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CAPACITY DETERMINATION OF PRE-DAMAGED BUILDINGS

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ABSTRACT :

There is a large building stock built in earthquake regions where earthquakes frequently occur. It is very probable that such buildings experience earthquakes more than once throughout their economic life. However, there is lack of procedures to determine the change in building capacity as a result of prior earthquake damage. This paper focuses on establishing a method that can be employed to determine the loss in the building capacity after experiencing an earthquake. In order to achieve this goal a number of frames were analyzed under several randomly selected earthquakes. Nonlinear time-history analyses and nonlinear static analyses were conducted to assess the prior and subsequent capacities of the frames under consideration. The capacity curves obtained by these methods were investigated to propose a procedure by which the capacity of previously damaged structures can be determined. For time-history analyses the prior earthquake damage can be taken into account by applying the ground motion histories successively to the structure under consideration. In the case of nonlinear static analyses this was achieved by modifying the elements of the damaged structure in relation to the plastic deformation they experience. Finally a simple approximate procedure was developed using the regression analysis of the results. This procedure relies on the modification of the structure stiffness in proportion to the ductility demand the former earthquake imposes. The proposed procedures were applied to an existing 3D building to validate their applicability.

KEYWORDS: Capacity curve, Pushover analysis, Prior earthquake damage

1. INTRODUCTION

Structures built in earthquake regions may be subjected to earthquake forces more than once throughout their life. Prior earthquake damage would lead to changes in the structural characteristics which in turn imply changes in the response of the structure against future earthquakes. This effect can be taken into account by performing successive time-history analyses of the structure under consideration. Due to well known difficulties associated with the time-history analyses, nonlinear static analyses that produce approximate results are generally employed.

There has been very limited research focusing on the inclusion of the prior earthquake damage on the subsequent analyses of structures (Cecen, 1979; Araki et al., 1990; Hanson, 1996; Aschheim and Black, 1999). In these studies, most of which were based on shake table tests, the main objective was to determine the change in the displacement capability of the structures subjected to prior earthquake damage. This was achieved by subjecting the structures to successive ground motions and comparing the pre and post damage states. The findings of these studies generally revealed that prior earthquakes result no significant change in the displacement capacity. These results are believed to be mainly due to consideration of small prior earthquakes.

In order to investigate comprehensively the effect of prior earthquakes a number of frames were analyzed under several earthquakes employing both nonlinear time history of subsequent ground motions and nonlinear static analysis. A simple method for determining the changes in structural characteristics due to prior earthquake damage is proposed for use in seismic assessment.

2. DETERMINATION OF THE PRIOR EARTHQUAKE EFFECT

Six reinforced concrete frames with varying properties were selected. These frames include a two story-two bay frame named as F2S2B, a four story frame comprised of three bays entitled as F4S3B, three five story frames having two, four and seven bays and termed as, F5S2B, F5S4B, and F5S7B respectively, and finally an eight story-three bay frame named as F8S3B. Dynamic properties of these frames are presented in Table 1. These frames were first analyzed under ten earthquake records using nonlinear time history analysis. As shown in Figure 1, each earthquake record was applied twice successively to determine the response for a subsequent earthquake of the same intensity and the response corresponding to the second application was taken into account. The list of the earthquake records considered is given in Table 2. These ground motion records are scaled for each frame such that the frame is pushed into different levels of nonlinearity provided that it remains in the moderate damage region corresponding to the life safety performance. This way the frames will be pushed to six pre-determined deformation levels corresponding to different levels of damage. The deformation levels were determined based on the approximate single degree of freedom analyses of each frame under the given ground motions.

Table 1. Dynamic Characteristics of Selected Frames

Frame	Mass (ton)	Period (T_1, sec)	Modal Participation Factor (Γ_1)	Modal Mass Factor (α_1)
F2S2B	275.255	0.488	1.336	0.834
F4S3B	195.125	0.838	1.249	0.828
F5S2B	260.171	0.615	1.285	0.808
F5S4B	1007.120	0.887	1.340	0.802
F5S7B	769.136	0.723	1.269	0.813
F8S3B	1816.070	1.064	1.409	0.727

