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Seismic structural damage assessment of reinforced concrete ductile framed structures

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ABSTRACT: To find a rational procedure for carrying out damage assessment analyses of reinforced concrete ductile framed structures under elastic code level earthquakes specified in New Zealand codes [NZS4203 1992], three main aspects are the earthquake scaling, the relationship between the member curvature ductility and structural displacement ductility, and the effect of varying hysteresis models and member damage indices on the damage indices for the storeys and structures respectively. These three aspects were studied by carrying out elastic and inelastic time-history analyses employing three fully ductile structures (design structural ductility of 5.0 assumed), four earthquakes, seven hysteresis rules and four member damage indices [Dong 2003]. Finally, a procedure for the seismic structural damage assessment is recommended.

1 INTRODUCTION

The member curvature ductility, interstorey displacement ductility and structural displacement ductility have been used as damage measures for the members, storeys and overall structures respectively in most of the current seismic codes [IAEE 1996]. This means that the degree of damage of the components, storeys and the structure are dependent only on the corresponding ductility demands and the ductility capacities. However many other damage assessment models have been developed and used to measure the degree of damage for reinforced concrete structures [Park 1985, Roufaiel 1987, Cosenza 1993 and Banon 1982], which are based on ductility, dissipated energy or stiffness variations. The damage in the members, storeys and overall structures are usually quantified by the corresponding damage indices. These damage indices are regarded as being more suitable for the damage evaluations rather than the corresponding ductilities [Carr 1993].

Varying the member damage indices and the hysteresis models may result in significant variations in the predicted damage indices or assessment. This would greatly influence a structural engineer when making an assessment of the damage in the structure of reinforced concrete for the elastic code level earthquake excitations. Hence the effect of varying member damage indices and hysteresis models on the damage indices for the storeys and the structures should be identified.

When carrying out damage assessment analyses of reinforced concrete ductile-framed structures under elastic code level earthquakes as required in New Zealand codes, the earthquake excitations to be used as earthquake inputs should be scaled so as to have the elastic responses in terms of the base shears and displacements (interstorey and roof level displacements) match or greater than the corresponding code values derived from the elastic code acceleration spectrum and the elastic code displacement spectrum respectively at the ultimate limit state to assure that enough strength earthquakes obtained.

Many scaling methods [Moss 1992] have been proposed and used by many researchers [Lin 1999 and Satyarno 2000]. However, the issues relating to these scaling methods are that the elastic responses to the scaled earthquakes may significantly smaller than the elastic code level base shears and displacements (interstorey drift and roof level displacement) implied in the codes. By studying the elastic responses and the response spectra of the three structures subjected to the four earthquake excitations scaled using six scaling methods, a new scaling method is proposed, by which the responses to the scaled earthquakes match or greater than both the base shears and the displacements (interstorey drift and roof level displacement) at the ultimate limit state.

The member damage indices, such as those of Park & Ang [Park 1985], Banon & Veneziano [Banon 1982], Roufaiel & Meyer [Roufaiel 1987] and Cosenza et al [Cosenza 1993] used in this study, are related to each other and sensitive to the member ultimate curvature ductility capacity, which is a property of a specified member. In order to check if the member ultimate curvature ductility capacity can exceed the curvature ductility demand in the members of the structures under the elastic code level earthquake excitations, the ratios between the member curvature ductilities to the overall structural displacement ductilities, defined using the Carr & Tabuchi [Carr 1993] approach, are obtained for each of the three structures considering the effect of varying the choice of seven different hysteresis models.

In this study, the damage indices for the storeys and structures are computed as the energy weighted average of the entire inelastic member damage indices in storeys and the whole structures respectively, presented by Park & Ang [Park 1985]. The four member damage indices and the seven hysteresis rules are used to determine the effect of varying either the member damage indices or the hysteresis models on the damage indices for the storeys and structures respectively.

