COMPARISON OF PERFORMANCE EVALUATION CONCEPT FOR RC BUILDINGS CONSIDERING TURKISH EARTHQUAKE CODE AND THE PROPOSED EMPIRICAL RELATION

Ü. Mert-Tugsal[1], B. Taskin[2] and F.I. Kara[3]

[1] *Res.Asst., Dept.of Civil Engineering, Istanbul Technical University, Maslak, Istanbul, 34469, Turkey. utugsal@ins.itu.edu.tr*
[2] *Asst.Prof., Dept.of Civil Engineering, Istanbul Technical University, Maslak, Istanbul, 34469, Turkey. btaskin@ins.itu.edu.tr*
[3] *Res.Asst., Dept.of Sciences of Earthquake & Construction, Gebze Institute of Technology, Kocaeli, Turkey. figokce@gyte.edu.tr*

ABSTRACT :

This paper shows the adequacy of proposed empirical relations for estimating the inelastic deformation demands of existing structures and the performance displacement obtained by applying nonlinear static procedures as defined in the Turkish Earthquake Resistant Design Code-2007. As widely preferred by engineering applications currently, structural performances of existing reinforced-concrete (RC) buildings are determined by computations based on nonlinear static analysis, in other words pushover analysis, as defined in many codes and standards. Since the application of nonlinear static procedures are limited depending on the number of stories, mass participation ratio, vertical or in-plan irregularities, recent studies are concentrated on establishing relationships for estimating inelastic deformation ratios and seismic capacity-demand index relationships, considering an equivalent single-degree-of-freedom (SDOF) system and realizing nonlinear dynamic analysis. Recently, our research team proposed an empirical relation for estimating the inelastic deformation demand, C_R, of existing structures, considering the stiffness-degrading nonlinear behavior of structural elements. For this purpose, sets of Turkish recorded earthquake motions are selected and nonlinear dynamic analyses are performed for equivalent SDOF systems having a wide period (T_n) range, consistent with other researchers' work, are taken into account. Assuming 8 different levels of yield strength reduction factor, R_y, and five strain hardening, α, a number of 214,400 runs of analysis is realized and an empirical expression is established employing T_n; α and R_y. Finally, 8 existing RC buildings are investigated in details and later performance displacements are calculated by applying the nonlinear static procedure as in TERDC-2007. Results are successfully compared with the inelastic displacement demands computed by the proposed empirical equation.

KEYWORDS: Reinforced-concrete, Performance evaluation, Existing buildings, Nonlinear dynamic analysis.

1. INTRODUCTION

Structural performance of existing buildings are currently determined by applying nonlinear static procedures defined in most earthquake design codes and standards; such as ATC-40 (1996), FEMA273 (1997), FEMA356 (2000) and TERDC (2007). In standards and engineering practice, nonlinear static procedures are preferred rather than nonlinear dynamic analysis of structures depending on the lack of appropriate and practical software and the amount of time required. However, recent studies are concentrated on establishing seismic capacity-demand index relationships and furthermore as an alternative to capacity-demand index relationships, there is a tendency among the researchers to use nonlinear dynamic time-history procedures as a part of a performance-based design approach, (Farrow and Kurama, 2003). More recently, Goel and Chopra (2004) developed a modal pushover analysis procedure, where the target displacement is determined from nonlinear dynamic analysis of an equivalent single-degree-of-freedom (SDOF) inelastic system and its peak deformation.

Inelastic deformation ratio of an equivalent SDOF system, as well known, is a function of natural vibration period, T_n, ductility factor, μ, and yield-strength reduction factor, R_y. Considering the attractiveness of bilinear model's rather simpler application and less time required for computing, many researchers conducted research using different structural characteristics; to name a few, yield strength, ductility, strain-hardening ratio, structural safety levels, *etc.* within a period range under the effect of recorded strong motions, (Song and Pincheria, 2000; Ruiz-Garcia and Miranda, 2002). Recently, Chintanapakdee and Chopra (2003), widened their research on SDOF bilinear-systems considering 260 ground motions, for constant-ductility and constant-yield strength systems using five different strain hardening ratios, for a wide period range of T_n=0.005 to 100 seconds. They showed that, independent of the force-deformation relation, numerical values of the inelastic deformation ratios have limits at very short and very long periods and can be expressed as functions of α and μ or R_y. They concluded with two equations for estimating the inelastic deformation demands for bilinear structures with known strength or known ductility.

