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NON-LINEAR ANALYSIS OF BRIDGE STRUCTURES

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Abstract: For the health tracking of civil infrastructures, it is essential to determine the non-linear behaviour connected to structural damage. For the precise assessment of these types of non-linear behaviours, it is essential to evaluation of how these structures will function when exposed to specific earthquake movement. To determine the behaviour, non-linear static or non-linear time history analysis approach can be utilized, but the locally destroyed impact has to be also regarded. With the prominent impact of basic mode of non-linear static approach, non-linear time history evaluation approach is broadly utilized for the evaluation of complex non-linear behaviour with many degrees of freedom and with local damages. In this study, the non-linear time history evaluation method with some restricted higher modes accounting the impact of local damages is suggested. Specifically, some RC piers are presumed to be surpassed the yield capability throughout earthquakes and trigger large inelastic deformations and damage. To identify the seismic response extremely impacted by the hysteretic behaviour of destroyed RC piers, the modified Takeda model is presented. As a confirmation of effectiveness of suggested approach, the non-linear responses of damaged bridge structure are investigated among suggested approaches and above described traditional non-linear analysis approach.

Keywords: Nonlinear Dynamics; hysterical model; modified Takeda model; modal order; substructure

KÖPRÜ YAPILARININ DOĞRUSAL OLMAYAN ANALİZİ

Özet: Altyapı tesislerinin doğru ve sağlık takibi için, yapısal hasarın doğrusal olmayan davranışının belirlenmesi esastır. Bu tür doğrusal olmayan davranışların kesin olarak değerlendirilmesi için, bu yapıların belirli deprem hareketlerine maruz kaldıklarında nasıl işlev görecekleri incelenmelidir. Davranışı belirlemek için, doğrusal olmayan statik veya doğrusal olmayan zaman artımı (time history) yöntemi kullanılabilir, ancak yerel etkiler de göz önüne alınmalıdır. Doğrusal olmayan statik yaklaşımla doğrusal olmayan zaman artımı yöntemi yaklaşımı, birçok serbestlik derecesi ve yerel hasarlar içeren karmaşık ve doğrusal olmayan davranışın değerlendirilmesi için yaygın olarak kullanılmaktadır. Bu makalede, yerel hasarların etkisini dikkate alan sınırlı yüksek modları olan doğrusal olmayan zaman artımı yöntemi önerilmiştir. Özellikle bazı betonarme ayakların, depremler sırasında esneme kapasitelerini aştığı ve bunun da büyük inelastik deformasyonları ve hasarları tetiklediği varsayılmaktadır. Hasar görmüş betonarme ayakların histerik davranışından aşırı derecede tetiklenen sismik etkiyi tanımlamak için değiştirilmiş Takeda modeli sunulmuştur. Önerilen yaklaşımın etkililiğinin doğrulanması için, hasar görmüş köprü yapısının doğrusal olmayan tepkileri, önerilen yaklaşımlar ve yukarıda açıklanan geleneksel doğrusal olmayan analiz yaklaşımı ile incelenmiştir.

Anahtar Kelimeler: Doğrusal olmayan dinamik; histerik model; modifiye Takeda modeli; modal sıralama; alt yapı

INTRODUCTION

The forces induced on a bridge structure with reinforced concrete (RC) piers during major earthquakes may exceed the yield capacity of some piers and cause large inelastic deformations and damages in the piers as depicted in Figure 1.

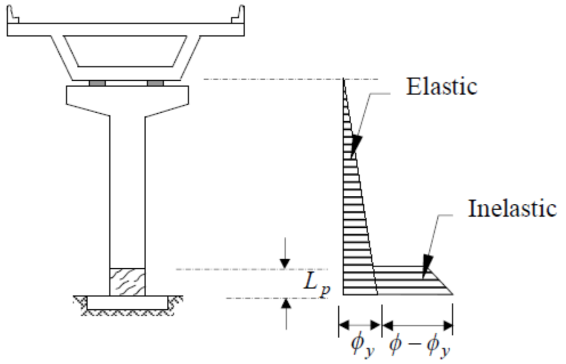

Figure 1. Inelastic behavior of a RC bridge pier

Since the seismic response of bridge piers is highly affected by the hysteretic behavior when they have damaged, reliable model for such behavior is needed. The system of strength deterioration of RC elements is generally construed by concrete spalling and interface bond slip among the concrete and embedded reinforcements. In the celebrated damage model for RC members suggested by Park and Ang (1985), cumulative damage is based on a linear combination of the maximum displacement and the cumulative hysteretic energy dissipation. Nevertheless, only one of the two variables is generally integrated in existing strength deterioration models in the literature, say several models are structured only on the maximum displacement, such as models in Lai et al (1984), Roufaiel and Meyer (1987), Chung et al (1989) and Youssef and Ghobarah (2001), though others on the cumulative energy dissipation, such as in Kunnath et al (1997), Mork (1991), Rahnama and Krawinkler (1993) and Sucuoglu and Erberik (2004).

The bilinear peak driven hysteretic model as demonstrated in Figure 2 and 3 presents a typical base for all the existing strength deterioration models. The strength deterioration can be indicated either by softening the skeleton curves (Figure 2) or shifting the reloading oriented points (Figure 3). In Figure 2 and 3, F_{yi} relates to the yield strength at the ith loading cycle; Δu_o and ΔF_o refer to the change of displacement and force of the oriented point, correspondingly. Triangles and circles indicate the maximum displacement point and the reloading oriented point in a loading cycle, correspondingly.

