評估性能設計中考慮非彈性地震力需求之殘餘變位與倒塌之影響

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評估性能設計中考慮非彈性地震力需求之殘餘變位與倒塌之影響

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ABSTRACT

The main objective of this research project is to residual deformation and collapse mechanism of structural system and to assess the seismic demand of this system. In this study three major works are developed:

a. Develop a physical-based inelastic hysteretic model which contains stiffness and strength degradation, pinching and residual deformation,

b. Based on the proposed model the seismic demands of structural system are examined. Emphasis is concentrated on the study of MDOF system,

c. Conduct the shaking table test of collapse. Examine the collapse behavior and develop the hysteretic model which included the collapse mechanism.

Through this study three research papers were generated and they are listed as follows:

1. Loh, C.H., L.P. Hong, C.L. Wu, W.I. Liao and Y.S. Yang
   “Evaluation of Maximum Drift Demand for Seismic Performance Assessment of RC Buildings,”
   Paper for the Int. Conf. of 5th Anniversary of 921 Ch-Chi Earthquake, Sept. 2004


PART 1:

Evaluation of Maximum Drift Demand for Seismic Performance Assessment of RC Buildings

ABSTRACT

The objective of this paper is to study the inelastic dynamic response of multi-degree-of-freedom systems and develop procedures to quantify seismic demands on such systems for use in preliminary design. Two kinds of typical reinforced concrete buildings (pure frame structure and wall-frame structure) with three different heights (4-, 6- and 12-story buildings) were selected for nonlinear time history analysis. The physical-based hysteretic model with post-yielding stiffness, stiffness degradation, load deterioration and pinching effects is implemented in the model structure. A comprehensive study of seismic demands (i.e. strength, deformation and energy) for frame structure and wall-frame structure is presented. The results presented are aimed at highlight the seismic response of the above mentioned systems in order to provide conceptual procedures which permit the evaluation of MDOF demands using the corresponding SDOF information. To incorporate collapse and residual deformation in structural performance assessment, a theoretical-based hysteretic model is presented in the study to include post-peak behavior into nonlinear dynamic analysis. This model can reflect the collapse behavior of a structural system Comparisons are also made in system responses with and without consideration of collapse mechanism.

INTRODUCTION

In performance-based seismic design, the design index can reflect the structural and non-structural damage that may occur during earthquake ground motion excitations. Many researchers continuously pay attention to study on several seismic demands as the design indices. Recently there has been a growing interest in deformation-based design procedures in which lateral displacement demands are used instead of lateral force demands. In order to perform this new procedure a method based on Displacement Modification Factor for estimating the maximal inelastic displacement of the SDOF systems is arranged and discussed by Miranda (2002[1]). And it is most desirable to establish relationships a spectral deformation quantity and structural deformation quantity that control design and performance. A simplified process that provides quick and reasonable estimates of seismic demand for the MDOF systems is proposed by Krawinkler (1997[2], 2000[3], 2003[4]). Thus, adequate damage control can be achieved if lateral deformations are controlled by providing enough lateral stiffness, lateral strength, and energy dissipation capacity to a structure. On the other
hand, another design procedure called probability-based seismic assessment was proposed by Cornell (2003[5]). It is a dual procedure which couples conventional probabilistic seismic hazard analysis with nonlinear dynamic response analyses. By the way, in order to get better understanding of the changes in the nature of the performances of the structure under severe ground motions Prof. Cornell also proposed the method Incremental dynamic analysis (2002[6]).

Moreover, it is also recommended that the damage index and the residual deformation of the structure should be considered in performance-based design (Bertero 1996[7], Pampanin 2003[8]). The damage index is related to the energy demand which is dissipated by cyclic loading of an excitation and the residual deformation plays an important role to cause the collapse of the structure.

The objective of this research is to increase our understanding of the inelastic dynamic response of multi-degree-of-freedom systems and to develop procedures to quantify seismic demands on such systems for use in preliminary design using response spectral representations of input ground motions, as shown in Fig.1. Seismic response of an equivalent SDOF system with the consideration of post-peak negative stiffness behavior and residual deformation was also investigated.

![Figure 1: Process for simplified demand estimation.](image)

**DEVELOP INELASTIC MDOF SHEAR BUILDING**

**Nonlinear analytical Model** In this study the nonlinear hysteretic model proposed by Mostaghel (1999[9],2000[10]) is used for analysis. This nonlinear model has four nonlinear control parameters to consider the pinching effects and the degradation of a system:

1. Post-yielding stiffness ratio $\alpha$
   
   $|\alpha| \leq 1$, and when $\alpha = 1$ that indicates an elastic system, if $\alpha = 0$ that indicates an Elastic-Perfect-Plastic model (EPP model).

2. Stiffness degradation control parameter $\lambda_k$
   
   $\lambda_k \geq 0$, and when $\lambda_k = 0$ indicates no stiffness degradation effect.

3. Load deterioration control parameter $\lambda_l$
\( \lambda_L \geq 0 \), and when \( \lambda_L = 0 \) indicates no load deterioration effect.

4. Pinching control parameter \( \lambda_p \)

\( 0 \leq \lambda_p \leq 1 \), and when \( \lambda_p = 1 \) indicates no pinching effect.

One of the principal advantages of the proposed formulation is that the resistance-deformation relations are characterized based on physical quantities. Fours nonlinear control parameters have definitely physical meanings, which is defined as “physically based analytical model”. Fig. 2 shows that the individual effects of these four nonlinear control parameters on restoring force diagram. This inelastic model will be implemented in multiple degree of freedom system to calculate the seismic demand of structure.

![Figure 2: The effects of four nonlinear model parameters on restoring force diagram.](image)

**Model Structures** For the sake of realizing the seismic performances of RC structures, three different kinds of typical reinforced concrete structures in Taiwan are chosen. The first one is 4-story RC building and the second one is 6-story RC building. Fig. 3a shows the plan of the 4-story RC building and the 6-story building is the same in plan. The third one is 12-story RC building as shown in Fig. 3b. The specified compressive strength of concrete \( \left( f_c' \right) \) and yield strength of reinforcement \( \left( f_y \right) \) are assumed to be 210 kg/cm\(^2\) and 4200 kg/cm\(^2\), respectively. Structural analysis for member design is carried out by using the commercial software ETABS 6.2. All of them are designed by means of ultimate strength method and are fitted in with Taiwan building codes by Liao (2000[11],2003[12]). In order to simplify the
complicated three-dimensional model and save the calculating time, the full-scaled MDOF stick model with inelastic hysteretic behavior was selected in this study. The natural frequencies and mode shapes between the stick model and the ETABS model are adjusted to have the same values. Two different types of frame structures are selected: the pure flexure frame model and wall frame model. Table 1 shows the comparison on the structural periods between the modified stick model and the ETABS model. The hysteretic models of these two model structures are developed using cyclic loading test of frame structures with and without wall. Fig.4 shows the comparison on restoring force diagram between simulation and test results for pure frame and wall frame structures.

Table 1: Comparisons of structural periods between the modified stick model and ETABS model

(a) 4-story building

<table>
<thead>
<tr>
<th></th>
<th>Modified Stick Model</th>
<th>ETABS Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>0.888</td>
<td>0.892</td>
</tr>
<tr>
<td>2nd</td>
<td>0.277</td>
<td>0.327</td>
</tr>
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</table>

(b) 6-story building

<table>
<thead>
<tr>
<th></th>
<th>Modified Stick Model</th>
<th>ETABS Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>1.456</td>
<td>1.458</td>
</tr>
<tr>
<td>2nd</td>
<td>0.468</td>
<td>0.506</td>
</tr>
<tr>
<td>3rd</td>
<td>0.264</td>
<td>0.312</td>
</tr>
</tbody>
</table>

(c) 12-story building

<table>
<thead>
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<th></th>
<th>Modified Stick Model</th>
<th>ETABS Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>1.173</td>
<td>1.176</td>
</tr>
<tr>
<td>2nd</td>
<td>0.364</td>
<td>0.426</td>
</tr>
<tr>
<td>3rd</td>
<td>0.195</td>
<td>0.273</td>
</tr>
</tbody>
</table>

SEISMIC DEMANDS

Quantification of seismic demands for performance assessment implies the statistical evaluation of Engineering Demand Parameters (EDP) as a function of ground motion Intensity Measures (IM) for different structural characteristics. The seismic demands in this study include: deformation demands, strength demands and energy demands.

**Deformation Demands** Both roof and inter-story displacement demands are considered in this study. For MDOF systems, physically, roof drifts estimates are necessary in order to determine the minimum separation between adjacent buildings to prevent pounding. It is suggested that the normalized parameter, maximum roof drift angle, which is defined as the ratio of maximal roof
displacement to the height of the building, \( \theta_{r, \text{max}} = \frac{S_{\text{roof}, \text{max}}}{H} \), is a useful and important index.

