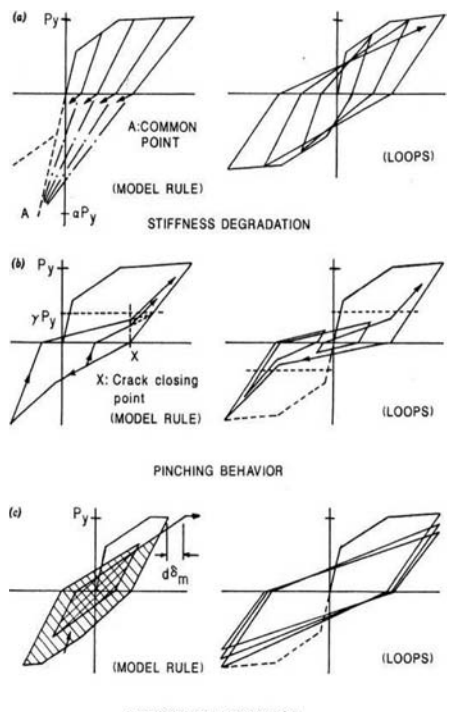

Figure 5. Definition of parameters in 3 parameters hysteresis loops (Park et al. 1987).

addition of maximum response damage, $d\delta_m/\delta_u$, and addition of normalized hysteresis energy, $dE/\delta_u P_y$.

$$\beta = \frac{d\delta_m/\delta_u}{dE/(\delta_u P_y)} = \frac{d\delta_m}{dE/P_y} \tag{1}$$

pinching behavior is represented by γ. According to Figure 5, by lowering the maximum point to the p_y

level, γ is defined in protraction of previous unloading line. Renewed loading lines aim the new target point to reach the displacement of crack closure. Pinching behavior causes the reduction of hysteresis loop's area and reduction of energy loss indirectly. By using three parametric models, hysteresis models can be recycled such as, Clough (Clough et al. 1965) which doesn't contain strength and stiffness reduction or Takeda (Takeda et al. 1970) that contains the effects of stiffness and strength reduction. Three hysteresis loops have been used in this research. Clough, Takeda and a model which its stiffness degradation, strength deterioration and pinching behavior parameters are maximum amount of normal time, have been used.

5 INPUT RECORDS

Seven earthquake records have been used for nonlinear dynamic analysis which the distance between them and the fault is 10–24 km. They are selected from soil type group (II) (BHRC 2005), which the effect of closeness to the fault and soft soil don't exist. For using these records, their specification should be compatible with site. So they have to be scaled in a way with their responses are mostly compatible with real earthquakes which is occurred in the site.

5.1 *Scaling earthquake records*

For comparing effect of deferent earthquake records on structure, they should be scaled by a unique criterion. Earthquake records which used in this research have been scaled to 0.35 g. The method of scaling record is describing in this section. All records have been scaled to their maximum PGA. Therefore their PGA will be equal to 1 g. The response spectrum of each scaled record has been specified with considering 5% damping ratio. The area under response spectrum curve between periods 0.1 and 3 second has been calculated and compared with the same area in the standard reflection curve (S_a/PGA) in Iranian code. The records have been multiplied to ratio of two mentioned areas. Finally all records multiplied to 0.35 as a PGA of site. The records specifications are presented in Table 1.

Table 1. Specifications of used records.

Number	Records' name	Year	Real PGA (g)	Scaled PGA (g)
1	KOBE	1995	0.821	0.446
2	KOCAELI, TURKEY	1999	0.376	0.57
3	LOMA PRIETA	1989	0.479	0.517
4	NORTHRIDGE	1994	0.514	0.45
5	N. PALM SPRINGS	1985	0.594	0.578
6	CAPEMENDOCINO	1992	0.385	0.709
7	SUPERSTITI HILLS	1987	0.377	0.45

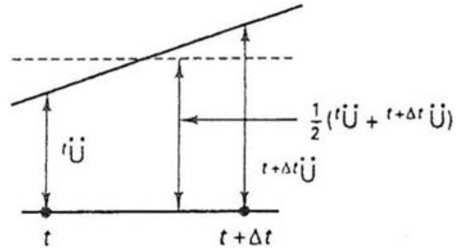

Figure 6. Newmark method (Bathel 1996).