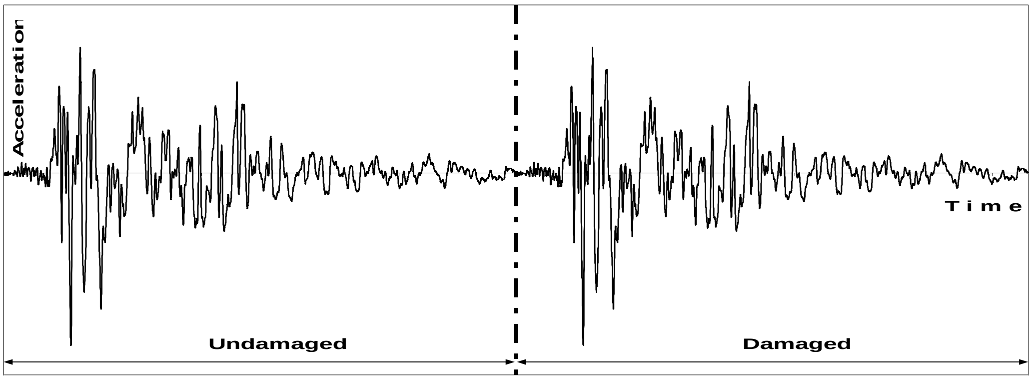

Figure 1 Successive application of ground motion

Table 2. Ground Motion Records

Rec. No	Record Name	Earthquake	Magnitude	Component	Site	PGA (g)	PGV (cm/s)	PGD (cm)
1	Düzce	Bolu-Düzce, 12/11/99	7.2	EW	Geomatrix or CWB (B) USGS ( )	0.513	86.1	170.12
2	Elcentro	Imperial Valley, 18/05/40	7.0	NS	Geomatrix or CWB (D) USGS (C)	0.319	29.8	13.32
3	Pacoima Dam	San Fernando, 09/02/1971	6.6	NS	Geomatrix or CWB (B) USGS ( )	1.170	54.3	11.73
4	Parkfield	Parkfield, 27/06/1966	6.1	N65E	Geomatrix or CWB (D) USGS (C)	0.476	75.1	22.49
5	El Centro 79a	Imperial Valley, 15/10/79	6.5	140	Geomatrix or CWB (D) USGS (C)	0.589	44.3	15.00
6	El Centro 79b	Imperial Valley, 15/10/79	6.5	NS	Geomatrix or CWB (D) USGS (C)	0.483	41.1	16.30
7	Chi-Chi	Chi-Chi, Taiwan, 20/09/99	7.6	360	Geomatrix or CWB (1) USGS (C)	0.359	42.1	16.40
8	Northridge-Pacoima	Northridge, 17/01/94	6.7	360	Geomatrix or CWB (A) USGS ( )	0.432	50.9	6.60
9	Cape Mendocino	Cape Mendocino, 25/04/92	7.0	360	Geomatrix or CWB (C) USGS (B)	0.549	42.6	13.40
10	Northridge	Northridge, 17/01/94	6.7	S00E	Geomatrix or CWB (D) USGS (C)	0.437	59.8	17.60

2.1. The Proposed Procedure

To take into account the influence of prior earthquakes in the nonlinear static analyses, first the pushover analysis of each frame was carried out. Then using equivalent single degree of freedom (SDOF) models obtained from the pushover curve and the dynamic properties of the frames, the displacement demand of the frame was obtained from nonlinear time history analyses of the corresponding SDOF system. The pushover results corresponding to the computed displacement demand are used to determine the plastic rotations at the ends of the yielding members. The rigidity of each element is then modified using equivalent linearization based on the plastic end rotations as illustrated in Figure 2. An additional pushover analysis of the frame with modified member rigidities was carried out to obtain its response reflecting the effect of prior earthquake damage.