Since inter-story drift is a major contributor to both structural and non-structural failure, especially for non-structural components, then it is strongly recommended that the normalized parameter, maximum inter-story drift angle, which is defined as \( \theta_{s, \text{max}} = \frac{S_{s, \text{max}}}{h} \) is a useful and important index.

![Figure 3](image1.png)

Figure 3: Plan view of typical reinforce concrete building structure; (a) 4-story & 6-story building, (b) 12-story building.

<table>
<thead>
<tr>
<th>Pure Frame Model</th>
<th>Wall Frame Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_{1F} ) (tons/cm)</td>
<td>113.31</td>
</tr>
<tr>
<td>( K_{2F} )</td>
<td>68.182</td>
</tr>
<tr>
<td>( K_{3F} )</td>
<td>61.614</td>
</tr>
<tr>
<td>( K_{RF} )</td>
<td>53.022</td>
</tr>
<tr>
<td>( T_{\text{first mode}} ) (sec)</td>
<td>0.89067</td>
</tr>
<tr>
<td>( K_{1F} ) (tons/cm)</td>
<td>540.54</td>
</tr>
<tr>
<td>( K_{2F} )</td>
<td>258.62</td>
</tr>
<tr>
<td>( K_{3F} )</td>
<td>180.18</td>
</tr>
<tr>
<td>( K_{RF} )</td>
<td>109.89</td>
</tr>
<tr>
<td>( T_{\text{first mode}} ) (sec)</td>
<td>0.48064</td>
</tr>
</tbody>
</table>

![Figure 4](image2.png)

Figure 4: Comparison between the simulation model and test results on restoring force diagram for: (a) pure frame structure, (b) wall frame structure.
Empirically, the reinforced concrete structure is in the scope of elastic behavior if inter-story drift angle is less than 0.5%. In the case of that the inter-story drift is over 2%~3% it will subject to severe damage and causing collapse in structures.

**Strength Demands** Both base shear demands, story shear demands and story overturning moment demands are considered in this strength demands.

**Energy Demands** To study the damage index energy demands are required. In this study hysteretic energy (HE) demand, distribution of HE over height, and normalized hysteretic energy. Hysteretic energy demand quantifies the fraction of input energy dissipated by yielding of the elements of a structure, therefore the first energy demand is the HE demand.

The concept of a story hysteretic energy demand, \( (HE)_{i,j} \), is computed by integrating the story shear with respect to the inter-story drift for each floor. For the statistical analyses the story hysteretic energy is normalized by the hysteretic energy demand for the complete MDOF systems, \( HE_{(mdof)} \). The ratio \( (HE)_{i,j} / HE_{(mdof)} = (HE)_{i,j} / \Sigma (HE)_{i,j} \) can be interpreted as a measure of relative damage between different stories. Normalized hysteretic energy is the basic parameter in the cumulative damage model expressed as bellow (Krawinkler and Zohrei 1983).

\[
(NHE)_{i,j} = \frac{(HE)_{i,j}}{V_y \delta_y}
\]

where \( V_y \) and \( \delta_y \) are the yield story shear and the yield story drift, respectively.

**PERFORMANCE ASSESSMENT OF FRAME STRUCTURES**

This section summarizes studies related to the non-linear response of MDOF system in which the lateral resistance is provided by moment resisting RC frame and RC wall frame. These studies provide valuable quantitative information related to the seismic demands and capacities of this structure.

**Input Strong Ground Motions** Ground motion data collected from hard site condition are selected for the inelastic response analysis. In performance-based seismic design, a design performance objective couples an expected or desired performance level with a level of possible seismic hazards. For the sake of comparing different seismic intensity, 3 hazard levels of design spectra are constructed as target spectra. The exceeding probabilities in 50 years are specified as 50%, 10%, and 2%, respectively. Three groups of excitation level, each composed of 10 earthquakes, are selected separately and scaled so that their acceleration spectra match the 3 target spectra. The selected ground motion data was normalized to the target spectrum in the range between 0.5 sec and 2.0 sec of structural period. Twenty
Seismic Demands of Frame structures  Two approaches to evaluate the seismic demands for inelastic MDOF systems are discussed. The first one approach is to estimate seismic drift demand from the elastic response spectra. Fig.6a shows the mean and mean+1σ normalized maximum roof drift spectrum for pure moment-resistant frame, and Fig.6b shows the spectrum of the mean and mean+1σ maximum story drift angle ratio (with respect to the maximum roof drift angle) over the height. According to the information provided in Figures 6a and 6b, together with the elastic response spectra one can easily evaluate the seismic drift demands which are relative to both structural and non-structural damage in the inelastic MDOF systems. The second approach is for conceptual design in performance-based design. It start with at determine the design ductility and then by multiplying several modification factors to predict the seismic responses of the inelastic MDOF systems. The following relationships between MDOF reinforced concrete frames and their corresponding SDOF systems have been established and can be utilized to facilitate the design process.

Global Deformation Demands – Roof Displacement  There are several stages within the design methodology in which modifications are needed to convert the available SDOF results to real MDOF systems. The need to predict global and inter-story drift in the seismic design of frame structures is important. Based on the results of the dynamic analyses a two step procedure was developed to estimate the roof displacement demands of elastic/inelastic frame structures, as shown in Fig.1. In the first step the roof displacement of an inelastic SDOF frame structure $\delta_{in,SDOF}$ is related to the elastic first mode spectral displacement $S_d(T_1)$
by a parameter $\alpha_1$. In the second step the roof displacement of the inelastic frame structure is related to the roof displacement of an inelastic SDOF frame structure by a parameter $\alpha_2$. A statistical evaluation of the results of the dynamic analysis results was used to quantify the parameters $\alpha_1$ and $\alpha_2$. The results of this evaluation are illustrated in Fig. 7 for pure frame structure. The inelastic roof drift of a MDOF frame structure can be estimated from the elastic displacement spectra.

Figure 6: (a) Medium curve of normalized maximum roof drift for pure frame and wall frame structures, (b) Medium curve of the ratio of the maximum story drift angle to the maximum roof drift angle for pure frame and wall frame structure.

**Global Deformation Demands – Ductility Demands** The ratio of maximum story ductility demand ($\mu_{s,\text{max}}$) over the height is examined. Fig. 8 shows the distribution of maximum inter-story ductility of pure frame structure at different story height for different level of ground motion excitation (50%/50 years, 10%/50 year, and 2%/50 years). It is found that the maximum story ductility is observed at the first floor for the 12-story frame structure while for the 6-story frame structure the maximum ductility ratio was observed at 2nd floor. In order to limit the story ductility demand of the MDOF systems to the target ductility of the first mode SDOF system,
the design base shear of the MDOF system \( (V_{y(MDOF)}) \) has to be greater than the strength demand of the SDOF system \( F_y \). The design base shear of the MDOF system, \( V_{base,MDOF} \), is also examined. The modification factor \( \beta_1 \), defined as the ratio between the base shear of an inelastic SDOF system and the yield force \( F_{y,SDOF} \) and the modification factor \( \beta_2 \), defined as the ratio between \( V_{base,MDOF} \) and the base shear of an inelastic SDOF system, are also examined. Figure 9 shows the comparisons on maximal story shear demands using different sets of ground motion data with 50%/50yr, 10%/50yr and 2%/50yr input levels. The modification factor for pure frame structure is shown in Fig. 11. The design implication of the results presented in Fig. 11 can be illustrated on a simple example. If a pure frame structure with a fundamental period of 1.17 sec can tolerate a ductility of 4.8, the base shear strength needs to be increased by a factor of approximately 1.8, compare to the inelastic strength demands of the corresponding SDOF system with the same period.

**Energy Demands** For the analytical model considered in this study the total HE dissipated in the structure can be computed as the sum of hysteretic energy dissipated at each floor. Fig. 10 shows the normalized story hysteretic energy envelopes demands using rock site earthquake records for high hazard level (a)4-story (b)6-story (c)12-story RC buildings. Fig.12 shows the ratio of hysteretic energy dissipated in a MDOF frame structure to that dissipated in the first mode SDOF system with corresponding target ductility ratio. This figure illustrates the effect of higher modes on the HE demands. The HE demand ratio for pure frame structure at a given value of \( \mu(SDOF) \) remains fairly constant in the short period range since hysteretic energy is dissipated only in the first story. For longer period structure the HE demand ratio is getting larger than in the short period structure. Similar result is observed for
Figure 8: Plot of mean and mean+1σ maximum inter-story drift angle & story ductility demands for different intensity level of ground excitation.
Figure 9: Comparisons on maximal story shear demands using different sets of ground motion data with 50%/50yr, 10%/50yr and 2%/50yr input levels.
Figure 10: Story hysteretic energy envelopes demands using rock site EQ. records for high hazard level (a)4-story (b)6-story (c)12-story RC buildings.
The current performance evaluation method is based on one or multiple structural response indices including maximum drift ratio (or, ductility) and cumulative inelastic energy dissipation, which are known to be able to fully characterize performance levels for systems where the main concern is to avoid collapse, but are unable to characterize the performance level of some modern structural systems (e.g., self centering frame buildings) where the structural integrity is not at risk during seismic attack. As such, an independent scale of
residual-deformation based performance measure can be effectively combined with the existing performance measure based on maximum response (or, cumulative damage) to form a more general performance domain. A model with post-peak negative stiffness behavior and residual deformation will be introduced in this section. Based on the theory-based piecewise linear hysteretic model, proposed by Mostaghel (1999), a revised theory-based piecewise linear model is proposed. This model can be used to define constitutive relationship between stress and strain, force and displacement, moment and curvature, or moment and rotation, depending on the applications as long as the quantity and quality of experimental results are sufficient for the determination of the values of key model parameters.