2 THE THREE STRUCTURES, SEVEN HYSTERESIS RULES, FOUR MEMBER DAMAGE INDICES

2.1 The three structures

The three ductile reinforced concrete ductile 3 bay moment resisting frames are 6, 12 and 18 storeys high respectively with a 9.2metre span length and interstorey height of 3.65 meters designed according to the current New Zealand codes [NZS4203 1992, NZS3101 1995] using the capacity design philosophy [Paulay 1992] with a structural displacement ductility of 5.0. The three structures were assumed to be situated on intermediate subsoil near the centre of Christchurch, New Zealand. 92% of the base shear is distributed to be the equivalent static lateral forces at each level, proportioned to the lumped weights and the heights at levels under consideration, and an additional horizontal force of the 8% left is added to the roof level force considering the effects of higher modes. Fig. 1 shows the elevation of the 6 storey structure.

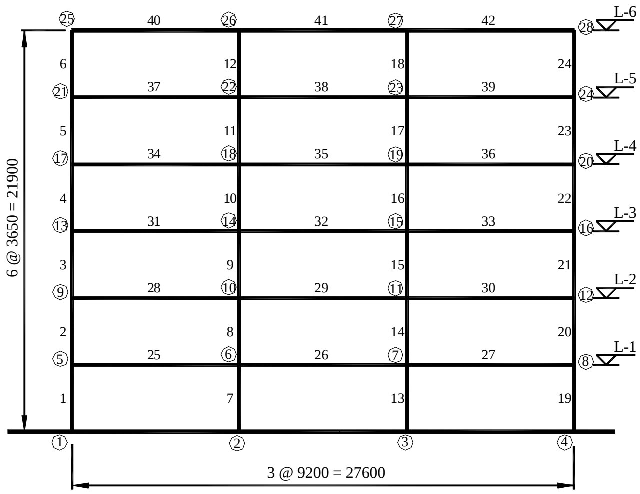

Fig. 1 Six storey frame member and node numbering and level definition

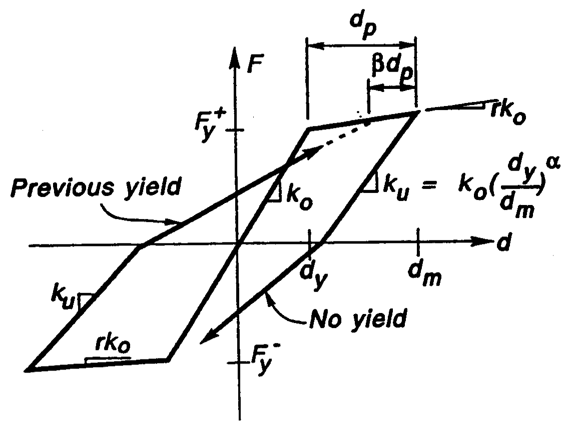

Fig. 2 Modified TAKEDA hysteresis rule [Otani 1974]

2.2 The seven hysteresis rules

The seven hysteresis models including the Elasto-Plastic, Bilinear, Modified Takeda (α=0.0, β=0.6), Degrading Bilinear (α=0.5) shown in Fig. 3, Clough as shown in Fig. 4, Modified Takeda (α=0.3, β=0.4) and the Q-Hyst (α=0.5) [Carr 1998] were used in modelling the inelastic behaviour in the reinforced concrete members dominated by the flexural actions. Fig. 2 shows the Modified Takeda rule [Otani 1974], which has two stiffness degrading factors a and b. The unloading stiffness after yielding is $(d_y / d_m)^a$ times the initial elastic stiffness k_o, which is similar to the approach used by Emori and Schnobrich [Emori 1978]. The response point during reloading moves toward the point whose displacement is $(d_m - b * d_r)$, where d_m is the displacement of the previously maximum inelastic response point. When the reloading stiffness factor b is zero for the Modified Takeda rule, this rule becomes the Q-Hyst rule. If the unloading stiffness factor a is zero for the Q-Hyst rule, i.e. no unloading stiffness degradation, the Q-Hyst rule becomes the Clough hysteresis rule.