More recently, Taskin *et al.* (2008) proposed empirical relations for estimating the inelastic roof displacement demands, in other words the "performance displacement", of existing RC structures, considering stiffness degradation during the nonlinear behavior of structural members using a set of recorded strong motions from Turkish earthquakes, mostly in the North Anatolian Fault Zone. Assuming a constant damping ratio of ξ=5% and realizing 214,400 runs of non-linear dynamic analyses for a number of 134 different periods within the same range of Chintanapakdee and Chopra, equivalent SDOF stiffness–degrading systems having yield strength levels of R_y= 1, 1.5, 2, 3, 4, 5, 6, and 8; and five different post-yield stiffness ratios of α=0%, 1%, 3%, 5% and 10% are computed for obtaining inelastic deformation demands, C_R. Finally, an easily applicable empirical equation for calculating the C_R is established by regression analysis and proposed for existing RC buildings, for which vibration period, strain hardening and yield-strength ratios can simply be calculated.

Having the aim of illustrating the consistency of the proposed equation, a number of 8 RC buildings are experimentally investigated in details by means of structural materials, reinforcement scheme and current damage state. After computer modeling of the structures, nonlinear static pushover analyses are realized as in TERCD-2007 and inelastic displacement demands are computed. Finally, these demands are successfully compared with the inelastic displacements calculated by the proposed expression.

2. THEORETICAL BASIS FOR THE EMPIRICAL EXPRESSION

2.1. Properties of the Earthquake Ensemble

More than 500 strong motion records from the database of Turkish General Directorate of Disaster Affairs-Earthquake Research Center's are inspected and later an ensemble is established from 40 strong motions of large magnitude (M>6.0) earthquakes representing 0.40g design zone, (Taskin *et al.*, 2007). Following Figure 1 shows the comparison of the acceleration response spectra for each filtered strong motion with design spectra delineated for the $Z1$ (stiff) and $Z4$ (poor) local site types in TERDC-2007 for a damping ratio of ξ=5%.

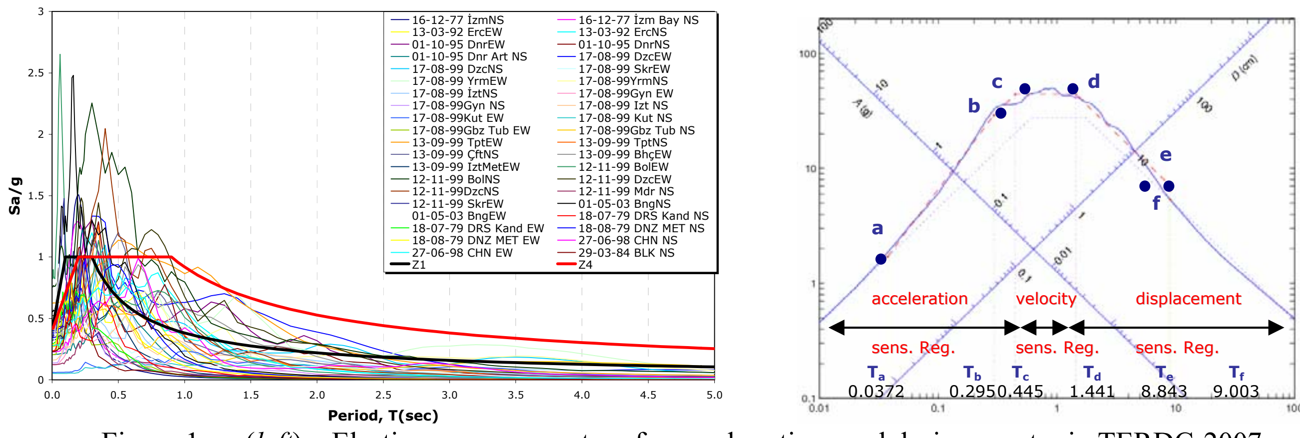

Figure 1 (*left*) Elastic response spectra of ground motions and design spectra in TERDC-2007
(*right*) Tripartite elastic response spectra of the earthquake ensemble

Mean tripartite elastic response spectrum, as well as the corresponding acceleration, velocity and displacement sensitive spectral regions and separating periods are also shown in the same figure.