Consider a SDOF hysteretic system, as shown in Fig.13., which contains one linear spring, the deformation of which is represented by \( x \), and a slider-spring element with a frictional surface of variable Coulomb damping coefficient such that the slider-spring will start to slip at a certain force level (e.g., \( k \cdot u \) at first yield). The slip may accelerate when the friction coefficient reduces to a lower level because of a decrease in the interlocking force between internal particles of the system, which, in the case of RC columns, physically means major shear cracks have fully developed. Such post-peak behavior is commonly observed in low-confinement RC columns, pre-Northridge steel connections, and wood shear wall system. To describe such a phenomenon, unknown \( u \) representing the deformation of the spring connected to the slider can be expressed as the following mathematical form (Wu & Loh, 2004):

\[
\bar{\delta} = \begin{bmatrix}
\phi_i \cdot M \left( u - \lambda_p \cdot \phi_i \cdot \delta^i \right) \cdot \bar{N} \left( x \right) + \phi_i \cdot M \left( u - \phi_i \cdot \delta^i \right) \cdot \bar{N} \left( x \right) \\
\left( -\phi_i - \phi_i \right) \cdot \bar{N} \left( x - \delta^i \right) \cdot M \left( u \right) \cdot N \left( x - \delta_{FS} \right) \\
\left( -\phi_i - \phi_i \right) \cdot \bar{N} \left( x - \delta^i \right) \cdot N \left( u \right) \cdot N \left( x - \delta_{FS} \right)
\end{bmatrix}
\]

\[u = H(x) \cdot \bar{N} \left( x \right) \cdot \left( \phi_i \cdot M \left( u - \lambda_p \cdot \phi_i \cdot \delta^i \right) \cdot \bar{N} \left( x \right) + \phi_i \cdot M \left( u - \phi_i \cdot \delta^i \right) \cdot \bar{N} \left( x \right) \\
+ \left( -\phi_i - \phi_i \right) \cdot M \left( x - \delta^i \right) \cdot N \left( x - \delta^i \right) \cdot M \left( u \right) \cdot N \left( x - \delta^i \right) \cdot \bar{N} \left( x \right) \\
+ \left( -\phi_i - \phi_i \right) \cdot M \left( x - \delta^i \right) \cdot \bar{N} \left( x \right) \cdot N \left( x - \delta^i \right) \cdot \bar{N} \left( x \right)
\]

and \( N(x) = H(x) \); \( M(x) = 1 - H(x) \); \( \bar{N}(x) = 1 - H(-x) \); \( \bar{M}(x) = H(-x) \), and \( H(x) \) is the Heaviside’s unit step function. The constant \( \lambda_p \), which takes a value between 0 and 1, denotes the resistance ratio and represents pinching of a hysteretic loop due to unequal strengths. There is no pinching in a structural component if \( \lambda_p \) is assumed to be 1. \( \delta^i \) is the yield displacement of the system; \( \delta^i \) is the displacement at which the system reaches its ultimate strength and the friction coefficient of the slider will start to decrease; \( \delta_r \) is the displacement at which the system reaches its residual strength; \( \delta_{FS} \) defines the failure surface in Fig.13 and represents the corresponding deformation of the linear spring when the
slider-spring reaches a certain deformation at a time step. The positive and negative signs in the superscript indicate asymmetric material property may exist under compression and tension. The failure surface may migrate to a lower level of material strength according to a prescribed flow rule when dissipated hysteretic energy accumulates with time. In addition,

\[
\phi_1 = \frac{1}{1 + \lambda h(t)}
\]

Load-deterioration function

\[
\phi_2 = \frac{1}{1 + \lambda h(t)}
\]

Post-peak stiffness function

\[
\phi_3 = \frac{(1 - \rho) \delta_r}{\delta_r - \delta_s}
\]

And, \( h(t) \) denotes the hysteretic energy absorbed by the system. Fig.14 demonstrates several numerical hysteretic models having negative post-peak stiffness in the 1st row and relevant experimental results showing the same characteristics are given in the 2nd row: (1) Bilinear hysteresis with capping and a welded haunch steel moment connection taken from Uang et al. (2000 [13]), (2) Pinching with collapse and a concrete shear wall taken from Oh et al. (2002 [14]), (3) Pinching with collapse and a low-ductility RC column taken from Elwood (2002 [15]). To quantify seismic demands on structural systems for use in conceptual design it is necessary to incorporate this model and develop a residual-deformation based performance measure.

Figure. 13: (a) Schematic representation of SDOF hysteretic system with consideration of post-peak behavior, (b) Schematic representation of failure surface.

**SHAKE TABLE TEST ON RC-FRAME WITH LOW-DUCTILITY COLUMNS**

A series of shake table tests are desirable in order to validate the proposed piecewise linear hysteretic model, and experimental results could be very supportive in determination of key parameter values. To investigate how the 4-story commercial-resident complexes sustained severe damage and some of them even collapsed in the 1999 Chi-Chi earthquake, a \( \frac{1}{2} \)-scale RC frame composed of 2 low-ductility columns inter-connected by a strong beam (as shown in Fig.15) was tested on the shake table earlier in May 2004. The NS component of TCU082
accelerogram recorded during the 1999 Chi-Chi earthquake was used to excite the frame specimen of a 0.35sec natural period, representing a 0.49sec commercial-resident complex at full-scale. The specimen was subjected to a sequence of TCU082 records, scaled to 0.63g and 1.16g, respectively, to obtain hysteretic behavior of the specimen. The hysteretic loops are shown in Fig. 16. Both bending cracks and minor shear cracks were observed during the first test. The post-peak negative stiffness can also be observed from the hysteresis loop of the first test. After the first test the second test cause the specimen to collapse. These test data are expected to yield helpful information such that considerable improvements can be achieved on numerical dynamic analysis of structural collapse in the near future.

CONCLUSIONS

The objective of this research discussed before is to develop procedures to quantify seismic demands on MDOF systems for use in conceptual design. Two types of lateral resisting systems of reinforced concrete structures are investigated in this study: pure frame model and wall frame model. Based on the nonlinear hysteretic model proposed by Mostaghel (1999, 2000) These two frame structures were modeled by this physical-based inelastic model. The parameters for evaluating the seismic drift demands of MDOF systems and the relationships between MDOF seismic demands and corresponding SDOF spectral demands are estimated from statistical analyses of dynamic response results obtained by subjecting these generic
structures to an ensemble of recorded ground motions. The main conclusions derived from the parametric study of the frame structures are summarized as follows:

• The maximum story drift angle can be estimated using the root drift information and the amplification factor increase with the increase of structural period.
• Because of higher mode effects, the base shear demand on either pure frame or wall frame structure is higher than the design base shear of SDOF system. This effect is more significant for wall frame structure.

In order to incorporate the residual deformation and collapse behavior a revised hysteretic
model was introduced. With the aid of almost full-scale shaking table test of frame structure to collapse, a more realistic model can be developed.

ACKNOWLEDGEMENTS

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Part 2

**Prediction of Equivalent Linearization Method Considering SSI Effect on Seismic Demand of Structures**

**Abstract**

The purpose of this paper is to estimate the yield strength and maximum inelastic deformation of a structural system based on equivalent linearization method. Except the consideration of seismic response of inelastic system the effect of soil-structure interaction on the nonlinear response of structures is also investigated. By using the sophistic Bouc-Wen force-deformation hysteretic model the maximum deformation of a SDOF nonlinear system is estimated. Analysis of the inelastic seismic demands to include the effect of Soil-Structure Interaction (SSI) on the nonlinear behavior of structures is also investigated through a comprehensive parametric study over a wide range of representative non-dimensional parameters of SSI system. Discussion on the estimated equivalent viscous damping ratio for system with flexure-type hysteretic model and with soil-structure interaction effect was made. Finally, rational evaluations of seismic demand placed on the SSI system to displacement-based seismic design of building structure is introduced.