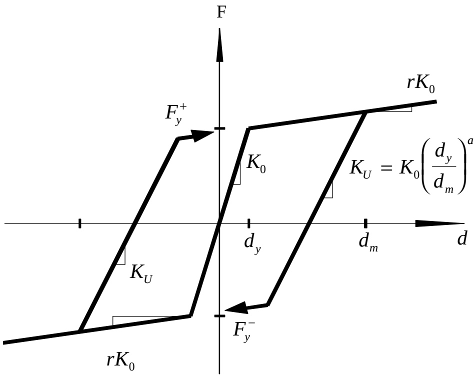

Fig. 3 Degrading Bilinear Model

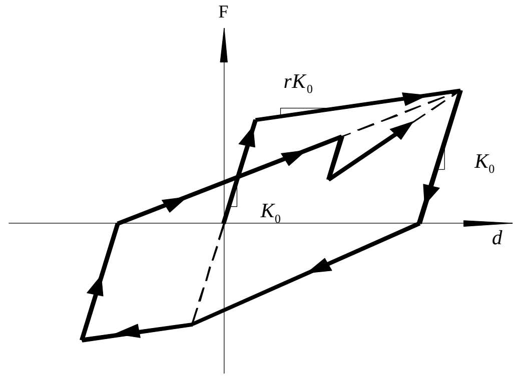

Fig. 4 Clough Degrading Model

2.3 The four member damage indices

Let m_m, m_y, m_u, F_m, F_y, F_u ,and E_h be the maximum, yield and ultimate ductilities, the maximum, yield and ultimate actions, and the dissipated hysteretic energy respectively, the four different member damage indices used are given below, in which the first two are ductility and dissipated energy related, and the third and the forth are stiffness degraded and normalised ductility related respectively:

Park & Ang:
$$DI_{mem} = \frac{m_m}{m_u} + \frac{bE_h}{F_y m_u} \tag{1}$$

where b is an experimental parameter (=0.05 in this study).

Banon & Veneziano:
$$DI = \frac{\sqrt{\left(\frac{m_m}{m_y} - 1\right)^2 + \left[1.1\left(\frac{2E_h}{F_y m_y}\right)^{0.38}\right]^2}}{Numerator \quad for \quad monotonic \quad loading} \tag{2}$$

Roufaiel & Meyer: $$DI = \frac{\dfrac{m_m}{F_m} - \dfrac{m_y}{F_y}}{\dfrac{m_u}{F_u} - \dfrac{m_y}{F_y}} \quad (3)$$

Cosenza et al : $$DI_e = \frac{m_m - 1}{m_u - 1} \quad (4)$$

3 THE FOUR UNSCALED EARTHQUAKES AND EARTHQUAKE SCALING

3.1 The four earthquakes

Four different natural earthquake records, i.e. Bucharest (1977-NS), El Centro (1940-NS), Northridge (Sylmar-949NW) and Kobe (1995-NS) were chosen for earthquake inputs, as shown in Table 1.

Table 1 General description of the four earthquake records

No.	Earthquake	Site	Date	Component	PGA (m/s^2)	PGV (m/s)	Ratio A/V (1/S)	Duration (S)
1	Romania	Bucharest	4/03/1977	N-S	0.21g	0.73	0.29g	16
2	Imperial Valley California	El Centro	18/05/1940	N-S	0.35g	0.38	0.92g	20
3	Northridge	Sylmar	17/01/1994	949-NW	0.80g	1.12	0.71g	20
4	Kobe, Japan	MA, observatory	17/01/1995	N-S	0.84g	0.92	0.91g	20

3.2 Six earthquake scaling methods

Six scaling methods were used in an attempt to find a rational method for earthquake scaling by which both the base shear and displacements (interstorey drift and roof level displacement) required in codes can be satisfied. The six scaling methods are shown below:

Method 1: $SF = R_1$ (5)

Method 2: $SF = 0.92R_1 + 0.05R_2 + 0.03R_3$ (6)

Method 3: $$SF = \frac{T_1R_1 + T_2R_2 + T_3R_3}{T_1 + T_2 + T_3} \quad (7)$$

Method 4: $$SF = \frac{M_1^eR_1 + M_2^eR_2 + M_3^eR_3}{M_1^e + M_2^e + M_3^e} \quad (8)$$

Method 5: $$SF = \frac{V^{design}}{V} \quad (9)$$

Method 6: $$SF = \frac{ID^{design}}{ID} \quad (10)$$

where R_n, T_n and M_n^e (n=1, 2 and 3) are the ratios of the design spectral accelerations to those for the unscaled earthquakes, the natural periods of free vibrations and the effective masses respectively for the first three modes. V^{design} and ID^{design} are the elastic code design base shear and elastic code design maximum interstorey drift respectively. The elastic code design interstorey drifts ID^{design} (and the elastic code design roof level displacements to be talked in section 3.3) were obtained using the square root of the sum of the squared (SRSS) combination method with the design spectral displacements for the first four modes, derived from the relationship between the spectral displacements and the spectral accelerations for a single degree of freedom system. The design spectral accelerations are available from the codes. *V* and *ID* are the maximum base shear and the maximum interstorey drift produced by the unscaled earthquake excitations.