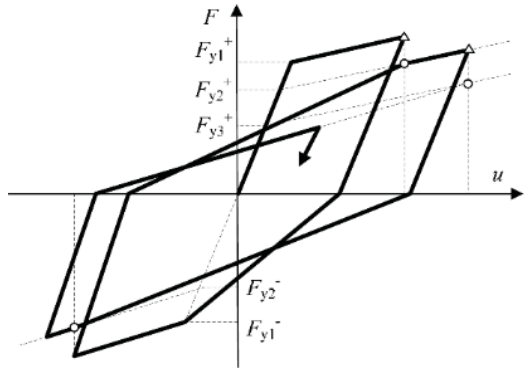

Figure 2. Strength deterioration in peak oriented hysteretic model: Soften the skeleton curve

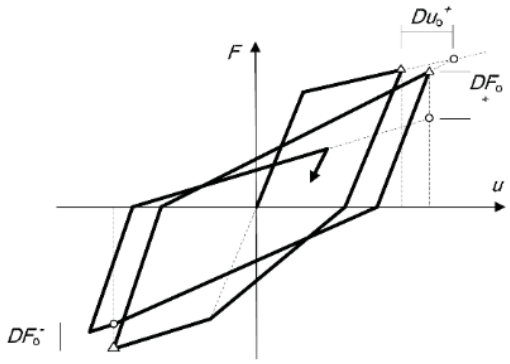

Figure 3. Strength deterioration in peak oriented hysteretic model: Move the reloading oriented point

Amongst different models, stress-strain constitutive models are the most popular as they provide more realistic representation of concrete behaviour such as stress-strain relationship, and non-linear behaviour in cracking and crushing; and they have been used in modelling of the structure based on the computationally powerful method, the Finite Element Method. In constitutive modelling of concrete materials, it is known that either plasticity-fracture or plasticity-fracture-damage models are required in order to simulate concrete behaviour well (Jefferson 1999, 2002a, 2003a). In literature, however, no one constitutive model is yet able to properly describe all aspects of non-linear concrete behaviour because of the complexity of multiaxial behaviour of concrete. In addition, not many constitutive models have been successfully implemented into engineering practice to deal with both complex RC structures and earthquake loadings. Therefore, another important objective of the research is to employ two of the most recently developed constitutive models, one based on the plasticity-fracture approach, namely Multi-crack model (Jefferson 1999) and the other based on plasticity-fracture-damage approach, namely Craft model (Jefferson 2003a, 2003b), for modelling concrete and RC structures under different types of loading. In spite of the numerical efficiency of these methods, however, enough many modes have to be included or the influence of truncated modes have to be corrected to achieve an approximated result with reasonable accuracy particularly on local behavior (Dikens et al. 1997).

In concrete material, strain-softening problem is a common phenomenon (Hillerborg et al. 1976, Bazant and Oh 1983). This is also considered in the constitutive models used in this study, based on continuum mechanics. Strain-softening can induce localised instabilities in the numerical procedure and consequently, non-unique solutions or mesh-dependency problems for numerical analysis (Crisfield 1982, Zienkiewicz and Taylor 1991, Crisfield 1996), and thus use of classical continuum mechanics in this case has been proved to be inadequate (Comi 2001, Jirasek and Bazant 2002). In an attempt to avoid mesh dependency problem, the fracture energy provisions of crack is used (Hillerborg et al. 1976). In the smeared cracking approach, cracking is assumed to be spread over a 'numerical' fracture process zone which is numerically or mathematically equated the characteristic length of an element. As this characteristic length is related to the adopted finite element size, the spurious mesh dependency can be eliminated (Bazant and Oh 1983, Oliver 1989). Due to these softening-related problems, the identification of model parameters and non-linear procedures play a crucial part in the validation and application of the models.

Seismic design of RC bridge piers is increasingly performed using dynamic analysis in the time domain, where the responses of the structure to appropriately selected time-histories is strongly dependent on the characteristics of the earthquake ground motions. Besides, the dynamic effects that arise from the random ground motions should be taken into account for the characterisation and the modelling of the non-linear and damage behaviour of RC bridge piers through its material models. However, seismic applications of finite element material models have not been widely used for such investigations due to technical challenges in implementing them into non-linear dynamic analysis. As a result, very little work has been done into the non-linear dynamic response and damage behaviour as well as their quantitative measures for RC bridge piers under earthquake time-histories (Kwan and Billington 2003, Hindi and Sexsmith 2004). Therefore, the non-linear dynamic response and damage are also pursued in this study, with the use of nonlinear material models for the analyses of RC bridge piers under artificially generated timehistories.

In this study substructuring method with modal correction vectors and modal sorting method are proposed to analyze reinforced concrete structures having locally damaged properties. The hysteretic behaviors of the damaged concrete structures are reproduced by multi linear hysteretic model with

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limited nonlinear parameters including the characteristics of stiffness degradation, pinching effect by shear and axial force and strength deterioration (Lee and Yun 2008). Figure 4 shows different hysteresis models.