2.2. Nonlinear Dynamic Analysis of SDOF Stiffness-Degrading Systems

The equivalent SDOF system of mass m, natural vibration T_n, elastic stiffness k_e, post-yield stiffness of $\alpha \times k_e$ and a damping ratio of ξ is assumed to exhibit stiffness degrading non-linear behavior, as illustrated in Figure 2.

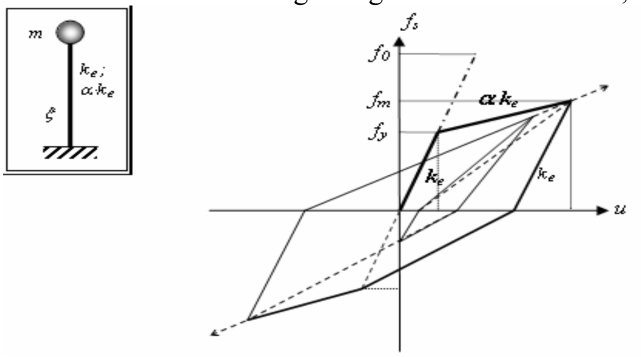

Figure 2 Stiffness degrading hysteretic relation for SDOF systems

Using the above force-deformation relation, following Eqn. 2.1, introducing the yield strength reduction, R_y, can be written, where f_0 and u_0 are the minimum strength and the corresponding displacement for the structure to remain elastic; f_m and u_m are the peak force and peak displacement of the inelastic system and f_y and u_y are the yield strength and the yield displacement, respectively.

$$R_y = \frac{f_0}{f_y} = \frac{u_0}{u_y} \tag{2.1}$$

If the dynamic equation of motion, $\ddot{u} + 2\xi\omega_n\dot{u} + f_s / m = -\ddot{u}_g(t)$, is numerically solved for the inelastic and its corresponding linear SDOF stiffness-degrading system, where $\ddot{u}_g(t)$ is the earthquake acceleration, the peak deformations u_m and u_0 can be obtained. Hence, the inelastic deformation ratio is calculated as below:

$$C_R = \frac{u_m}{u_0} \tag{2.2}$$

As shown by Chopra (2001), very short period or very long period structures' dynamic behaviors become independent of the strong motion. Therefore, these limiting values have a theoretical importance and analysis results should satisfy this issue. First limiting case, which is T_n tends to zero, is taken into account by selecting the first structure with T_n=0.005s; and for the second case, which is T_n tends to infinity a structure with T_n=100s is considered. The rest 132 SDOF systems are selected between these two limiting vibration periods. Using the hysteretic behavior in Figure 2 and substituting it into Eqn. 2.1, the inelastic deformation ratio C_R can be written for the first limiting case as follows:

$$L_R = \left(\frac{u_m}{u_0}\right)_{T_n \approx 0} = \frac{1}{R_y}\left(1 + \frac{R_y - 1}{\alpha}\right) \tag{2.3}$$

L_R is used to define C_R for $T_n \approx 0$. For the limiting case of $T_n \approx \infty$, the system is so flexible that the peak deformation of the system will be equal to the peak deformation of the ground, well known as "equal-displacement" rule (Veletsos and Newmark, 1960). Consequently, for this limiting case, $u_m = u_0 = u_g$ and the inelastic deformation ratio will be $C_R \approx 1$.

2.3. Nonlinear Static Analysis Procedure in the TERDC-2007

Nonlinear static procedures based on pushover analysis are widely accepted and enforced evaluation methods since they practically let engineers to gain insight to nonlinear seismic behavior of structures. Inelastic variation of the base shear with respect to the top-story displacement, in other words the *pushover-curve*, is obtained considering monotonic increments in adaptive load patterns, such as the equivalent seismic loads, first mode shape, *etc*. Then, inelastic demand spectrum for the structure and the capacity spectrum, which is transformed from the pushover curve, are compared and inelastic demands are obtained from intersection of the two curves. According to the procedure of TERDC-2007, capacity curve is established from the pushover curve by transforming the coordinates into modal displacement d_1 and modal acceleration a_1 as:

$$d_1^{(i)} = \frac{U_{yN1}^{(i)}}{\Phi_{yN1}\Gamma_{y1}} \qquad\qquad a_1^{(i)} = \frac{V_{y1}^{(i)}}{M_{y1}} \tag{2.4}$$