3.3 Maximum elastic responses of the three structures versus the six scaling methods

The elastic responses in terms of base shears, interstorey drifts, roof level displacements and spectral accelerations for the first six modes of the three structures were computed by carrying out dynamic step-by-step integration time history analyses and spectral analyses using RUAUMOKO [Carr 1998] using the four earthquake records scaled in accordance with each of the six scaling methods. Then these maximum responses were compared to their corresponding elastic design values required in the codes.

It is found that the base shears and interstorey drifts for the earthquakes scaled by using method 5 (base shear scaling method) and method 6 (interstorey drift scaling method) respectively are exactly the same as those required at the design level. This is independent of the fundamental periods of the free vibration of the structures and the earthquake types. As an example, Figs. 5 and 6 show the base shears and interstorey drifts of the 6 storey structure for the four earthquakes and for the Bucharest (1977-NS) earthquake respectively versus the six scaling methods.

It is also found, as shown in Fig. 7 for the 6 storey structure, that the maximum roof level displacements of the three structures to the earthquakes scaled by using both the base shear scaling method and the interstorey drift scaling method were similar when compared to the corresponding design values except for the Bucharest [1977-NS] excitation. This means that using any of the two scaling methods will match the design roof level displacement.

It is found from Fig. 6 that the maximum interstorey drift (at level 2) to the earthquake scaled using the base shear method gave larger drift than the design value. This means that the base shear scaling method can satisfy the design interstorey drift for the 6 storey structure, as well as match to the design base shear and the design roof level displacement. Hence this scaling method can be used for earthquake scaling for the 6 storey structure and similar short period structures.

However, as shown in Figs.5, 6 and 7, the base shear, interstorey drift and roof level displacements of the 6 storey structure to the Northridge (Sylmar-949NW) earthquakes scaled by using the four scaling methods other than the base shear and interstorey drift scaling methods were significantly smaller or larger than the corresponding design values. Hence, these four scaling methods are not appropriate for use as earthquake scaling methods.

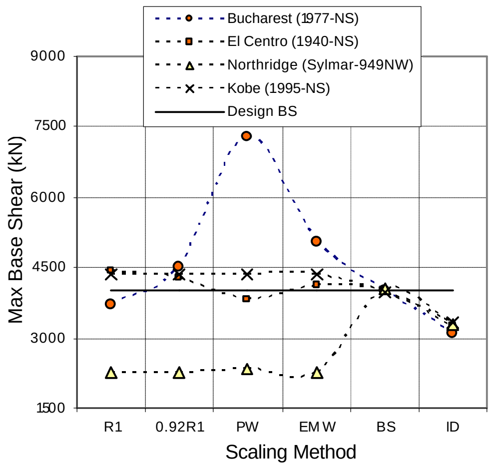

Fig.5 Maximum base shear of the 6S versus the six scaling method

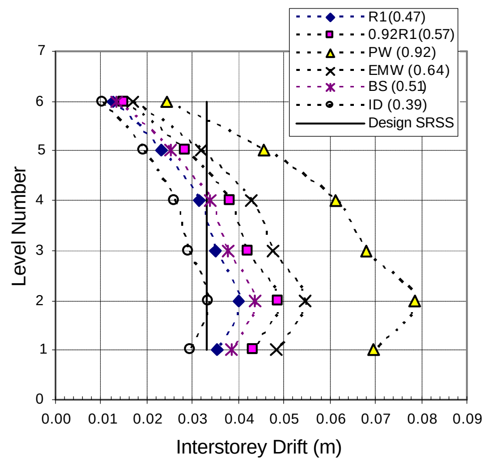

Fig. 6 Maximum interstorey drift of the 6S under the Bucharest (1977-NS) versus the six scaling method