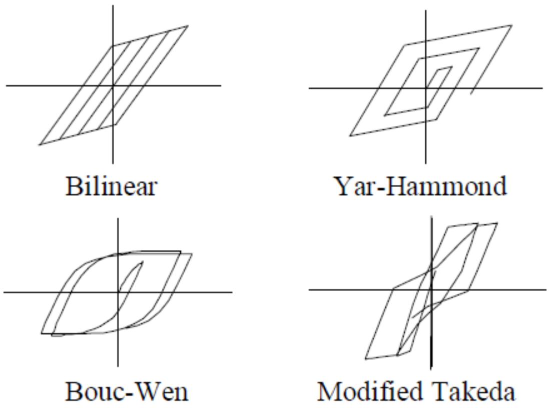

Figure 4. Hysteresis models

SEISMIC RESPONSE ANALYSIS OF RC BRIDGE PIER

It is obvious that under different earthquake time history records, the structure experiences different response and damage. In order to analyse and compare the response and damage behaviour of the structure, a method for quantifying the damage has to be devised and used. One class of methods to quantify damage is the use of a "damage index" to create a single measure that adequately represents the complex seismic behaviour. Damage indices aim to provide a mean of quantifying numerically the damage in reinforced concrete structures sustained under cyclic and earthquake loading (Hindi and Sexsmith 2001, 2004). In earthquake engineering literature, there have been various damage measures proposed and considered in the experimental and theoretical studies to explain damages observed in the structures under artificial ground motions or in actual structures subjected to real earthquake motions such as Park and Ang (1985), Chung et al. (1989), Chai et al. (1995), Fajfar and Gaspersic (1996), Ghobarah et al. (1999), and Hindi and Sexsmith (2001).

Many studies have been performed in the analysis or characterisation of seismically-induced damage to reinforced concrete members and, in particular, RC bridge piers (Banon et al. 1981, Park and Ang 1985, Roufaiel and Meyer 1987, Stephens and Yao 1987, Jeong and Iwan 1988, Chung et al. 1989, Williams and Sexsmith 1995, William et al. 1997, Ghobarah et al. 1999, Hindi and Sexsmith 2001, and Erberik and Sucuoglu 2004, Kim et al. 2005). However, the majority of these studies are based on data from static cyclic tests in both numerical and experimental areas.

NONLINEAR HYSTERIC BEHAVIOR

Roufaiel and Meyer (1987) proposed an extension of the spread plasticity model developed earlier by Meyer et al. (1996). The new model includes the effect of shear and axial forces on the flexural hysteretic behavior based on a set of empirical rules.

The hysteretic moment-curvature relation is described by Takeda's model. The variation of axial loads due to overturning moments is not accounted for. The analytical results are compared with available experimental data and show very good agreement. A set of new damage parameters is proposed which correlate well with the residual strength and stiffness of specimens tested in the laboratory. In the modified Takeda model, four different kinds of braches may exist in the hysteresis of the moment-curvature (M -ϕ) relationship as in Figure 5. Basically, The Takeda model (Takeda et al. 1970) includes the phenomenon of stiffness degradation in reinforced concrete members subject to cyclic loading. Roufaiel and Meyer (1987) introduced a model that accounts for the pinching effects due to shear force and strength degradation after yielding.

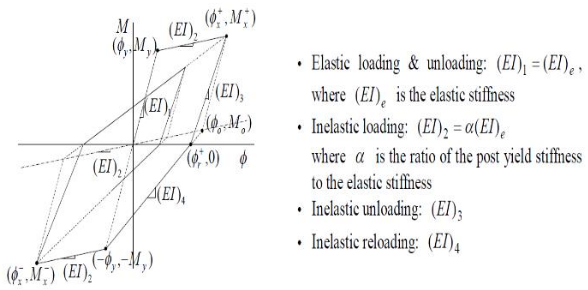

Figure 5. Hysteretic moment - curvature behavior of the modified Takeda model

Park and Ang (1985) formulated and suggested a hysteretic model utilizing damage index to define the gradual strength degradation and softening perform in reinforced concrete structural members caused by repetitive increase of the maximum deformation maintained by the member. Nevertheless, few of them contain all the factors of the deterioration of strength, stiffness, and ductility features in a comprehensive hysteretic design to get the inelastic behavior of reinforced concrete members under big reversal cycles loading. The hysteretic designs for modeling the phenomena of stiffness degradation, pinching effect, and strength deterioration and softening are in short, described here. When a reinforced concrete structural member is subjected to repeated cyclic loading above its elastic limit, the evolvement of concrete cracking and plastic behavior of the reinforcing steel with linked anchorage slip would head to the deterioration of the stiffness of the reinforced concrete member at each cyclic loop. A Q-HYST degrading stiffness hysteretic model proposed by Saiidi and Sozen (1979), improved from the Takeda model (Takeda et al. 1970), can efficiently account the unloading stiffness. Laboratory assessments performed on reinforced concrete specimens by Popov et al. (1972) and Ma et al. (1976) have identified that there is a strong correlation among the degree of pinching and the magnitude of shear at the section, and that pinching effect minimizes the load resistance of the member during reloading.

In this study, the concrete bridge pier is assumed to be locally damaged at the bottom of the pier due to severe earthquake ground motion and the dominant nonlinear hysteretic behaviors can be effectively represented by four parameters like as yield moment(M_y), stiffness degradation(α), pinching (β) and stiffness deterioration(γ). Figure 6 shows typical hysteretic behavior of a RC members subjected to cyclic loadings for several cases of these parameters.