Here, i is the pushing step; 1 represents the first mode of the structure; y is the direction of loading; N is the symbol of the top-story; Φ_{yN1} is the modal displacement in the top-story; M_{y1} is the effective modal mass and Γ_{y1} denotes the instantaneous participation factor for an earthquake in y direction. Elastic design spectrum having the axes S_{ae1} and S_{de1} is transformed into inelastic demand spectrum as follows:

$$S_{di1} = C_{R1} S_{de1} = C_{R1} \frac{S_{ae1}}{\left(\omega_1^{(1)}\right)^2} \tag{2.5}$$

C_{R1} in Eqn. 2.5 is the spectral displacement ratio and can be calculated depending on the initial vibration period, $T_1^{(1)}=2\pi/\omega_1^{(1)}$. Finally, inelastic displacement demand of the structure is at roof level is obtained by Eqn. 2.6:

$$U_{yN1}^{(p)} = \Phi_{yN1}\Gamma_{y1} d_1 \tag{2.6}$$

After establishing an empirical relationship for obtaining C_R, U_{yN1} calculated by the above procedure, which is the total displacement at roof level, will be compared with u_m of Eqn. 2.1 as $C_R \times u_0$.

3. EMPIRICAL EXPRESSION FOR INELASTIC DISPLACEMENT RATIO

3.1. Analysis of Equivalent SDOF Stiffness-Degrading Systems

Nonlinear dynamic analyses are performed for the 5% damped equivalent SDOF stiffness-degrading systems, subjected to the earthquake ensemble with periods T_n=0.005 to 100 s; having constant-R_y values of 1, 1.5, 2, 3, 4, 5, 6, and 8 and post-yield stiffness ratios of α=0%, 1%, 3%, 5% and 10%. For each SDOF system, peak nonlinear displacements u_m are computed and inelastic deformation ratios C_R are obtained. For illustrating the effect of different levels of yield strength or different percentages of post-yield stiffness ratio on the inelastic deformation demand, following Figure 3 is plotted, showing the C_R variation along period axis for selected constant values of α=3% (left) or R_y=4 (right).

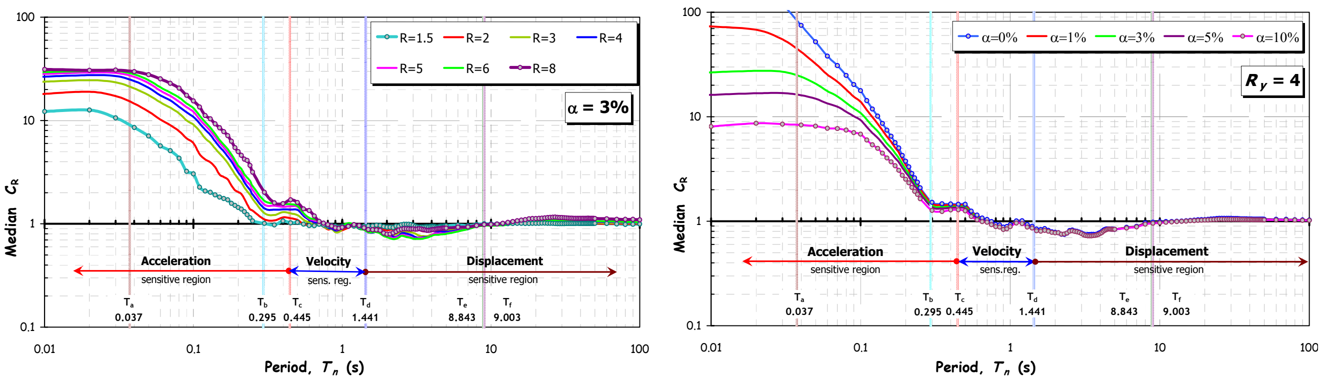

Figure 3 Median C_R values for α=3% (left) and R_y=4 (right)