**Keywords**: equivalent linear SDOF system, inelastic response spectrum, ductility demand, soil-structure Interaction,

**Introduction**

In performance-based seismic design of engineering building the most important issue is to develop simple and effective methods for designing, analyzing and checking of structures. These analytical procedures for displacement-based design must be able to predict the seismic demands of structures, i.e. forces and deformations. Generally, the displacement-based design approach aims to design a structural system for a prescribed target displacement for a given earthquake motion characterized by linear response spectra generated for various levels of viscous damping. Therefore the equivalent viscous damping concept is an important component of displacement-based earthquake engineering design procedures (Priestley and
Kowalsky, 2000). The displacement-based design involves several steps and one of which is to estimate the seismic deformation of an inelastic SDOF system representing the first mode of vibration of the MDOF system, and the response of the inelastic SDOF system is accomplished by approximate methods in which the nonlinear system is replaced by an “equivalent” linear system. Therefore, the evaluation of available approximate methods to estimate maximum inelastic displacement demands of SDOF systems from maximum displacement demands of elastic SDOF systems is especially valuable for the displacement-based design procedures.

It is well known that due to the effect of soil-structure interaction the response of a structure supported on soil may be different from that of the identical structure on fixed-based state. To study the seismic demand in the displacement-based design the effect of soil-structural interaction must be taken into consideration. The principal effect of the soil-structural interaction is to increase the natural period of the structure and, usually, to increase its effective damping ratio. Therefore, the SSI can be thought to affect the base shear demand. However, the SSI effect on the response of nonlinear structures has attracted less attention. The available regulations also don’t consider the effect of SSI on the nonlinear structural behavior. It is believed that the interaction results from the flexibility of the soil-foundation system (inertial interaction) and the difference in stiffness between the foundation and the surrounding soil (kinetic interaction) may have significant impact on the ductility demand of structures on the examination of the seismic demand of building (Ciampoli, et al. 1995; Kabeyasawa, et al. 2000). In this study the behavior of nonlinear structure with SSI is investigated for a wide range of representative non-dimensional parameters.

Based on the above statement, the purpose of this paper is to assess the relevance of soil-structure interaction (SSI) effects on the dynamic response of structure responding in the inelastic range and, in particular, on the seismic demand of the superstructure with and without consideration of SSI effect. Finally, the applicability of equivalent linearization method is considered as one of the simplified methods to estimate the seismic demand.

**Force-Deformation Hysteretic Model**

Generally, the elastic-perfectly plastic (EPP) model was used to represent the hysteretic behavior of inelastic system and was used to investigate the structural inelastic response when subjected to earthquake ground motions. The EPP force-deformation hysteretic model is characterized by having a constant loading stiffness up to yielding which reaches to a yielding strength $F_y$, which is assumed to be equal in both loading directions, and then after yielding the system has no lateral stiffness and the unloading stiffness is equal to the loading stiffness. Only two parameters are needed to characterize the EPP model, the initial stiffness $K_e$ and the yielding strength $F_y$. Besides, many hysteretic models have been proposed to represent the
force-deformation characteristics of reinforced concrete structure when subjected to reverse
cyclic loading. One of the models to include the effect of stiffness degradation, strength
deterioration and pinching was proposed by Bouc and Wen (19 ). The form of hysteretic
restoring force $z$ is formulated as (Baber & Noori, 1985; Foliente 1995):

$$
\dot{z} = \frac{A \dot{\delta} - V(\beta |\dot{\delta}| z^{(n-1)} z + \eta |\dot{\delta}| z^{\nu})}{\eta[1 + af(z)[A - V(\beta |\dot{\delta}| z^{(n-1)} z / \dot{\delta}^{\nu} + r |z|])]} \quad (1)
$$

This Bouc-Wen hysteretic model is capable of representing a wide variety of hysteretic,
deteriorating types of behavior concerning cyclic energy dissipation and displacement. The
model parameters $\beta, \gamma$ and $n$ determine the hysteretic shape and $A$ the tangent stiffness.
The parameters $\nu$ and $\eta$ are factors that control system deterioration. The strength
degrading parameter $\nu$ is a function of system energy dissipation, $\varepsilon(t)$, where

$$
\nu = 1 + \delta, e(t) \quad (2)
$$

$$
\varepsilon(t) = \int zdu \quad (3)
$$
in which $\delta$ is the strength degrading control parameter. The stiffness decay parameter $\eta$ is a
function of system deformation.

$$
\eta = 1 + s, |u_{max}|^p + |u|^2 \quad (4)
$$

where $S$ and $p$ are the control parameters which represent the variation of the stiffness decay
rate. The pinching effect in the hysteretic loop is added by incorporating a time-dependent
“slip-lock” element as shown schematically in Figure 1. The slip-lock element behaves quite
similar to a hardening nonlinear spring with the special characteristics being nearly zero
stiffness in the “slip” zone and infinite stiffness in the “locking” zone. The equation for
“slip-lock” behavior is expressed as followed:

$$
a = A_s |u|^p \quad (5)
$$

$$
f(z) = e^{-|z|sgn(u) - \tau/\zeta} \quad (6)
$$
in which sgn is the signum function. The parameter $z_s$ controls the sharpness of the slip, and
slip is symmetric about $z=0$. The parameter $\zeta$ controls and shifts the effective slip region to
be symmetric about an arbitrary. The slip length $a$ is assumed to be a function of the
relative displacement $u$, where $A_s$ and $r$ are control parameters to modify the slip length,
and linked to the size of crack openings.

In order to properly represent the restoring force behavior of an actual structure, it is
necessary to estimate the appropriate modal parameters of the proposed hysteretic model. This
includes the estimation of model parameters, such as: $A, \beta, \gamma, n, \delta, \ldots$ etc., from experimental
data. Many experiments on the study of the capacity of reinforced concrete columns were conducted. Cyclic loading tests had been carried out on the reinforced-concrete columns to observe the capacity of columns, and data collected from the cyclic loading test of structural members can provide valuable information to the estimation of model parameters. To identify the model parameters, a systematic two-stage system identification method was proposed (Loh & Chung, 1993). Based on the proposed modified Bouc-Wen model with the consideration of both strength deterioration, stiffness degradation and pinching effects, the model parameters were identified. Figure 2 shows the hysteretic behavior of the specific column (flexible-type failure) subjected to cyclic loading tests (BMR1-Model). A comparison of experimental and simulation results of RC column hysteretic behavior is also shown. A good agreement between the proposed model and the experimental results was observed.

The nonlinear system to be considered herein is a single degree of freedom (SDOF) oscillator, as shown in Figure 3a, subjected to ground motion, $\mathbf{g}$. The governing equation for the oscillator is expressed as followed:

$$m\ddot{u}(t) + c\dot{u}(t) + \alpha ku(t) + (1 - \alpha)kz(t) = -m\ddot{\mathbf{g}}(t)$$

in which $m$, $k$, $c$ represent mass, stiffness and viscous damping of a SDOF system, respectively. $\alpha$ represents the ratio of post-yield/pre-yield stiffness and $z(t)$ is the hysteretic restoring force. Time domain analysis of the nonlinear system can be conducted through numerical study. Figure 3a represents the inelastic SDOF system with fixed base and Figure 3b represents the SSI system which will be discussed later on.

**Study on the Equivalent Damping Ratio**

The seismic response of an inelastic system, as described in Eq.(7), can be replaced by an “equivalent” linear system which is defined (Iwan et al. 1979):

$$m\ddot{u}(t) + c_{eff}\dot{u}(t) + k_{eff}u(t) = -m\ddot{\mathbf{g}}(t)$$

where $k_{eff} = mw_{eff}^2$ ; $c_{eff} = 2mw_{eff}\xi_{eff}$. The most common method for defining the equivalent viscous damping is to equate the energy dissipated in a vibration cycle of the inelastic system and of the equivalent linear system. Consider an inelastic SDOF system with bilinear force-deformation relationship on initial loading, the equivalent viscous damping ratio is proposed as (Chopra and Goel, 1999):

$$\xi_{hy} = \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)}$$

The total viscous damping ratio of the equivalent linear system is
\[ \tilde{\xi}_{\text{eff}} = \xi + \tilde{\xi}_{\text{hy}} \]  \hspace{1cm} (10)

where \( \tilde{\xi} \) = viscous damping ratio of the inelastic vibrating within its linear elastic range \((u \leq u_y)\). Kowalsky also proposed an equivalent viscous damping ratio using the Takeda hysteretic model and expressed (Priestley & Kowalsky, 2000):

\[
\tilde{\xi}_{\text{eff}} = 0.05 + \left[ 1 - 0.95 \frac{1}{\sqrt{\mu}} - 0.05 \sqrt{\mu} \right] \frac{\pi}{\mu}
\]  \hspace{1cm} (11)

In this study, on the estimation of equivalent damping ratio, it is defined that the total input energy is absorbed by equivalent viscous damping in the SDOF system with the equivalent stiffness at the point of maximum response. The equivalent damping ratio is then defined as (Teshigawara et al., 2000):

\[
\tilde{\xi}_{\text{eff}} = -\frac{\int_0^t \ddot{u} \dddot{u} \, dt}{2\omega_{\text{eff}} \int_0^t \ddot{u} \, dt}
\]  \hspace{1cm} (12)

where \( \dddot{u} \) is the input ground acceleration, \( t \) is the duration time, and \( \omega_{\text{eff}} = 2\pi/T_{\text{eff}} \). The equivalent period of the linear system is defined as:

\[
T_{\text{eff}} = T \sqrt{\mu}
\]  \hspace{1cm} (13)

This definition of the period of the equivalent linear system is the same as been used in Kowalsky’s method. Comparison between the inelastic response spectrum with the elastic response spectrum using equivalent linear system is examined.