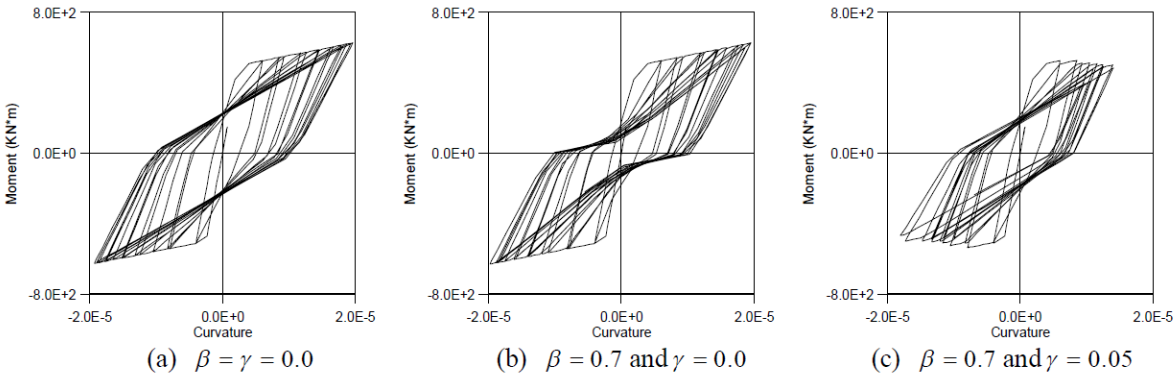

Figure 6. Moment vs. curvature in a RC bridge pier (M_y = 500KN.m y M = × , and $\alpha = 0.03$)

NONLINEAR DYNAMIC ANALYSIS

In this approach, the seismic response of the framework is examined utilizing step-by-step time history analysis. The major methodology of this process is nearly identical to the static method of analysis. Nevertheless, this technique varies in the principle that the design displacements are not set up utilizing the targeted displacement; yet, are estimated through dynamic an adanalysis by submitting the building model to an ensemble of the ground motions. The determined seismic response is very sensitive to the ground motion characteristics, and the examination is performed for more than one ground motion record.

To execute the non-linear dynamic analysis, the equation recommended by the Newmark's method (Chopra 1995) may be properly prolonged. Structured on review of analytical techniques, the non-linear dynamic analysis method is implemented for the analytical study because of its precision and effectiveness in identifying the inelastic seismic response of a system exposed to the ground motion data. The evaluation of previous research works show that the past research works have adopted static methods in majority for simplicity. However, the present research works in majority have adopted dynamic analysis (especially non-linear dynamic analysis) to accomplish much better precision to estimate the realistic seismic demands. Moreover, different seismic design codes prescribe dynamic analysis for medium and tall structures and it has been applied by recent analysts as well (Karavasilis et al. 2008; Panda and Ramachandra 2010). Therefore, non-linear dynamic analysis method has been implemented in the present study to determine the seismic response of the building models.

Integration Method using Nonlinear Modal Equations

The modal analysis approaches to nonlinear systems have been and continue to be an attractive idea, mainly because of the ability of these approaches to give fairly accurate solutions when only a few modes are considered, and because they provides directly the mode shapes and natural frequencies of the analyzed system, information that, even for nonlinear systems, is usually desirable to have. The equation of motion for a system with nonlinear properties when subjected to an earthquake ground acceleration may be written as

$$M\ddot{X}(t) + C\dot{X}(t) + KX(t) + R(t) = -M\{L\}\ddot{x}_g(t)$$

(1)

where *M, C* and *K* are the mass, damping and initial stiffness matrix; *X (t),* $\dot{X}(t)$ and $\ddot{X}(t)$ are the displacement, velocity and acceleration vectors;$\{L\}$ is the influence vector accounting the direction of the earthquake excitation; $\ddot{X}_g(t)$ the ground acceleration, and *R(t)* is the nonlinear residual force vector. If the physical coordinates of Eqn. 1 are transformed to modal coordinates assuming the diagonal modal damping, the typical modal equation of the motion can be obtained as

$$\ddot{q}_n(t) + 2\varsigma_n\omega_n\dot{q}_n(t) + \omega_n^2 q_n(t) = \bar{f}_n(t) \qquad (n = 1, 2, \cdots, l)$$

(2)

where $q_n(t)$, $\dot{q}_n(t)$ and $\ddot{q}_n(t)$ the modal displacement, velocity and acceleration for the n-th mode; ζ_n z and ω_n are the corresponding damping ratio and natural frequency; and $\overline{f_n}(t)$is the modal load which includes the nonlinear residual force which depends on the unknown concurrent structural response. Hence, the above modal equations can be solved iteratively at each time by updating the nonlinear residual force.

Substructuring Method

Nonlinear damage is defined as the case when the initially linear-elastic structure behaves in a nonlinear manner after the damage has been introduced. One example of nonlinear damage is the formation of a fatigue crack that subsequently opens and closes under the normal operating vibration environment.

The substructuring method is probably effective in the model improving of large-scale structures and associated purposes. In these research, the global structure is divided into smaller and more controllable substructures. The substructures are assessed independently to acquire their specified solutions, which are then built to restore the options to the global structure by imposing constraints at the interfaces.

In this study, to serve as a set of vectors with which to create the coupled system behavior within the sub-structures, the fixed interface normal modes are considered. Figure 7 shows the substructure model of locally damage bridge pier structure.