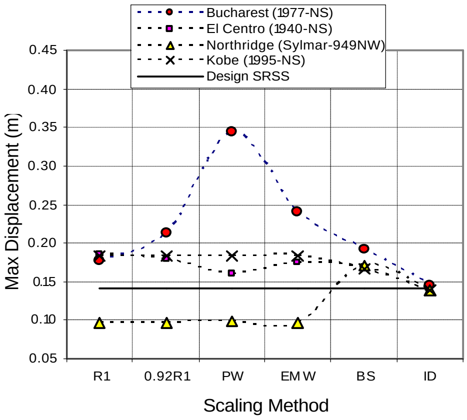

Fig. 7 Maximum roof level displacements of the 6S under the four earthquakes versus the six scaling method

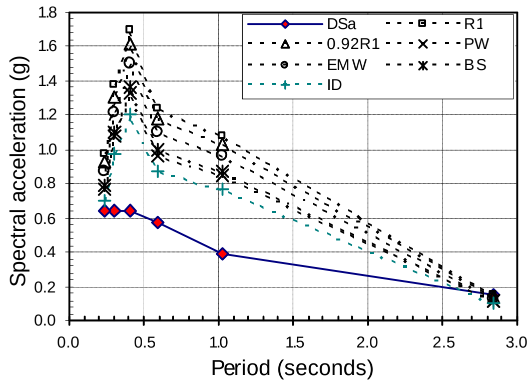

Fig.8 5% damped spectral accelerations for the first six modes of the 18S structure under the kobe (1995- NS) earthquake versus the six scaling method

From Fig.8, it is found that the spectral accelerations for the first six modes of the 18 storey structure to the earthquakes scaled using both the base shear scaling and the interstorey drift scaling methods were significant larger than those at design level represented by the solid line. When the spectral accelerations are very similar to the design values, in the case as using the frequency scaling method [Moss 1992], the base shear and interstorey drift to the earthquakes scaled using the frequency scaling method will be significant smaller than those for the base shear scaling and interstorey drift scaling methods, i.e. significant smaller than the design base shear and design interstorey drift. Hence, the frequency scaling method is not recommended for earthquake scaling.

In order to have earthquakes scaled so as to match the requirements implied in the codes for longer period structures such as the 12 and 18 storey structures, the scaling factor *SF* may be determined by using Eq. 11, which is the larger of the two scaling factors obtained using the base shear and interstorey drift scaling methods:

$$SF = max\left\{\frac{ID_{SRSS}^{design\ spectra}}{ID_{SF=1.0}},\ \frac{V^{design}}{V_{SF=1.0}}\right\} \tag{11}$$

where $ID_{SRSS}^{design\ spectra}$: design interstorey drift from SRSS of the spectral displacements.

V^{design} : design base shear determined by using equivalent static method.

$ID_{SF=1.0}$: the maximum interstorey drift for unscaled earthquake excitation.

$V_{SF=1.0}$: the maximum base shear for unscaled earthquake excitation.

4 RATIOS OF MEMBER CURVATURE DUCTILITY TO STRUCTURAL DISPLACEMENT DUCTILITY

The earthquakes scaled by using the Eq. 11 were used as elastic code earthquakes to excite the three structures while using each of the seven hysteresis models to study the relationship between the member curvature ductility and the structural displacement ductility. The displacement ductility for the storeys and structures were determined based on the Carr & Tabuchi [Carr 1993] approach by which the yield displacements for the storeys and structures were defined. The overall structural yield points are defined as the intersection point of two trend lines representing the relationship between the maximum base shear and the roof-level displacement for the elastic and inelastic responses respectively.

The storey yield points are defined in a similar way. The response points are the interstorey shears and interstorey drifts rather than the base shears and roof-level displacements for the overall structural yield points. Fig.9 shows the two elastic and inelastic trend lines for defining the structural yield displacement of the 6 storey structure with using the Elasto-Plastic hysteresis model as an example.