In this study the Bouc-Wen hysteretic model is used in the inelastic response analysis. A set of 6 acceleration time histories recorded at hard site condition and normalized to 0.32g was used. Nonlinear dynamic analysis was conducted to verify the accuracy of using equivalent linear system to represent the inelastic system. Since the yield force of the Bouc-Wen hysteretic model is a function of model parameters (i.e. \( A, \beta, \gamma \) and \( n \)) and can be expresses as:

\[
F_y = k \left( \frac{A}{\beta + \gamma} \right)^{\gamma_n}
\]  \hspace{1cm} (14)

Then in the time history analysis, with a specified structural stiffness (or structural period), one can adjust \( A, \beta, \) and \( \gamma \) (keep \( n \) unchanged) and determine \( F_y \) when a specified ductility ratio was identified. To evaluate the accuracy of the proposed equivalent linear system comparison between the inelastic time history response analysis and the approximate method using equivalent linear system must be examined when subjected to the same earthquake ground motion. \textbf{Figures 4a and 4b} (solid line) shows the comparison on the estimated
normalized yield base shear force as well as the estimated maximum displacement with respect to the structural period for ductility ratio equals 2.0 and 4.0, respectively. The prediction of seismic demands, including forces and deformations, is quite accuracy and reliable by using the proposed model.

By using the responses from the inelastic time history response analysis and calculate the equivalent viscous damping ratio (Eq.12) and the effective structural period (Eq.13), for a specified system ductility ratio, the relationship between effective damping ratio and structural period can also be examined. This effective damping ratio spectrum (which plots the estimated effective damping ratio with respect to effective structural period $T_{eff}$) can also be examined. It is found that the calculated effective damping ratio is almost independent to different value of effective structural period, as shown in Figure 5a. Is is also observed that the relationship between effective damping ratio and system ductility is almost independent to structural period, as shown in Figure 5b. Comparison on the estimated effective damping ratio using Equation (12) and the model proposed by Chopra, and Kowalsky is shown in Figure 6. It is found that the proposed method on the estimation of equivalent damping ratio (using BMR-1 hysteretic model) is much close to the result of Kowalsky method.

**Effects of Soil-Structure Interaction on the Ductility Demand of Structures**

Soil-structure interaction (SSI) effects consist of the difference in the structural response evaluated assuming an ideal rigid foundation and the actual soil foundation, respectively. To examine the seismic demand of building it is necessary to investigate the effects of soil-structure interaction (SSI) on the inelastic seismic response of building-foundation systems. However, the current seismic design regulation did not consider the effect of SSI on the nonlinear structural behavior. Ghannard & Ahmadnia (2001) had studied the effect of SSI on ductility demand. More detail examination on the effect of soil on the seismic demand of structures using more realistic structural model needs to be conducted.

Formulated thus, the building-foundation system has three significant degrees of freedom, namely, horizontal translation of the top mass, horizontal translation of the base mass, and rotation of the system in plate of motion. The structural base is assumed to be a rigid plate of radius $r$ and negligible thickness, and no slippage is allowed between the base and the soil. Figure 3b shows the model of building-foundation system. The equations of motion which show the coupling effect of horizontal translation and rocking are expressed as follows (Rodriguez et al. 2000):

\[
m\ddot{u} + c\dot{u} + k_{z}(u) = 0 \quad (15a)
\]

\[
m\ddot{\phi} + c_{h}\dot{\phi} + k_{h}u_{o} = 0 \quad (15b)
\]
From equation (15a), the building structure consists of an inelastic restoring force (represented by Bouc-Wen hysteretic model), $k_z(u)$, and with viscously damped single degree of freedom system, resting on a viscoelastic half-space with density $\rho$, shear modulus $G$, and Poisson’s ratio $\nu$. The inelastic hysteretic restoring force of the super-structure is the same as shown in Figure 2 where $u$ is the horizontal displacement of the top mass relative to the base, excluding rotations; $u_0$ the translation of base mass relative to the free-field motion; $\phi$, the rotation of base mass; and $u_t$ the total horizontal displacement of top mass with respect to a fixed vertical axis, i.e. $u_t = u_s + u + h\phi + u_0$. $k_h$ and $k_\phi$ are frequency-dependent dynamic stiffness of translation and rocking of the foundation medium at the interface with the structural base. These terms can be interpreted as the stiffness coefficients of equivalent frequency-dependent springs. $c_h$ and $c_\phi$ are frequency-dependent damping of translation and rocking of the foundation. It accounts for the effect of energy dissipated by radiation and internal friction of the soil material. Properties of these elements depend on the system characteristics and the exciting frequency. The equivalent horizontal and rocking springs appearing in (15b) and (15c) are expressed as (Tsicnias et al. 1984)

$$
k_h = K_h \cdot k_h^\nu (\omega_0, \nu), \quad \text{and} \quad k_\phi = K_\phi \cdot k_\phi^\nu (\omega_0, \nu)
$$

and damping coefficient of the horizontal and rocking dashpots are expressed as

$$
c_h = (K_h r / V_s) \cdot c_h^\nu (\omega_0, \nu), \quad \text{and} \quad c_\phi = (K_\phi r / V_s) \cdot c_\phi^\nu (\omega_0, \nu)
$$

where $V_s = \sqrt{G / \rho}$ is the shear wave velocity of the soil medium, $K_h (=8V_s^2 \rho r/(2-\nu))$ and $K_\phi (=8V_s^2 \rho r^3 / 3(1-\nu))$ are the static horizontal and rocking stiffness of soil, and $k_h^\nu, k_\phi^\nu, c_h^\nu$ and $c_\phi^\nu$ are dimensionless coefficients depending on the Poisson’s ratio and of the dimensionless frequency parameter $\omega_0 = \omega h / V_s$. To simplify the analysis, in this study the soil spring and damping values are assumed as frequency independent. These coefficients can be obtained from Reference 14. Coupling between the horizontal and rocking components of the base dynamic stiffness generally is small and is neglected in the present formulation. This soil-structure model is then analyzed through direct step-by-step integration in time domain.

Basically, the response of soil-structure system depends on the dimension of the structure, dynamic properties of the structure and the soil, and the applied excitation. The effect of these factors can be best described by the following non-dimensional parameters:
(1) A non-dimensional frequency, \( a_0 \)

\[
a_0 = \frac{\omega h}{V_s}
\]  

(18)

where \( \omega \) is the circular frequency of the fixed-base structure.

(2) Aspect ratio of the building

\[
h_r = h/r
\]  

(19)

(3) The ratio of structure mass to soil mass, \( M_r \) (\( M_r=1.0 \) is used in this study)

\[
M_r = \frac{m}{\rho r^2 h}
\]  

(20)

(4) Poisson’s ratio of the soil \( \nu \) (\( \nu=0.33 \) is used in this study)

Sensitivity study of these parameters on the estimation of seismic demands will be examined in the following section.

**Effect of SSI on the Nonlinear Behavior of Structures**

The effect of SSI may induce the result of three simultaneous effects on structural system: (1) the change of the system natural period, (2) the modification of the damping of the structural system, and (3) the change in the ductility demand of the structure as a part of soil-structure system. The SSI effect on the ductility and strength demands of structure is investigated in this section.