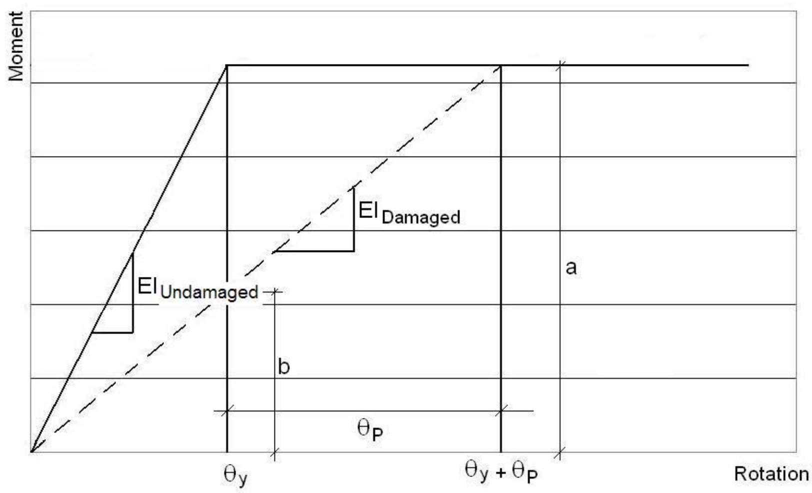

Figure 2 Determination of Modified Rigidity

The application of this procedure is summarized as follows;

Step 1. Analyze the undamaged structure to obtain its capacity curve.

Step 2. Calculate the performance point (displacement demand) and obtain the plastic rotations of elements at this point from the output file. The total rotation is obtained by adding the yield rotation to the plastic rotation.

Step 3. Use Equation 1 to compute the modification factor and multiply the moment of inertia of the elements with the modification factor.

Step 4. Determine the ultimate rotation of the damaged element by subtracting the plastic rotation from the ultimate rotation of the undamaged section.

Step 5. Re-analyze the structure to obtain the capacity curve for the damaged structure.

Step 6. Use the capacity curve obtained in Step 5 to calculate the displacement demand for the earthquake effect considered. Note that the earthquake effect can be represented by the response spectrum if a ground motion record is not available in which case approximate procedures such as the Capacity Spectrum Method (ATC, 1996) or the Displacement Coefficient Method (FEMA, 1997) can be used. This procedure uses the same general approach as proposed by other researchers (Bazzuro et al., 2004).

$$\frac{EI_{damaged}}{EI_{undamaged}} = \frac{\theta_y}{\theta_y + \theta_p} \tag{1}$$

The frames employed were analyzed using time-history analyses under the given earthquake ground motions at different deformation levels to obtain the maximum top floor displacements. Similar analyses were carried out using the pushover curves with corresponding SDOF systems and the results are compared in Figure 3. It is evidenced that SDOF based analyses results can reasonable predict the time-history analysis results. This indicated that the behavior of these frames is dominated by the first mode.

2.2. Comparison of Results

In Figure 4, the undamaged and damaged capacity curves are presented together with the maximum top displacement obtained from the time-history analyses corresponding to each deformation level separately for frame F4S3B. The curve named as “Capacity Curve” represents the capacity curve of the undamaged structure and the curve named as “Damaged Capacity Curve” stands for the capacity curve of the damaged structure. The vertical line represents the target deformation level that represents the degree of damage expected. The time-history solutions of the undamaged and damaged structure are presented in the graphs by filled and unfilled symbols. It is clearly seen from the time-history analyses results that as the degree of damage due to prior earthquakes increases the deformation due to subsequent earthquakes also increases. As evidenced from these results, the initial stiffness of the structures decreases as the damage level due to prior earthquake increases. This is an expected result because a higher damage level will cause an increase in both the number of yielding elements and the amount of plastic rotation that the elements experience. As the number of elements going into the inelastic range increases, there will be more elements whose moment of inertia is decreased thus leading to a softer structure. Similar behavior is observed for other frames as well. The capacity curve of the damaged structure is expected to converge to that of the undamaged structure at the performance point. In general this tendency is achieved in the frames analyzed.

Comparison of the displacement demands obtained for the damaged structure using time-history and nonlinear static analyses revealed that the percentage error is generally within 30 percent when all the results are averaged as shown in Table 3. Since all approximate procedures are intended to provide satisfactory results on the average, the observed error margins are considered to be within acceptable limits.

In the approximate procedure outlined above, the residual displacement and unloading stiffness of the member load-deformation relationships were included through the use of the secant stiffness. This assumption has been tested in Figure 5. Firstly the unloading stiffness of the equivalent SDOF system was considered to be equal to its initial stiffness. The ground motions were applied to the undamaged SDOF system and the residual displacement was recorded. This residual displacement was added to the undamaged SDOF maximum displacement. This total displacement was compared with the SDOF results obtained using the capacity curve. As can be seen from Figure 5 there is large scatter in the results. The assumption employed in this study assumes a ratio of 1.00 shown by the dashed line. This assumption seems reasonable when compared to the mean of the data computed as 1.05 and shown with the solid line in the figure. The procedure proposed here may be used for a given seismic effect represented by a response spectrum so the inclusion of residual displacement that requires a ground motion record is not possible in this case.