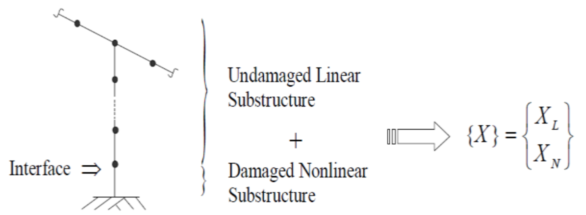

Figure 7. Substructure model of locally damage bridge pier structure

Denoting the properties associated to the linear and nonlinear substructures by the subscripts L and N, respectively, the equation of motion in Eqn. 3.1 can be written as follow:

$$\begin{bmatrix} M_{LL} & M_{LN} \\ M_{NL} & M_{NN} \end{bmatrix} \begin{Bmatrix} \ddot{X}_L \\ \ddot{X}_N \end{Bmatrix} + \begin{bmatrix} C_{LL} & C_{LN} \\ C_{NL} & C_{NN} \end{bmatrix} \begin{Bmatrix} \dot{X}_L \\ \dot{X}_N \end{Bmatrix} + \begin{Bmatrix} R_{LN} \\ R_{NN} \end{Bmatrix} =$$

(3)

In order to reduce the problem size using mode superposition in linear substructure, the following coordinate transformation can be applied:

$$\begin{Bmatrix} X_L \\ X_N \end{Bmatrix} = \begin{bmatrix} \Phi_{LL} & \psi & -K_{LL}^{-1} K_{LN} \\ 0 & 0 & I \end{bmatrix} \begin{Bmatrix} q_L \\ p \\ X_N \end{Bmatrix}$$

(4)

where L q is the linear modal response vector and p is the modal correction vectors to compensate the influence of the truncated modes. The modal correction vectors can be created using a mathematically consistent Rayleigh-Rits approximation where the assumed Ritz basis vectors are derived using the special force truncation vector. The nonlinear behavior of locally damaged structural dynamic systems can be obtained by solving the nonlinear modal equation with transformation of Eqn. 1 using above Eqn. 3.4. Various substructuring methods differ from each other by the determination of the reduction matrix, T.

Modal Sorting Method

When the modal analysis is used for the structural dynamic systems, the truncation of modes may cause significant difficulty in obtaining reasonable dynamic response (D'Aveni, A. and Muscolino, G. 2001), particularly for the locally damaged behavior. In this study, a modal sorting technique is proposed to select the modes with larger contribution to the DOF near the damaged location. The *j*-th modal contribution to the *i*-th DOF Ξ_{ij} under earthquake load nay be evaluated as

$$\Xi_{ij} = \phi_{ij} \Gamma_j S_j$$

(5)

where ϕ_{ij} is the *j*-th eigenvector at the *i*-th DOF, Γ_j is the modal participation factor at the *j*-th mode; S_j is the deformation response spectrum of the ground motion at the *j*-th natural period at $\omega = \omega_j$. The modes are sorted by the order of the magnitudes of those modal contribution values for a specific DOF. With the sorted modal vectors, the global displacement vector can be obtained as

$$X(t) = \tilde{\Phi} Q(t)$$

(6)

where $\overline{\Phi}$ is the matrix of the sorted eigen-vectors matrix, and Q(t) is the corresponding modal displacement vector. Then, as like the substructuring method, by using above Eqn. 6, it is possible to obtain the nonlinear modal equation and to apply the modal integration method to obtain the nonlinear dynamic responses.

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SAMPLE STUDIES FOR HYSTERIC BEHAVIOR

Simplified Pier Model

This example is a simplified bridge model with a pier in the middle of the deck. It is assumed that the deck is supported by a single bridge piers and earthquake load is applied, so the effect of the deck may be considered as an additional lumped mass on the top of the pier. The pier is fixed the ground level. The earthquake acceleration is applied to the pier in the form of body force so that the relative displacement responses of the pier can be obtained directly. As mentioned above, for simplicity, axial load is not included in this study. The pier is modeled by 10 beam elements. The total number of DOFs is 30. The geometric and sectional properties of the pier are shown in Figure 8(a). The nonlinear hysteretic behavior is assumed to be occurred at the bottom of the pier during the earthquake. Figure 8(b) and (c) show the applied ground acceleration and the relation between the moment and the curvature in the bottom area of this model. To corroborate the possible differences in predictions between the models for a specific earthquake, EL CENTRO 1940 N-S motion (NS, peak ground acceleration (PGA) = 0.139g, 1979) was used as input for the three models and the displacement time history was computed and compared. A scaled El Centro earthquake is used. Figure 9 shows the time-scaled time histories used for simulation: NS 1940 El Centro record,