**Strength Demand** The strength demand is defined as the required yield force \( F_y \) in a structure for a giving specific ductility. As the soil influences the structural responses, the strength demand of the soil-structure system would be different from that of the fixed-base structure. In this regards, models with the aspect ratio \( h_r = 1 \) and 3, have been selected as the representatives of the building. Also, the non-dimensional frequency \( a_0 =1 \) and 2 are assigned for the cases of buildings located on stiff and soft soils, respectively. Figure 7 shows the comparison on elastic strength spectrum for system with and without considering SSI for case of \( a_0=1.0 \) and 2.0 and with \( h_r=1.0 \) and \( h_r=3.0 \), respectively. For the examination of the elastic seismic demand it is found that for system with SSI the elastic strength spectrum \( (F_y/W) \) is smaller than the system without SSI effect. But the effect of SSI on the elastic strength spectrum becomes insignificant when the \( h_r \) becomes larger and \( a_0 \) becomes smaller. It means that for structure on stiffer site (smaller \( a_0 \)) and larger aspect ratio the SSI effect is not important if the structural system is linear. The inelastic yield strength demand spectrum normalized by the weight of the structure with and without consideration of SSI effect is shown in Figure 8. From the observation of Figure 8 several conclusions can be made:

a. Generally, the inelastic yield strength spectrum with the consideration of SSI effect is
larger than the spectrum without considering SSI (fixed base).

b. For SSI system on the stiff soil \((a_0=2.0)\) with small aspect ratio \((h_r=1.0)\) the inelastic yield strength spectrum is very close to the case of fixed-base inelastic spectrum, particularly for system with large ductility \((\mu=4.0)\).

c. For SSI system on soft soil \((a_0=1.0)\) the inelastic yield strength spectrum is larger than the fixed-base system. For case of stiff soil \((a_0=1.0)\) the soil-structure curves leads to larger values for inelastic strength demands than the case of soft soil \((a_0=2.0)\).

d. For SSI system the inelastic yield strength spectrum is larger for system with smaller ductility than for system with larger ductility.

e. Generally, from reading the inelastic strength demand of the fixed-base system for any structural period one may lead to underestimate of the value for the soil-structure system. It is seen that for slender size buildings the soil-structure curves lead to larger values for strength demands.

f. Figure 9 shows the comparison of inelastic strength demands with \(\mu=2.0\) and 4.0 for case of \(a_0=1.0\), \(h_r=3.0\) and for case of \(a_0=2.0\), \(h_r=3.0\), respectively. It is found that for system with smaller ductility ratio will lead to larger values of strength demands.

Comparison on inelastic displacement spectrum for system with and without considering SSI is also shown in Figure 10. For stiffer structure \((a_0=1.0)\) and lower aspect ratio \((h_r=1.0)\) the inelastic displacement is significant as compare to other cases in longer period. Generally speaking, the displacement demand is more significant for system with considering SSI effect.

Study of Equivalent Linear System on SSI System

To gain a simplified physical model of the problem, an equivalent SDOF system is also introduced to represent the soil-structure interaction system. The properties of this equivalent linear system are selected such that the same responses result as for the SSI system model. Neglecting the effects of the foundation mass and the mass moment of inertia, an approximation for the stiffness and damping ratio of the equivalent system are provided by the following equations (Wolf, 1994), respectively:

\[
\frac{1}{K_{eq}} = \frac{1}{k_{eff}} + \frac{1}{k_h} + \frac{h^2}{k_\phi}
\]

\[
\zeta_{eq} = \left(\frac{\omega_{eq}}{\omega_{eff}}\right)^3 \zeta_{eff} + \left(\frac{\omega_{eq}}{\omega_h}\right)^3 \zeta_h + \left(\frac{\omega_{eq}}{\omega_\phi}\right)^3 \zeta_\phi
\]

where
\[
\omega_{\text{eff}} = \sqrt{\frac{k_{\text{eff}}}{m}}, \quad \omega_{\phi} = \sqrt{\frac{k_{\phi}}{m}}, \quad \zeta_{\phi} = \frac{c_{\phi} \omega_{\phi}}{2k_{\phi}}
\]

Without loss of generality, in this study, the following specific values are assigned to these parameters:

\[
a_0 = 1.0 \quad \nu = 0.33 \quad M_r = 1.0
\]

Material damping ratio of the structure is set to 5\% of the critical damping at the frequency of the soil-structure system. The results are related to a building with the aspect ratio of \( h/r = 1.0 \), and 3.0 located on soil half-spaces. **Figure 11** shows the estimated yield strength spectrum and the inelastic displacement spectrum using the proposed equivalent linear system model (Eqs. 21 & 22). The inelastic time history response analysis of the SSI system is also calculated. As shown in this figure the agreement on the estimated yield strength spectrum and the inelastic displacement spectrum between the two models is in consistence. It is concluded that the equivalent linear SDOF system using the approximation for the stiffness and damping ratio of the SSI system can estimate the strength and the displacement demands of the system. **Figure 12** shows the effective damping ratio \( \zeta_{\text{eff}} \) with respect to system ductility ratio using BMR1 inelastic hysteretic model for different structural period. It is concluded that, for case of SSI with \( a_0 = 1.0 \) and \( h/r = 3.0 \), the estimated effective damping increase with the increasing structural ductility and for stiffer structure (with smaller T-value) the estimated effective damping ratio is larger than softer structure (with larger T-value). This is due to the contribution of SSI effect. The ratio of yield strength \( F_y \) between SSI model and fixed-base model is also investigated, as shown in **Figure 13**. It is observed that the ratio is greater than one for cases of (a) \( a_0 = 2 \) and \( h/r = 3 \), (b) \( a_0 = 1 \) and \( h/r = 3 \), and the effect of structural system ductility is not so sensitive to this ratio. If the SSI effect is not included in the consideration on the estimation of yield strength, under-estimation on yield strength will be obvious.

**Conclusions**

The purpose of this study is to predict the seismic demands of building structures during earthquake excitation. The effects of soil-structure interaction (SSI) on the seismic demand of the buildings are the main concern in this study. A simplified model is used to conduct the parametric study on the effect of SSI which taking inelastic hysteretic behavior (Bouc-Wen hysteretic model) in the super-structure into account only. The effect of foundation flexibility \( (a_0) \) and building aspect ratio \( (h/r) \) on structural strength demands has been investigated. An equivalent linear system for super-structure to predict the nonlinear response of the system is also introduced. It was shown that the main features of the SSI effect could be captured by a
simplified model. Through this study on SSI effect on seismic demand the results indicate that, with the consideration of SSI effect the seismic demands are higher in slender structure and in stiff soil. The yield strength is higher for system with the consideration of SSI than the system with fixed-based structure.

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References


Hysteretic Element + Slip-lock Element = Composite Element

**Figure 1:** Hysteretic Element in Series with Slip-Lock Element

**Figure 2:** Comparison of force-deformation characteristics of flexure-type inelastic model between experiment (left) and simulation (right).

**Figure 3:** Schematic model of inelastic SDOF system: (a) with fixed base, (b) with soil-structure interaction.
Figure 4(a): Comparison between the inelastic time history response analysis and the equivalent linear time history analysis by plotting the yield force spectrum with respect to constant ductility ($\mu = 2.0$ and 4.0 respectively) using BMR1 inelastic model (without SSI effect).

Figure 4(b): Comparison between the inelastic time history response analysis and the equivalent linear system time history analysis by plotting the inelastic deformation spectrum with constant ductility ($\mu = 2.0$ and 4.0 respectively) using BMR1 inelastic model (without SSI effect).
Figure 5: (a) Estimated effective damping ratio with respect to effective structural period $T_{eff}$ for different ductility ratio using BMR1 inelastic hysteretic model, (b) Plot of effective damping ratio $\xi_{eff}$ with respect to system ductility ratio using BMR1 inelastic hysteretic model for different structural period,

Figure 6: Comparison on the estimated effective damping ratio $\xi_{eff}$ with respect to system ductility ratio by using time history response with BMR1, and with Chopra model, and Kowalsky model.
Figure 7: Comparison on elastic strength spectrum for system with and without considering SSI; (a) for case of $a_0=1.0$ and 2.0, with $h_r=1.0$, (b) for case of $a_0=1.0$ and 2.0, with $h_r=3.0$, (c) for case of $a_0=1.0$ with $h_r=1.0$ and 3.0, (d) for case of $a_0=2.0$ with $h_r=1.0$ and 3.0.
Figure 8: Comparison on yield strength spectrum for system with and without considering SSI; (a) for case of \( a_0 = 1.0 \) and 2.0, with \( h_r = 1.0 \), (b) for case of \( a_0 = 1.0 \) and 2.0, with \( h_r = 3.0 \), (c) for case of \( a_0 = 1.0 \) with \( h_r = 1.0 \) and 3.0, (d) for case of \( a_0 = 2.0 \) with \( h_r = 1.0 \) and 3.0.
Figure 9: Comparison on yield strength spectrum for $\mu=2.0$ and 4.0 and for system with and without considering SSI; (a) for case of $\alpha=1.0$, $h_r=3.0$, (b) For case of $\alpha=2.0$, $h_r=3.0$
Figure 10: Comparison on inelastic displacement spectrum for system with and without considering SSI; (a) for case of $a_0=1.0$ and 2.0, with $h_r=1.0$, (b) for case of $h_r=1.0$ and 3.0, with $a_0=2.0$, (c) for case of $a_0=1.0$ and 2.0, with $h_r=3.0$
Figure 11: Plot the comparison between (a) yield strength spectrum and, (b) inelastic deformation spectrum, with constant ductility (μ = 2.0 and 4.0 respectively) using equivalent linear system method and inelastic time history response analysis.