3. SUMMARY

The aim of this study was to determine a procedure that can be used to assess the change in building capacity after experiencing an earthquake so that the probable damage of a second earthquake that hits the structure can be estimated. Six frames and ten randomly selected earthquakes were used for this aim. Initially the selected frames were analyzed by the time-history analysis under successively applied ground motions to include the prior earthquake effect on these frames. Alternatively, an approximate procedure based on the pushover analyses of the original (undamaged) and damaged frames was applied. The base shear-roof displacement pairs obtained by the time-history analyses were compared with the capacity curve obtained by the nonlinear static procedure to confirm the reliability of the pushover procedure.

It is very important to note that a major earthquake occurring at a far site or a small earthquake occurring at a near site can result in damage in the structures. The degree of damage might vary from none to moderate. Especially light damage is not visually apparent. In cases like this, the change in the original capacity of the building needs to be taken into consideration when its performance for future earthquakes is evaluated. The approximate procedure proposed for the determination of the capacity curve of a structure that has been subjected to prior earthquakes revealed that conservative results within reasonable error bounds were obtained as compared to time-consuming time-history analysis results. So the proposed procedure can alternatively be used for the assessment of the buildings that have been subjected to prior earthquakes.

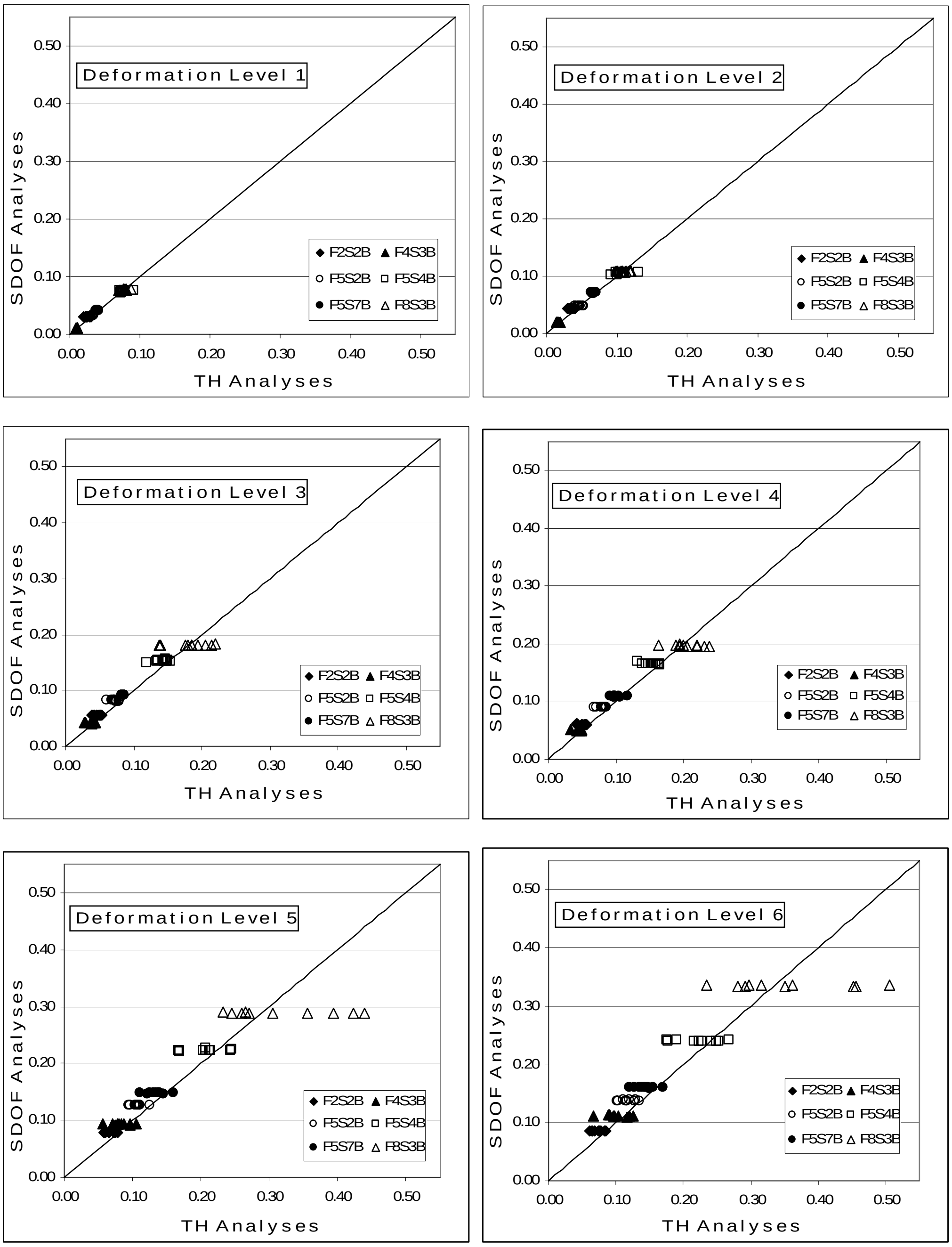

Figure 3 Comparison of SDOF and MDOF Results

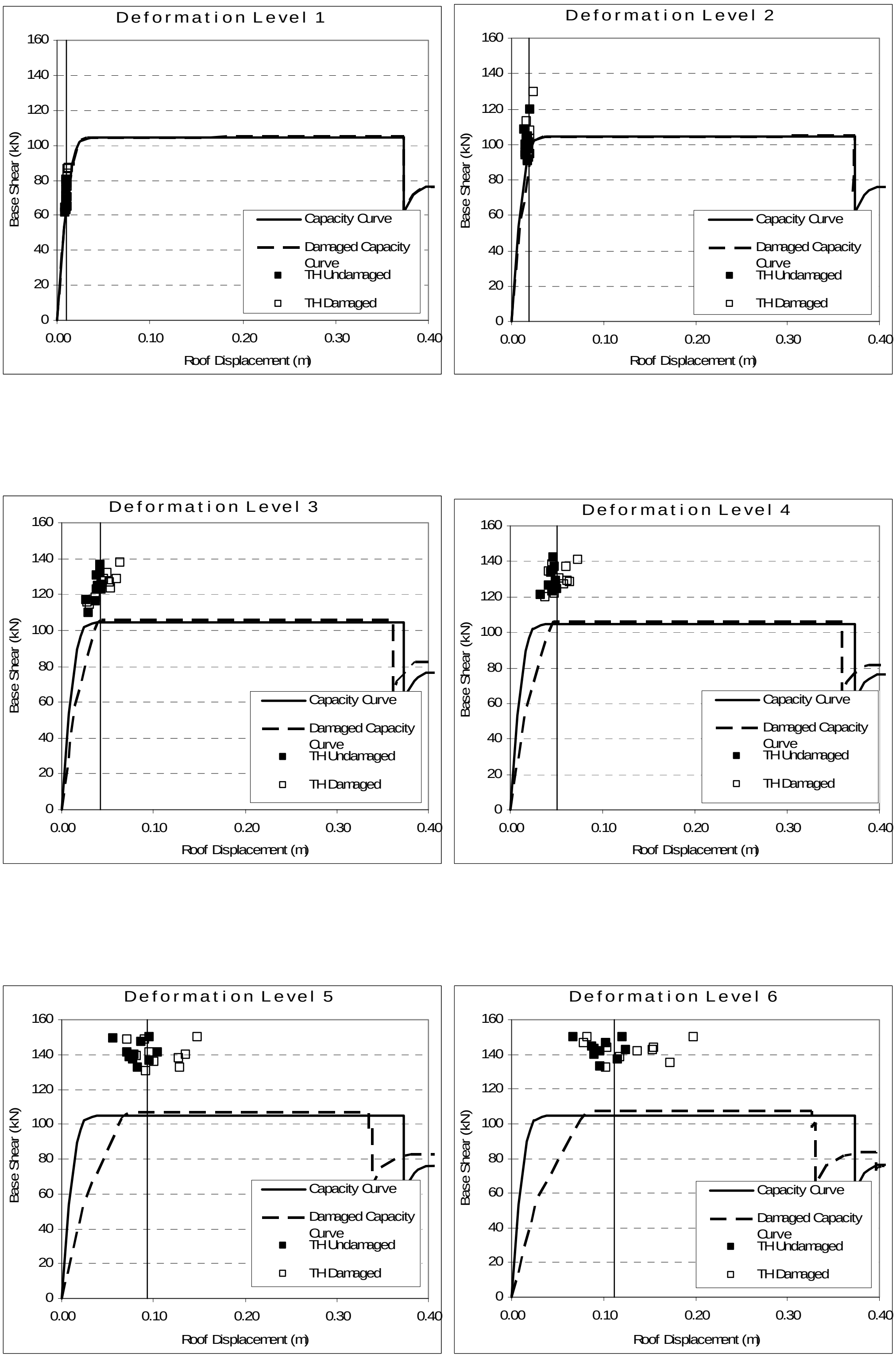

Figure 4 Undamaged and Damaged Capacity Curves and Time-History Results of Frame “F4S3B”

Table 3 Percent Error of Nonlinear Static Analyses Relative to Time history Analyses Results

Deformation Level	F2S2B	F4S3B	F5S2B	F5S4B	F5S7B	F8S3B
I	27.26	5.46	1.82	3.77	3.85	4.69
II	18.45	20.62	12.86	9.60	8.49	8.05
III	31.27	28.46	29.52	12.96	20.54	21.59
IV	27.67	39.13	31.46	15.37	24.75	20.40
V	37.42	30.01	35.63	18.59	30.35	28.09
VI	42.43	26.68	28.95	19.25	31.93	32.96

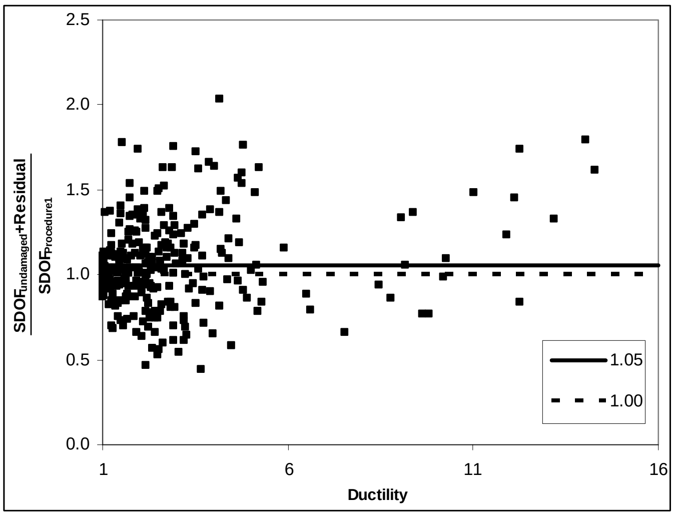

Figure 5 Inclusion of Residual Displacement