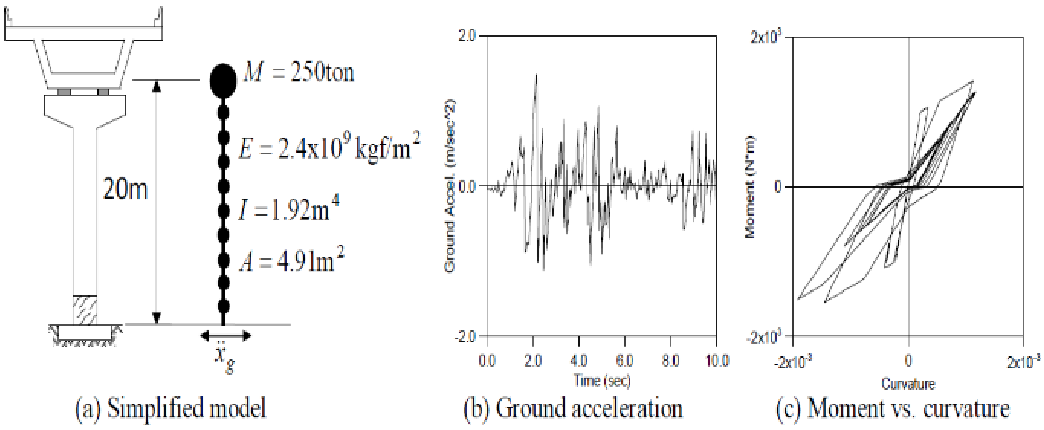

Figure 8. Cantilever model of bridge pier subjected earthquake excitation

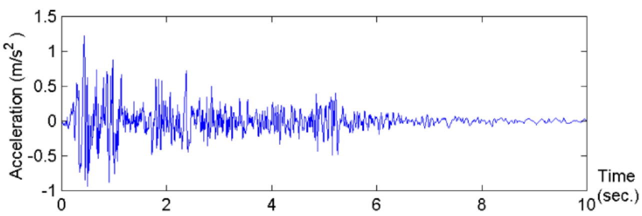

Figure 9. Time-scaled time histories used for simulation: NS 1940 El Centro record

The nonlinear hysteretic responses are obtained by the nonlinear modal integration, substructuring and modal sorting method and are compared with the response by the direct step by step integration method (Wilson-θ, θ=1.4). The first natural frequency is obtained as 0.41 Hz, while the damping ratio is assumed as 5% viscous damping for each mode.

The nonlinear parameters M_y, α, β and γ in this model are assumed to have the values of 1,000(tonf·m), 0.1, 0.7 and 0.02, respectively. As several design codes require at least 90 percent of the modal participating mass is included in the calculation of response for each principal direction, the first mode (96% modal participating mass) and lower 10 modes (100% modal participating mass) are considered to compare the nonlinear behaviors. In using the sorting method, the near node of damaged member is taken as the sorting point. When the first mode is used, three methods give a little different result compared to the result obtained with direct step by step integration method as shown in Figure 10. However, when the lower 10 modes are included and one modal correction mode is included in substructuring method, all of three methods give good results as shown in Figure 11. Especially, the modal sorting and substructuring methods give a better accurate result than the using nonlinear modal integration method. From this example, it is found that the substructuring method with modal correction vectors can effectively applied to the locally damaged structural dynamic systems and improves the accuracy of nonlinear response.

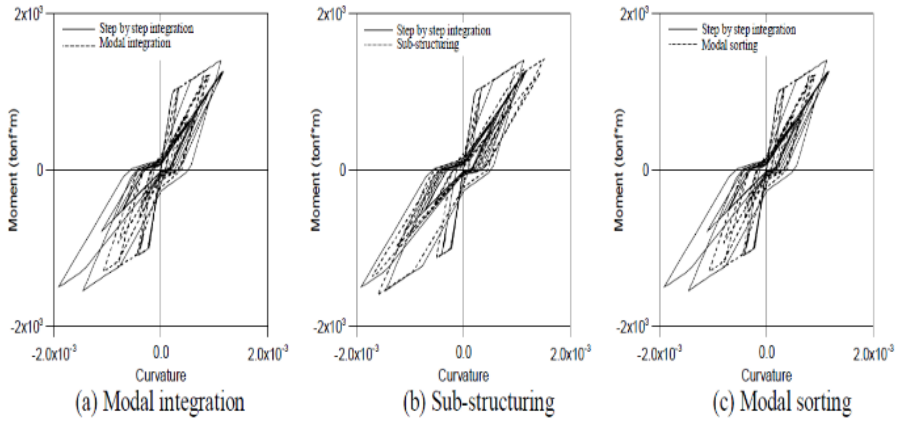

Figure 10. Moment vs. curvature with the first mode

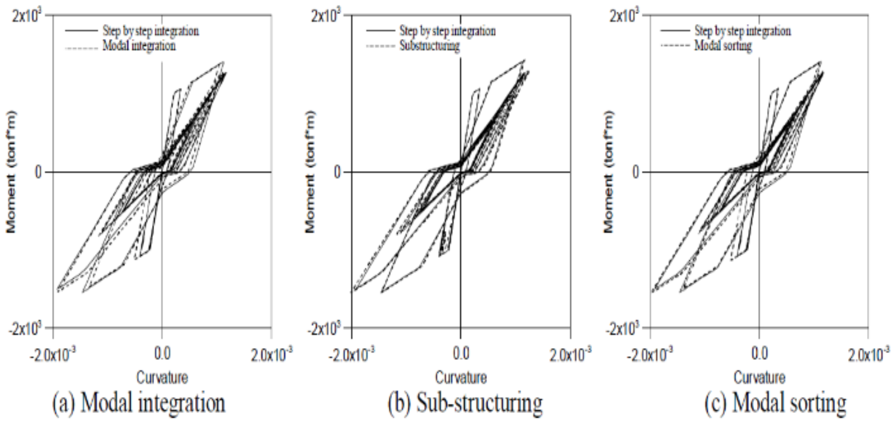

Figure 11. Moment vs. curvature with the lower 10 modes

Continuous Bridge Model

This example is a four span continuous bridge model subjected to an earthquake load. It is assumed that the deck and the pier have uniform cross-sections. The bridge structure is modeled by 3D frame elements as in Figure 12. Table 1 shows the materials properties of specimens. The bottom of the bridge pier is assumed to be damaged by a scaled El Centro earthquake (NS, PGA = 0.4g, 1940) acting in the transverse direction of

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bridge. Figure 13 shows the displacement and time histories used for simulation: NS 1940 El Centro record and Figure 14 shows the angular velocity responses of the RPS under El Centro Earthquake.