6 NONLINEAR DYNAMIC ANALYSIS

6.1 *Newmark method for dynamic analysis*

The Newmark integration scheme can also be understood to be an extension of the linear acceleration method. The following assumptions are used (Bathel 1996):

$$\dot{U}_{t+\Delta t} = \dot{U}_t + [(1-\delta)\ddot{U}_t + \delta\ddot{U}_{t+\Delta t}]\Delta t \tag{2}$$

$$U_{t+\Delta t} = U_t + \dot{U}_t\Delta t + \left[\left(\frac{1}{2}-\alpha\right)\ddot{U}_t + \alpha\ddot{U}_{t+\Delta t}\right]\Delta t^2 \tag{3}$$

where α and δ are parameters that can be determined to obtain integration accuracy and stability. Newmark originally proposed as an unconditionally stable scheme the constant average acceleration method (also called trapezoidal rule), in which case $\delta = 1/2$ and $\alpha = 1/4$ (Figure 6).

6.2 *Selected method for dynamic analysis*

Inelastic nonlinear dynamic analysis of RC moment frames has been done by computer program, IDARC (Kunnath et al. 1992). Moment-curvature curves have been used for presenting the nonlinear behavior of members which made by using concrete and steel stress-strain curves, member's dimension, and reinforcement and confinement limitation. Newmark method is also used for analyzing, and hysteresis models, which mentioned before, are used as the members' hysteresis behavior. Nonlinear dynamic analysis and nonlinear static analysis results are shown in Table 2.

Table 2. Nonlinear static and dynamic results.

	4 Story Frame			8 Story Frame			12 Story Frame		
Records' Name	Clough	Takeda	Severe	Clough	Takeda	Severe	Clough	Takeda	Severe
Kobe	201.1	258.3	227.9	173.2	284.4	516.5	262.6	323.1	736.8
Kocaeli	167.9	262.1	336.7	400.4	462.9	528.3	554.0	538.4	843.8
Lomap	173.4	224.3	163.7	231.4	256.1	223.2	178.6	208.1	247.9
Northridge	185.3	272.7	276.4	296.9	450.5	443.5	333.0	353.3	376.2
Palm Springs	244.7	342.1	361.9	298.7	352.9	373.6	355.2	364.0	439.0
Capemendocino	143.3	157.1	218.0	180.5	243.9	259.3	220.5	324.8	435.6
Superestiti hills	200.4	277.6	Fail	288.1	358.0	402.8	355.6	392.5	721.4
Average of Dynamic Analyses	188.0	256.3	264.1	267.0	344.1	392.5	322.8	357.7	543.0
Static Analyses with First Mode Loading Shape	210.58			460.53			659.17		
Static Analyses with Uniform Loading Shape	202.16			399.77			627.53		
C_2	1	1.363	1.405	1	1.289	1.47	1	1.108	1.682

7 NONLINEAR STATIC ANALYSIS

Nonlinear static analysis has been used based on displacement which is mentioned in FEMA-356. (FEMA 2000) In this method structure's nonlinear model is pushing by a specified loading pattern until reaching specified displacement called target displacement. Deformations and internal forces are calculated and compared with allowable amounts, after achieving to the target displacement. Target displacement which is the maximum displacement that structures can experience in earthquake loading is calculated by spectrum value in effective period of structure and applying special factors.

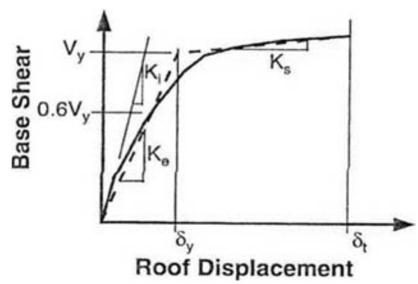

Figure 7. Simplified force-displacement curve (FEMA 2000).