Figure 3 demonstrates that, although the nonlinear deformation demand increment for the systems in the acceleration sensitive region is very high, it is negligible within the velocity sensitive region and almost independent from any parameter in the displacement sensitive region. This plot also proves the validity of the limiting values for very short and very long period systems. Especially for the low-rise structures with periods T_n=0.1~0.4s the median inelastic deformation ratio C_R, changes dramatically from two to almost seven times when R_y=1.5 and R_y=8 systems are compared. The results presented in Figure 3 indicate that, existence of the strain-hardening reduces the deformation demand when compared to elasto-plastic case (α=0%), almost all through the period range. Figure 4 shows the median dispersion of previous plots similarly for α=3% and R_y=4. For the limiting values of T_n, the dispersion tends to zero due to the independence from the strong motion. When the yield strength reduction factor increases, dispersion also increases except for a small period range for extremely short periods. Ratio of post-yield stiffness does not seem to have a significant effect of C_R dispersion.

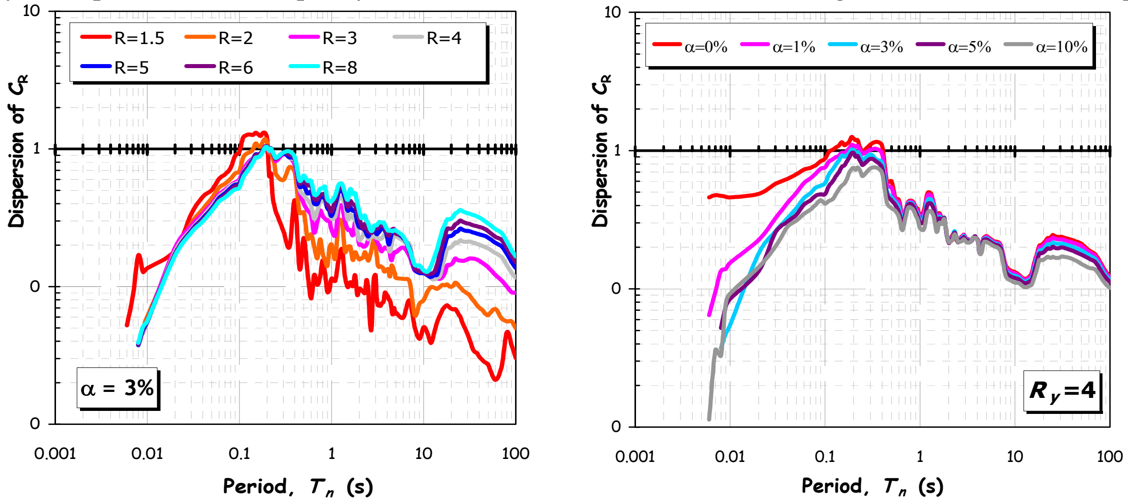

Figure 4 Median dispersion variation of C_R for α=3% (left) and R_y=4 (right)

3.2. *Proposed Empirical Expression for C_R Calculation*

Using the above nonlinear dynamic analysis results for 134 SDOF stiffness degrading systems, with 8 different R_y levels and five different α percentages; it is aimed to establish an empirical expression that will serve for fast calculation of the inelastic deformation ratio, therefore the nonlinear displacement demand of existing RC structures.

Many researchers studied on developing similar relations, mostly for elasto-plastic systems. Recent studies, however, include bilinear systems considering local site effects or hysteretic parameters. Two of the latest promising relationships for estimating the mean inelastic deformation ratio of constant ductility systems are Miranda's (2001) for elasto-plastic systems and most recently Chopra and Chitanapakdee's (2003) formulations for bilinear systems. In this research similar expressions with the latter are aimed to be established, hence its superior advantages of a covering even the near fault effects. Expressions are developed as a function of the normalized period of the structure with respect to the period separating the acceleration and velocity sensitive regions as in Eqn. 3.1.

$$C_R = 1 + \left[(L_R - 1)^{-1} + \left(\frac{a}{R_y^b} + c \right) \left(\frac{T_n}{T_c} \right)^d \right]^{-1} \tag{3.1}$$

The coefficients a, b, c and d of the above regression equation are derived as the sum of the square root of differences between the computed values and the formulation. Herein, it is preferred to select the regression parameters with the highest correlation coefficient, which led the values as a=25.1; b=1.9; c=2.2 and d=2.3 with a 99.8% of correlation. Figure 5 demonstrates the comparison of the proposed empirical expressions and the computed C_R for a selected post-yield stiffness ratio of 3% and yield strength reduction factors of 2, 4 and 6.