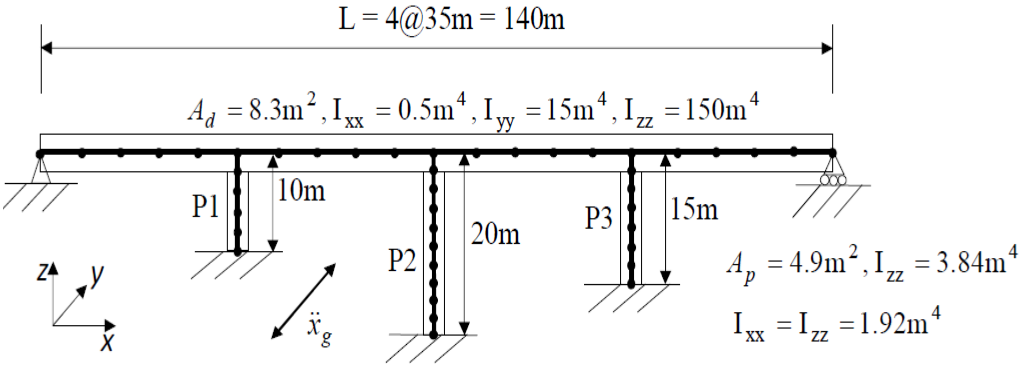

Figure 12. Continuous bridge model

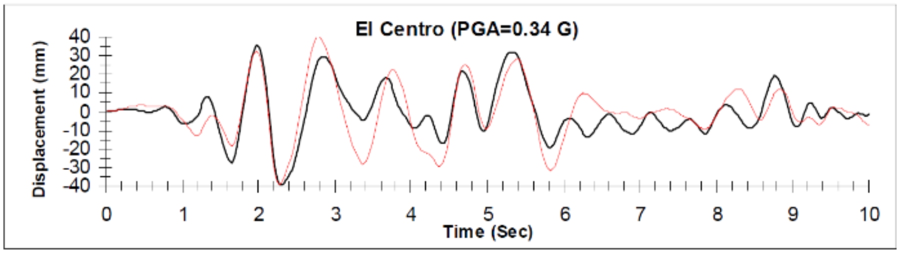

Figure 13. Displacement and time histories used for simulation: NS 1940 El Centro record.

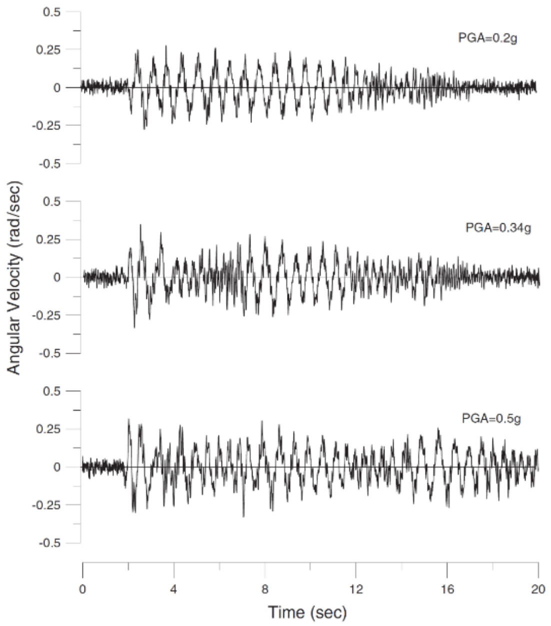

Figure 14. Angular velocity responses of the RPS under El Centro Earthquake.

Table 1. Materials properties of specimens

Concrete	Piers
Unit weight, kN/m³	25
Compressive strength, MPa	27
Elastic modulus, MPa	24,648
Steel reinforcement	Yielding strength, f_y = 400 MPa

The six DOF's are assigned at each node and the total number of DOF's is 231. The nonlinear parameters M_y, *α,* β and *γ* are assumed to have the values 1,000 (tonf·m), 0.1, 0.7 and 0.5, respectively.

The nonlinear hysteretic behaviors in pier 2 and 3 which are obtained by three nonlinear analysis methods and compared with the results obtained by the direct step by step integration method like as above simplified pier model. The fundamental natural frequency of this model is obtained as 2.56 Hz. The viscous damping ratio is assumed as 5% for each mode. At least the lower 25 modes should be included to obtain 90% of the modal participation mass for the appropriate modal analysis. The input ground acceleration is shown in Figure 15(a). Figure 15(b) and (c) are the relationships of moment vs. curvature subject assumed earthquake loading in Pier 2 and Pier 3, respectively.