REFERENCES

ATC (1996). Seismic Evaluation and Retrofit of Concrete Buildings, *Applied Technology Council (ATC-40 Report)*, Vol. I, Redwood City, California.

Araki, H., Shimazu, T. and Ohta, K. (1990). On Residual Deformation of Structures After Earthquakes, *Proceedings of the Eight Japan Earthquake Engineering Symposium-1990*, Tokyo, Vol. 2, pp. 1581-1586. (In Japanese with English abstract)

Aschheim, M., Black, E. (1999). Effects of Prior Earthquake Damage on Response of Simple Stiffness-Degrading Structures, *Earthquake Spectra*, Vol. 15, No. 1, pp. 1-24.

Bazzurro, P., Cornell, A., Menun, C., Luco, N., Motahari, M. (2004). Advanced Seismic Assessment Guidelines, *Pacific Gas & Electric* (PG&E)/PEER Lifelines Program Task 507

Cecen, H. (1979) Response of Ten-Story Reinforced Concrete Model Frames to Simulated Earthquakes, *Ph.D. Thesis*, Department of Civil Engineering, University of Illinois at Urbana.

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

Hanson, R.D. (1996). The Evaluation of Reinforced Concrete Members Damaged by Earthquakes, *Earthquake Spectra*, Vol. 12, No. 3, Earthquake Engineering Research Institute, Oakland, California.