4. INVESTIGATED RC BUILDINGS

The employability of the enhanced expression in this study in designating seismic performance of existing buildings is investigated. For this purpose, nonlinear static analyses defined in TERDC-2007, performance displacements of 8 RC buildings having different structural properties that are summarized in Table 4.1, are carried out and these values are compared with the proposed empirical expression. All buildings are investigated in details by means of structural material quality, reinforcement amount and detailing of bars and local site

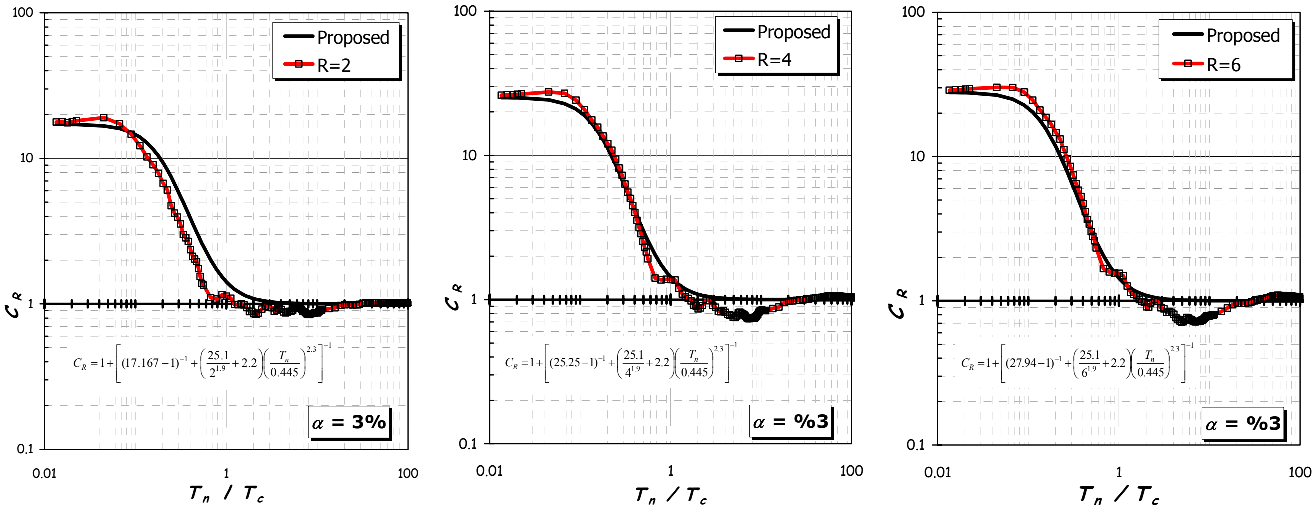

Figure 5 Proposed C_R expression for α=3% and R_y=2 (left); R_y=4 (center); R_y=6 (right)

conditions. Concrete class is found to be varying from C14~C20 (f_{ck}=14~20 MPa), while the reinforcing steel is S220 (f_{yk}=220 MPa) class. Afterwards, all 8 buildings are modeled considering the experimentally determined sectional characteristics and consequently the nonlinear behavior is introduced for each structural element. Nonlinear static pushover analysis is performed as defined in TERDC-2007 and inelastic displacement demand for the roof level U_{yN1} is calculated for each building. Finally, these demand values are compared with the inelastic deformation demand computed by using the proposed empirical expression values. Below Table 4.1 tabulates some characteristics of the buildings, where T_1 is the first vibration period, W is the total weight of the building, $V_{t,0}$ is the elastic seismic force depending on the local site conditions and structural period and R_y is the yield strength ratio. For the entire building stock, a structural damping of 5% and a strain-hardening of 3% are taken into account. In the last two columns, performance displacements computed by the two methods are compared.