Figure 12: Plot of effective damping ratio $\xi_{\text{eff}}$ with respect to system ductility ratio using BMR1 inelastic hysteretic model and SSI model ($a_0=1.0$ and $H_f=3.0$) with different structural period.
Figure 13: Ratio of yield strength spectrum between SSI system and fixed based system with $\mu=2.0, 4.0$ and $6.0$; (a) for $a_0 = 2$ and $h_r = 3$, (b) for $a_0 = 1$ and $h_r = 3$. 
Part 3

Consideration of Collapse and Residual Deformation in Reliability-Based Performance Evaluation of Buildings

SUMMARY

The consideration of collapse and residual deformation in structural response analysis has been shown favorable in this study in view that performance-based design methodology could be successfully implemented in the near future to account for distinct nonlinear dynamic behavior of traditional structural systems and modern advanced structural systems with self-centering devices. In the framework of dual-level design, ordinary building stocks generally reach a structural state of near collapse at the 2% in 50 years hazard level. On the other hand, residual deformation, when combined with maximum deformation, has been shown effective in evaluation of structural performance under seismic excitation especially for advanced structural systems. To incorporate collapse and residual deformation in structural performance assessment, collapse experiments are recommended to establish such hysteretic models with post-peak material behavior.

introduction

Performance-based seismic design philosophy has been recently incorporated into new generation code documents. To successfully implement dual-level design methodology, hysteretic models that take into account post-peak material behavior is favorable since ordinary building stocks generally reach a structural state of near collapse at the 2% in 50 years hazard level. On the other hand, with increasing use of modern advanced structural systems with self-centering devices, residual deformation, when combined with maximum deformation, has become very effective in assessment of structural performance under seismic excitation. Moreover, residual deformation is also a major concern to building owners and design engineers since it well represents the final status of building structures after earthquakes. As such, a conceptual Performance Matrix has been recently proposed in Pampanin et al. [1], using both maximum and residual deformations as performance evaluation indices. The reasons to evolve a currently popular and relatively simple evaluation method into a more knowledge-demanding performance evaluation method is based on an understanding of the following aspects:

1. The consideration of near-fault motions with severe directivity velocity pulses and/or static displacement fling could lead to permanent deformation of the structural system when highly
nonlinear structural behavior occurs. Moreover, a 2% in 50 years hazard level usually make an ordinary structure almost reach its collapse state.

2. Buildings designed to older code documents are susceptible to severe damage or may even collapse during a severe seismic event. This is especially true for the observed low-cycle collapse of reinforced concrete frame buildings with light transverse column reinforcement during the 1999 Chi-Chi earthquake.

3. Modern advanced structural systems, in contrast to traditional systems, may be capable of re-centering itself back to the original position after earthquakes.

The former two aspects are the main concerns of this study, while one may refer to Pampanin et al. regarding the 3rd aspect on modern advanced structural systems. Modern advanced structural systems with re-centering device have inspired the use of residual deformation as an additional performance evaluation index in the near future. As we know, the current performance evaluation method is based on one or multiple structural response indices including maximum drift ratio (or, ductility) and cumulative inelastic energy dissipation, which are known to be able to fully characterize performance levels for systems where the main concern is to avoid collapse, but are unable to characterize the performance level of some modern structural systems (e.g., self centering frame buildings) where the structural integrity is not at risk during seismic attack. Besides, in the three aforementioned cases maximum response itself may not be able to fully represent the final status of the structure. As such, an independent scale of residual-deformation based performance measure can be effectively combined with the existing performance measure based on maximum response (or, cumulative damage) to form a more general performance domain. For different seismic intensity levels it would result in a full 3-dimensional performance domain, which is schematically described in Figure 1 (Pampanin et al.).

![Figure 1. Framework for Residual-Maximum Performance Based Approach: Performance Matrix (source: Pampanin et al. 2003).](image-url)
MATHEMATICAL FORMULATION OF HYSTERETIC MODEL WITH CONSIDERATION OF POST-PEAK BEHAVIOR

The governing equation of motion for an SDOF oscillator demonstrating hysteretic behavior can be expressed as:

\[ m\ddot{x} + c \dot{x} + kx + k \sum_{i=1}^{n} \alpha_i u_i = -m\ddot{x}_g \]  

(1)

where \( m \) is mass, \( c \) is viscous damping coefficient, \( k \) is initial stiffness of the system, \( \alpha_i \) is post-to-preyield stiffness ratio (or, strain hardening ratio), \( u_i \) is auxiliary state variables, \( x_g \) is ground displacement, and \( x \) is relative displacement of the SDOF oscillator with respect to the ground. Dots indicate time derivative. The hysteretic models found in the literature to describe nonlinear behavior of deteriorating structural systems under cyclic loading can be categorized into three groups: (1) smooth hysteretic model, e.g., Wen [2], Baber and Noori [3], Wang and Wen [4], Sivaselvan and Reinhorn [5], etc.; (2) rule-based polygonal hysteretic model, e.g., Park et al. [6], Shi [7], Elwood [8], Ibarra et al. [9], etc.; (3) theory-based piecewise linear hysteretic model, e.g., Mostaghel [10], etc. In this study, a new theory-based piecewise linear model is proposed, and it can be used to define constitutive relationship between stress and strain, force and displacement, moment and curvature, or moment and rotation, depending on the applications as long as the quantity and quality of experimental results are sufficient for the determination of the values of key model parameters.

The SDOF hysteretic system in Figure contains one linear spring, the deformation of which is represented by \( x \), and a slider-spring element with a frictional surface of variable Coulomb damping coefficient such that the slider-spring will start to slip at a certain force level (e.g., \( k \cdot u_y \) at first yield). However, the slip may accelerate when the friction coefficient reduces to a lower level because of a decrease in the interlocking force between internal particles of the system, which, in the case of RC columns, physically means major shear cracks have fully developed. Such post-peak behavior is commonly observed in low-confinement RC columns, pre-Northridge steel connections, and wood shear wall system. To describe such a phenomenon, unknown \( u \) representing the deformation of the spring connected to the slider can be expressed in the following mathematical form:

\[
\begin{align*}
\mu \delta = & x \Omega \left\{ \phi \cdot M \left( u - \lambda_p \cdot \phi \cdot \delta \right) \cdot M (x) + \phi \cdot M \left( u - \phi \cdot \delta \right) \cdot N (x) \\
& + \left( -\phi - \phi \right) \cdot N (x - \delta) \cdot M (u) \cdot N (x - \delta) \\
& + \left( -\phi - \phi \right) \cdot N (x - \delta) \cdot N (u) \cdot N (x - \delta) \\
& + \left( -\phi - \phi \right) \cdot M (x) \cdot N (u) \cdot N (x - \delta) \\
& + \left( -\phi - \phi \right) \cdot M (x) \cdot N (u) \cdot N (x - \delta) \\
& + \left( -\phi - \phi \right) \cdot M (x) \cdot N (u) \cdot N (x - \delta)
\right\}
\end{align*}
\] 

(2)
Figure 2. Schematic representation of SDOF hysteretic system with consideration of post-peak behavior.

in which, \( N(x) = H(x) ; M(x) = 1 - H(x) ; \overline{N}(x) = 1 - H(-x) ; \overline{M}(x) = H(-x) \), and \( H(x) \) is the Heaviside’s unit step function. The constant \( \lambda_p \), which takes a value between 0 and 1, denotes the resistance ratio and represents pinching of a hysteretic loop due to unequal strengths. There is no pinching in a structural component if \( \lambda_p \) is assumed to be 1. \( \delta_y \) is the yield displacement of the system; \( \delta_u \) is the displacement at which the system reaches its ultimate strength and the friction coefficient of the slider will start to decrease; \( \delta_r \) is the displacement at which the system reaches its residual strength; \( \delta_{FS} \) defines the failure surface in Figure 3 and represents the corresponding deformation of the linear spring when the slider-spring reaches a certain deformation at a time step. The positive and negative signs in the superscript indicate asymmetric material property may exist under compression and tension. The failure surface may migrate to a lower level of material strength according to a prescribed flow rule (e.g., Figure 4, etc.) when dissipated hysteretic energy accumulates with time. In addition,

Stiffness-degradation function
\[
\phi_k = \frac{1}{1 + \lambda_k h(t)} \tag{3}
\]

Load-deterioration function
\[
\phi_l = \frac{1}{1 + \lambda_l h(t)} \tag{4}
\]

Post-peak stiffness function
\[
\phi_c = \frac{(1 - \rho) \delta_s}{\delta_r - \delta_u} \tag{5}
\]

And, \( h(t) \) denotes the hysteretic energy absorbed by the system. A multilinear system with more than one slider-spring element may also be used to obtain a smoother hysteretic loop. For brevity, one slider-spring model is used in the study since such an approximation is commonly used in practice, and is usually accurate enough in most cases, and can also lessen computational efforts. Unlike Bouc-Wen hysteretic model, the physical meanings of the
proposed model parameters are self-evident, so the parameter values can be determined with no need to go through complicated nonlinear regression procedures.