7.1 *Calculating major effective period*

The major effective period can be obtained by bilinear model:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \tag{4}$$

where T_i is the base period of structure with linear behavior assumption and K_i is lateral elastic stiffness as shown in Figure 7.

7.2 *Calculating target displacement*

In this method target displacement is calculated based on Equation 5.

$$\delta_t = C_0C_1C_2C_3S_a\frac{T_e^2}{4\pi^2}g \tag{5}$$

where, T_e is the effective period in the considered direction. S_a is the amount of site spectrum in effective period with specified damping. C_0 is modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system. In this research it is considered to be equal to 1.2. C_1 is used as a modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response and it is equal to 1 when $T_e > T_0$. Therefore in this research it is considered to be equal to 1. C_2 is a modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. Value of C_2 is different for different framing systems and structural performance levels. But it is equal to one for intermediate moment frames in all performance levels. For some structures which they don't have a stable and complete hysteresis loops, cyclic motions can cause to extension of damage and not only lead to strength degradation but also it cause to increase deformations and decrease strength. Because this factor is assumed to be 1 in FEMA-356 (FEMA 2000), in this research with different hysteresis loops

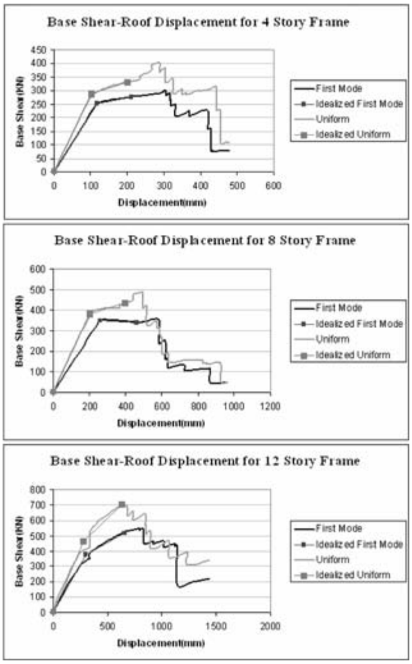

Figure 8. Capacity curves obtained from nonlinear static analysis.

and by using nonlinear dynamic analysis and comparing it with nonlinear static analysis, this factor is calculated. C_3 is a modification factor to represent increased displacement due to dynamic P-Δ effects. Because studied structures have a positive post-yield stiffness, this factor is assumed to be equal to 1.

Two kind of loading pattern are used for doing nonlinear static analysis based on FEMA-356 (FEMA 2000): 1- according to the shape of first vibration mode 2- loading pattern which is in proportion with floors mass. After performing analyses and by using capacity curves and idealizing them to bilinear curves, the target displacement has been calculated. Capacity curves of structures are shown in Figure 8. By calculating target displacement for each structure, it is possible to equivalent the nonlinear static analysis results with nonlinear dynamic analysis results using hysteresis model without stiffness degradation and strength deterioration (C_2 is 1), so C_2 can be defined by calculating the ratio of nonlinear dynamic analysis of the models which have stiffness degradation and strength deterioration to the first model (which doesn't have any degradation and deterioration). Results are shown in Table 1.

Results are separately shown for Clough, Takeda, a model with high stiffness degradation and strength deterioration, and nonlinear static analysis. The results have been calculated and averaging has been done for calculating C_2. C_2 is more than 1 in all the conditions and it is 25.3% more than 1 in Takeda model. If the structure has incomplete hysteresis loops, high stiffness degradation and strength deterioration and pinching, this factor is 51.9% more.

8 CONCLUSIONS

Nonlinear static analysis is an efficient method for seismic prediction of structure which is widely used based on displacement. It doesn't consider member's nonlinear behavior changes which are the results of cyclic behaviors, so by considering these effects in a factor this problem can be approximately solved. C_2 is the factor for calculating target displacement. So for the structures which their deformation is medium and the stiffness degradation and strength deterioration is normal, the factor for Takeda model accrued and it is 25.3% more than 1. If it's deformation, stiffness degradation, strength deterioration and pinching be so high, the factor will be 51.9% more than 1.

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