Table 4.1. Characteristics of the Inspected Buildings

								NL Static Analysis	Proposed Equation
Building #	Structural. Type	No. of Stories	f_{ck} (MPa)	T_1 (s)	W (kN)	$V_{t,0}$ (kN)	R_y	U_{yN1} (m)	U_{max} (m)
1	Wall+frame	4	14+20	0.26	17,100	17,100	2.10	0.020	0.024
2	Frame	4	14	0.62	17,100	16,570	3.39	0.126	0.102
3	Frame	6	20	1.02	24,550	22,220	4.25	0.212	0.243
4	Frame	6	20	1.03	24,550	22,050	3.73	0.221	0.245
5	Wall+Frame	11	18	0.51	35,613	18,367	3.72	0.062	0.039
6	Frame	5	15	0.74	17,834	8,724	4.19	0.084	0.067
7	Frame	4	18	0.67	10,695	4,203	3.63	0.062	0.060
8	Frame	4	18	0.87	10,695	3,389	3.22	0.088	0.072

When the nonlinear static analysis performance displacements are compared with the ones computed by the empirical expression results, an average of 82.9% success rate is captured. When #5 building is removed from the list, for which the torsional irregularity is very high and number of stories for the application of nonlinear static analysis is not very convenient, then the success rate for estimating the nonlinear displacement demand increases to 85.7%. Figure 6 comparatively exhibits these results.

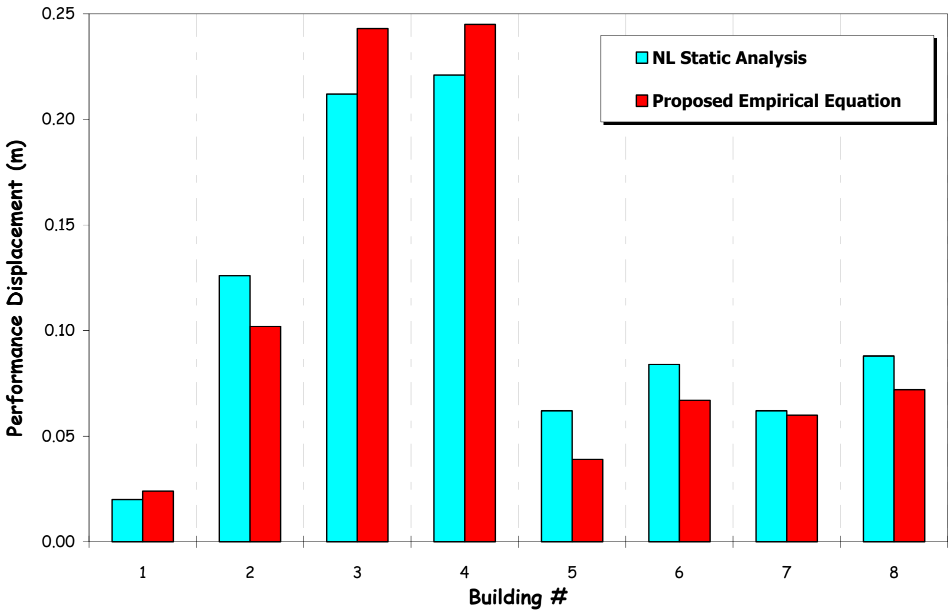

Figure 6 Comparison of NL static analysis results with proposed equations for 8 RC buildings

5. CONCLUSIONS

Currently in engineering applications, seismic performance of existing buildings is mostly evaluated by nonlinear static analysis. According to the procedure, a performance displacement is calculated and structural elements' capacities are controlled for this displacement level. In the first part of this paper, a handy empirical expression to estimate the inelastic displacement demand of an existing RC structure is introduced and later the expression results are compared with code procedure for a number of 8 buildings. This research has led the following conclusions so far:

- An empirical expression based on the nonlinear dynamic analysis of equivalent SDOF stiffness-degrading systems, is established as a function of natural vibration period T_n; yield-strength reduction factor R_y; amount of strain-hardening α and the characteristic spectral period T_c.
- During the nonlinear dynamic analysis, a set of recorded strong ground motions from Turkish earthquakes of Danger Zone-1 are selected, calibrated and filtered and an earthquake ensemble is established.
- In the empirical expression, stiffness degradation effect for RC structural elements, which is a significant parameter especially for structures with short periods, is considered.
- A number of 8 existing RC buildings are investigated in details so far. These buildings are modeled and computed by nonlinear static procedure as defined in the TERDC-2007 and performance displacements at roof levels are calculated. Then, inelastic deformation ratios and displacement demands are computed by the use of the proposed empirical expression. The success for the estimation of performance displacement is found out to be 82.9%, however when one building with a high level of torsional irregularity and number of stories more than 7, is removed from the list, success ratio increased to 85.7%. Therefore, empirical expression results are found to be encouraging, even for a few number of buildings.
- Increasing the amount of the building stock is a necessary and important issue, so that the precision of proposed empirical expression can be demonstrated. Furthermore, contribution of secondary modes should also be studied as a future work for the structures with irregularities.

Acknowledgement

This study is a part of the 106M048 research project supported by the Scientific & Technical Research Council of Turkey (TUBITAK) and gratefully acknowledged. Basis of this research has the inspiration from the fruitful discussions with Professor Chopra of UC Berkeley.

REFERENCES

Applied Technology Council, (1996). Seismic Evaluation and Retrofit of Concrete Buildings, Report No. **ATC-40**, Redwood City, CA-USA.

Building Seismic Safety Council, (1997). NEHRP Guidelines for the Seismic Rehabilitation of Buildings, **FEMA-273**, Federal Emergency Management Agency, Washington D.C.-USA.

Building Seismic Safety Council, (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, **FEMA-356**, Federal Emergency Management Agency, Washington D.C.-USA.

Chintanapakdee, C. and Chopra, A.K., (2003). Evaluation of the Modal Pushover Analysis Procedure Using Vertically 'Regular' and Irregular Generic Frames, EERC Report, **EERC-2003-03**, University of California, Berkeley-USA.

Chopra, A.K., (2001). Dynamics of Structures: Theory and Applications to Earthquake Engineering, Prentice-Hall Inc., NJ-USA, pp: 844.

Chopra, A.K., Chintanapakdee, C., (2003). Inelastic Deformation Ratios for Design and Evaluation of Structures: Single-Degree-of-Freedom Bilinear Systems, EERC Report, EERC-2003-09, University of California, Berkeley-USA, pp: 87.

Farrow, K.T. and Kurama, Y.C., SDOF demand index relationships for performance-based seismic design, *Earthquake Spectra*, **19**(**4**), 799-838, 2003.

Goel, R.K. and Chopra, A.K., (2004). Evaluation of modal and FEMA pushover analyses: SAC buildings, *Earthquake Spectra*, **20**(**1**), 225-254.

Ministry of Publicworks & Settlement, (2007). Turkish Earthquake Resistant Design Code - TEDRC, Ankara, Turkey.

Miranda, E., (2001). Estimation of inelastic deformation demands of SDOF systems, *Journal of Structural Engineering-ASCE*, **127**(**9**), 1005-1012.

Ruiz-Garcia, J.E. and Miranda, E., (2002). Influence of stiffness degradation on strength demands of structures built on soft soil sites, *Engineering Structures*, **24**, 1271-1281, 2002.

Song, J.K. and Pincheria, J.A., (2000). Spectral displacement demands of stiffness-and strength-degrading systems, *Earthquake Spectra*, **16**(**4**), 817-853, 2000.

Taskin, B., Kara, F.I., Mert-Tugsal, Ü., and Sezen, A., (2007). Empirical Relations for Calculating the Target Displacements of Existing & New-Design Structures Considering Turkish Earthquakes, *Proc. Int. Conference on Modern Trends in Structural Engineering for Seismic Design*, Ariel-Israel, 8-11 October.

Taskin, B., Sezen, A., Mert-Tugsal, Ü. and Kara, F.I., (2008). Empirical Relations For Calculating The Target Displacements of Existing & New Design Reinforced-Concrete Structures Considering Turkish Earthquake, Scientific & Technical Research Council of Turkey (TÜBITAK), Project No: **106M048**, Turkey.

Veletsos, A.S., Newmark, N.M., (1960). Effect of Inelastic Behavior on the Response of Simple Systems to Earthquake Motions, *2nd World Conference on Earthquake Engineering*, Tokyo-Japan, Vol.II, pp:895-912.