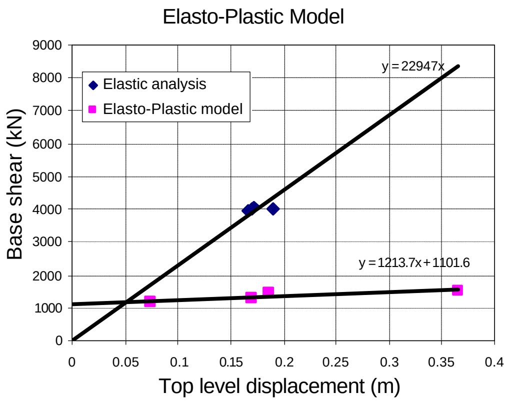

Fig.9 Definition of the structural yield displacement of the 6 storey structure using the Elasto-Plastic model for Carr & Tabuchi approach

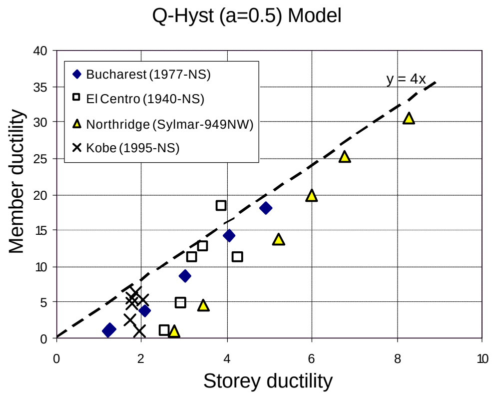

Fig. 10 Relationship between the member curvature ductility and the storey displacement ductility for the 6 storey structure under the four scaled earthquakes using the Q-Hyst (α=0.5) model

The trend line ratios of the member curvature ductility to the storey displacement ductility are 4, 3 and 3 for the 6, 12 and 18 storey structures respectively. This is independent of the choice of the hysteresis models. The ratios of the member ductility to the structural ductility and the required member ductility for a design structural ductility of 5.0 are summarised in Table 2.

Table 2 Ratios of the maximum beam curvature ductility to maximum structural displacement ductility, and required beam curvature ductility for structural ductility 5.0

	Ratios of beam member duct. to storey duct.	Ratios of storey duct. to structural duct.	Ratios of member duct. to structural duct.	Structural ductility	Required member ductility
6S	4.0	1.03~1.40	4.12~5.60	5.0	21~28
12S	3.0	1.48~2.46	4.44~7.38	5.0	22~37
18S	3.0	1.34~1.92	4.02~5.76	5.0	20~29

5 EFFECT OF VARYING HYSTERESIS MODELS AND MEMBER DAMAGE INDICES ON DAMAGE INDICES FOR STOREYS AND STRUCTURES

The ultimate member curvature ductilities of up to 30 can be obtained for beam members carefully designed and detailed to the New Zealand codes which are compared with those shown in Table 2 and were used in computation of the member damage indices. Four member damage indices and seven hysteresis models were employed in computing the member damage indices using the program RUAUMOKO [Carr 1998].

It has been found that by varying either the hysteresis models or the member damage indices, the distributions of storey damage indices remain unchanged for each of the three structures when subjected to any of the four design earthquakes. The storey damage index distributions are earthquake type dependent only for a specified structure, as is expected.

Fig. 11 shows the ratios of storey damage indices for each of the seven hysteresis models to those of the average of the storey damage indices for the three hysteresis models, the modified Takeda (α=0, β=0.6), Clough and modified Takeda (α=0.3, β=0.4) for the 6 storey structure under the Bucharest (1977-NS) earthquake using the Park & Ang member damage index. Fig. 10 shows the structural damage indices for each of the four member damage indices of the 6 storey structure under the El Centro (1940-NS) earthquake using each of the seven hysteresis models.

It is found, from Figs. 11 and 12 as an example, that the three hysteresis models, i.e. the modified Takeda (α=0, β=0.6), Clough and modified Takeda (α=0.3, β=0.4) predict similar storey damage indices and overall structural damage indices. This is independent of both the earthquake type and the member damage indices for a structure. When compared to those storey and structural damage indices for the three hysteresis models, the Elasto-Plastic, Bilinear and the Degrading Bilinear (α=0.5) hysteresis models predict significant smaller storey damage indices, as shown in Fig. 9. The Elasto-

Plastic and Bilinear models predict significant smaller overall structural damage indices for the 12 and 18 storey structures under specified earthquake excitations. However the Q-Hyst (α=0.5) model predicts, in general, larger values in both the storey and the structural damage indices as shown in Fig. 9 for the 6 storey structure as well as for the 12 and 18 storey structures. Hence the Q-Hyst (α=0.5) model is recommended to be used for the computations of storey and structural damage indices when realistic member hysteresis responses are unavailable, based on a conservative engineering safety point of view.