Figure 3. Schematic representation of failure surface.

Figure demonstrates several numerical hysteretic models having negative post-peak stiffness in the upper row and relevant experimental results showing the same characteristics are given in the lower row: (1) Bilinear hysteresis with capping (upper left), and a welded haunch steel moment connection taken from Uang et al. [11] (lower left), (2) Pinching with collapse (upper middle), and a concrete shear wall taken from Oh et al. [12] (lower middle), (3) Pinching with collapse (upper right), and a low-ductility RC column taken from Elwood [8] (lower right).

Figure 4. Evolution of failure surface and its controlling parameters.
PERFORMANCE EVALUATION METHOD CONSIDERING PEAK AND RESIDUAL DEFORMATIONS

A preliminary study has been conducted, and numerical results show that collapse consideration is essential in order to implement the newly proposed Performance Matrix in the next generation design codes. Experience shows that peak and residual deformations can be predicted with accuracy only if post-peak negative stiffness and collapse are taken into account in structural response analysis. Collapse analysis on a 12-story RC building is presented as an example in the following.

Description of 12-story RC Model Building

A 12-story RC building with a natural period of 1.61 sec was designed in Liao and Wang [13]. The building has a 25cm yield displacement and a 10% pre-to-postyield stiffness ratio according to static pushover analysis results. 5% viscous damping of critical is assumed. To perform collapse analysis on the building, an equivalent SDOF system using base shear formulation suggested by Collins et al. [14] is used. Reliable estimate of global (roof) drift ratio of the building is expected using the suggested procedure. For non-deteriorating bilinear system, $\phi_k = 1$, $\phi = 1$, $\lambda_k = 0$, $\lambda_l = 0$ and $\lambda_p = 1$. $\lambda_p = 0.3$ is assumed for pinching system; $\lambda_k = 1$ and $\lambda_l = 0.1$ for deteriorating system. $\rho = 0.25$ is assumed for residual
strength. When collapse is a concern, \( \delta_r = 4 \delta_y \), \( \delta = 6 \delta_y \), and \( \phi = 0.1 \) is assumed for the system.

**Suites of Uniform Hazard Ground Motions**

Design spectra corresponding to 2 hazard levels of Design and Maximum Considered Earthquakes from the Taiwanese seismic design code are constructed in Figure 6 to represent the base shear coefficients of the Xinyi district of Taipei basin. According to probabilistic seismic hazard analysis results (Jean [15]), an intermediate hazard level of 5% exceedance probability in 50 years is added to our response analyses. To get 10 uniform hazard earthquake motions at each hazard level, TSMIP array data in Taipei basin within a time frame of 1994 – 2002 are selected to match the design spectra in a wide range of periods. Important characteristics of these selected motions are summarized in Tables 1 through 3. The scaling factors are preferentially no larger than 12.61, which imply that no small magnitude earthquakes are taken to represent much higher hazard levels of large magnitude earthquakes. Due to the limitation of data obtained from the field, it is observed that most of the selected motions are from the 1999 Chi-Chi earthquake and the March 31, 2002 Hualien earthquake. Median, 16- and 84-percentile spectra of the 10 selected motions are plotted against the target design spectra for comparison (Figure 6).

Table 1. Important characteristics of 10 earthquake motions in the 10% in 50 years hazard level.

<table>
<thead>
<tr>
<th>Date (GMT)</th>
<th>( M_L )</th>
<th>Focal Depth (km)</th>
<th>Station ID</th>
<th>Duration (sec)</th>
<th>PGA (g)</th>
<th>Scaling Factor</th>
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Table 2. Important characteristics of 10 earthquake motions in the 5% in 50 years hazard level.

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Table 3. Important characteristics of 10 earthquake motions in the 2% in 50 years hazard level.

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Nonlinear Time History Analysis and Engineering Implications

Nonlinear time history analysis is performed on the equivalent SDOF system to yield estimates of maximum and residual roof drift ratios of the 12-story RC building using the 5th order Cash-Karp Runge-Kutta method to implement adaptive time stepping algorithm. Numerical results are given in the format of Performance Matrix. Although not significant, the bilinear system generally has a lower level of maximum drift ratios, but its residual drift ratio is slightly higher. This is because in this study an identical level of strength deterioration and stiffness degradation is assumed for bilinear and pinching systems. Comparisons are also made in system responses with and without consideration of collapse mechanism. It is observed that response estimates at the 10% in 50 years level coincide for both cases since collapse, mostly, does not occur. However, as long as 2% in 50 years hazard level is of concern, the response estimate will strongly depend on whether collapse is taken into consideration. This observation surely has very important implications in seismic design, and performance evaluation of structural systems under seismic excitation. If the use of Performance Matrix is a necessity in the framework for performance-based earthquake engineering, then the consideration of collapse in dynamic analysis will help map a structure’s performance into its corresponding Performance Level cells with confidence, as shown in Figure 1. In passing, it is noted that the traditional analysis approach usually provides only information on whether the building collapses according to engineering judgment and experience, but a confident estimate of maximum and residual drift is not possible.
Figure 6. Performance Matrix of the 12-story RC model building with bilinear (left) and pinching (right) hysteretic behavior.

**Shake Table Test on RC Frame with Low-ductility Columns**

A series of shake table tests are desirable in order to validate the proposed piecewise linear hysteretic model, and experimental results could be very supportive in determination of key parameter values. Although not many, a few collapse experiments had been conducted to this date. Among those are gravity load collapse of ½-scale RC frame by Elwood [8] and small-scale steel frame test by Vian et al. [16]. To investigate how the 4-story commercial-resident complexes sustained severe damage and some of them even collapsed in the 1999 Chi-Chi earthquake, a ½-scale RC frame composed of 2 low-ductility columns inter-connected by a strong beam (Figure 7) was tested on the shake table earlier in February 2004. The frame specimen originates from the collapse experiments by Elwood with a few modifications to account for:
1. Realistic vertical deformation on columns of the frame specimen with no alternative path for load redistribution.

2. Typical 4-story commercial-resident buildings since the column design is taken from a real 4-story building in central Taiwan.

3. Soft 1st story design that is commonly observed for the 4-story commercial-resident complexes in Taiwan, and during the 1999 Chi-Chi earthquake a large number of such buildings sustained damage only at the 1st story.

The NS component of TCU076 accelerogram recorded during the 1999 Chi-Chi earthquake was used to excite the frame specimen of a 0.35sec natural period, representing a 0.49sec commercial-resident complex at full-scale. TCU076, stationed at Nantou Elementary School, is less than 250m from the 4-story target building. The specimen was subjected to a sequence of TCU076 records, scaled from 25gal to 700 gal to obtain hysteretic behavior of the specimen. The achieved table motion of TCU076 and its response spectra are given in Figure 8. Two hysteretic loops are shown in Figure 9, corresponding to intensity levels of 50gal and 700gal, respectively. Only bending cracks were observed during the test, which agrees with the reconnaissance report of the target building after the 1999 Chi-Chi earthquake. Such design, representing an upper bound of the building performance in central Taiwan, did help prevent the building from collapse during a severe earthquake event. In order to collect more experimental data for validation of the proposed hysteretic model, two more shake table tests are under preparation and they are expected to yield helpful data such that considerable improvements can be achieved on numerical dynamic analysis of structural collapse in the near future.

Figure 7. Front view of the experimental setup of the low-ductility RC frame.
CONCLUSIONS

The preliminary finding of this ongoing study is the introduction of performance-based earthquake engineering into the seismic design documents has indicates the necessity of considering post-peak behavior of structural systems into nonlinear dynamic analysis especially at the hazard level of very rare events such as 2% exceedance probability in 50 years. To do so, a new mathematical form is presented in this study to describe hysteretic loops with post-peak behavior. Unlike other rule-based models, the proposed mathematical form can be readily embedded into existing in-house and commercial software with no
excessive coding efforts. It is also shown that collapse consideration is necessary for ordinary building stocks if Performance Matrix is to be incorporated into the next generation design codes.

ACKNOWLEDGEMENTS

The financial support for this ongoing research from the National Science Council of Taiwan under grant number NSC92-2811-E-002-023 is gratefully acknowledged. The authors would like to thank the Central Weather Bureau of Taiwan for providing TSMIP ground motion data and Chin-Hsun Yeh of NCREE for preliminary processing of those data. Experimental facilities and technical support from NCREE are much appreciated. Special thanks are extended to Pei-Yang Lin and Lu-Sheng Lee for their assistance in conducting the shake table test. All opinions expressed in this paper are solely those of the authors, and, therefore, do not necessarily represent the views of the sponsor.

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