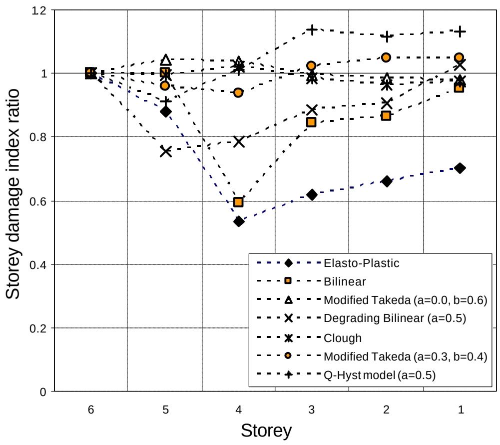

Fig. 11 Storey damage index ratios of the 6 storey structure under the Bucharest (1977-NS) earthquake using the Park & Ang member damage index

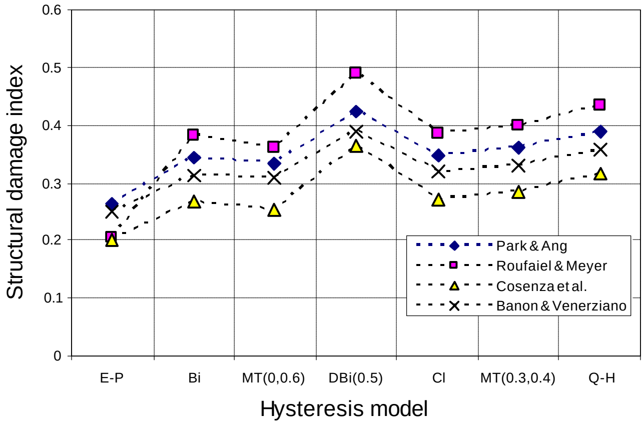

Fig. 12 Structural damage indices of the 6 storey structure under the El Centro (1940-NS) earthquake using each of the seven hysteresis models

In general, the damage indices for the storeys and structures are in the following order for the four member damage indices, (Roufaiel & Meyer) > (Park & Ang) > (Banon & Veneziano) > (Cosenza et al.) for using each of the seven hysteresis models except the Elasto-Plastic model. Due to that the damage indices for the members, storeys and structures for the Park & Ang damage model are based on the laboratory tests, and the interpretations of the damage indices for the degrees in damage are available, the Park & Ang damage model is recommended for computing the damage indices for the members, storeys and structures until the interpretations of the damage indices for the other three member damage indices are available.

6 RECOMENDED PROCEDURE FOR DAMAGE ASSESSMENT

Based on the studies presented above, a procedure for seismic damage assessment of a reinforced concrete ductile framed structure is recommended as follows:

Step 1: Carry out a modal response analysis to obtain the natural periods of free vibration and the mode shapes for the first few modes. The total effective mass for the modes to be combined should exceed 90% of the total mass of the structure.

Step 2: If the natural periods of the structures are less than or approximately 1.35 seconds, the base shear scaling method can be used for earthquake scaling, followed directly by Step 8. Otherwise proceed through the following steps.

Step 3: Calculate the design spectral displacements for the chosen first few modes from the design displacement spectrum.

Step 4: Compute the design spectra SRSS interstorey drift using the design spectral displacements and mode shapes for the first few modes.

Step 5: Calculate the design base shear applying the method used in the preliminary design stage with structural ductility of 1.0 (the equivalent static method in this study).

Step 6: Carry out dynamic elastic time-history analysis for the structure under consideration subjected to the unscaled earthquake to obtain the maximum interstorey drift and base shear.

Step 7: Determine the scaling factor *SF* by Eq. 11.

Step 8: Conduct the damage analyses using the Q-Hyst (α=0.5) hysteresis model and the Park & Ang damage model.

6 CONCLUSIONS

From the above studies, three main aspects, i.e. a rational earthquake scaling method, relationship between member curvature ductility and structural displacement ductility, effect of varying hysteresis rules and member damage indices on damage indices for storeys and structures are obtained by carrying out elastic and inelastic time-history analyses. This leads to a proposed procedure for a rational seismic damage analysis of reinforced concrete ductile framed structures.

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