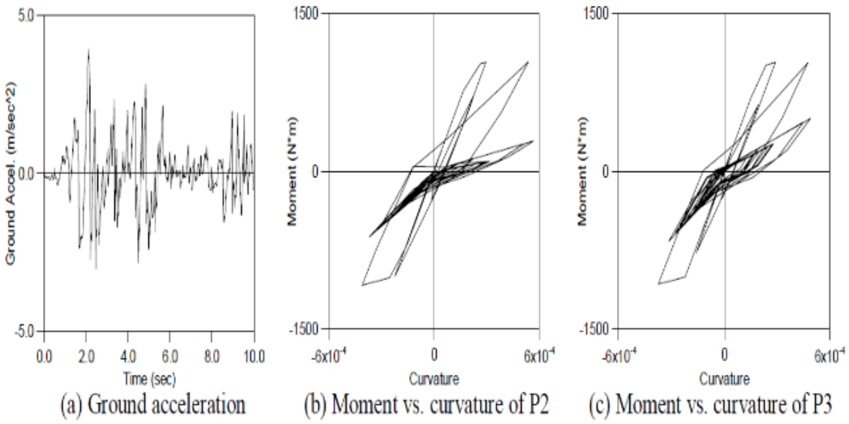

Figure 15. Ground acceleration and Moment vs. curvature of piers

In this example, one modal correction vector is included in substructuring method and the node located in the top of each pier which is assumed to be damaged is taken as the sorting point in sorting method. In the Pier 2, the modal integration method gives less accurate than the results of other two methods as shown in Figure 16. Especially, the more accurate analysis result can be obtained from the analysis using the modal sorting method compared with the other methods.

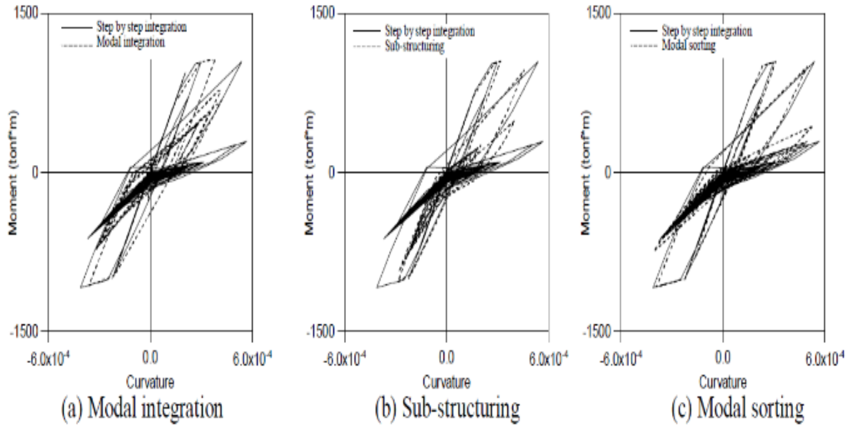

Figure 16. Moment vs. curvature of P2 with 25 modes

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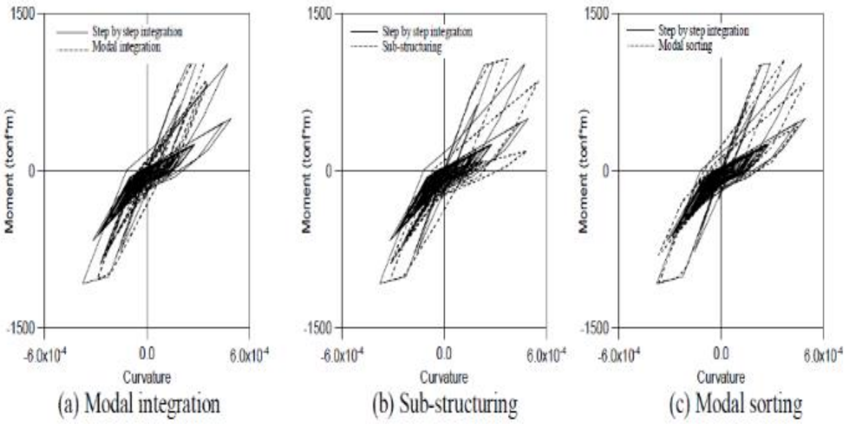

Figure 17. Moment vs. curvature of P3 with 25 modes

However, the analysis results in Pier 3 have a little difference from the result by direct step by step integration method as shown in Figure 17. This may be the result from the restraint by boundary condition and the influence of distortion of higher modes in damage area. As similar to the Pier 2, using the modal integration method also shows insufficient accuracy to describe the hysteretic behavior of locally damaged structure. In spite of somewhat discrepancies in these analysis results, however, it is even expect that the proposed modal sorting method can be used to reduce the problem size effectively and can be applied to the analysis of the locally damaged structural systems together with the substructuring method.

CONCLUSIONS

Some efficient modal methods to analyze the nonlinear hysteretic behavior of locally damaged RC structures are proposed and compared with the time integration schemes which are usually used in the analysis of nonlinear structural dynamic systems having inelastic behavior. The hysteretic model is reproduced using the modified Takeda model, in which important nonlinear characteristics of the damaged RC members, such as stiffness degradation, pinching effect and strength deterioration are included with a limited number of parameters. To verify the efficiency of proposed methods, the bridge structures are assumed to have some damages in the bottom of piers during severe earthquake and modal integration method, substructuring method and modal sorting method have applied to analyze the nonlinear hysteretic behavior and to compare with the result by Wilson θ method.

From the verification, it is found that the modal integration method has less accuracy than the other two methods and the modal sorting and substructuring methods are expected to give reasonable accuracy with limited modes in the analysis of locally damaged structural dynamic systems.

- The utilize of simple designs may generate decent estimations if the appropriate geometry is selected.
- The experimental outcomes of the total scale bridge testing, and the companion element tests, demonstrated that bridge actions is extremely reliant of the degree of displacement.
- When primarily modeling a bridge structure, there is a temptation to presume that the foundation structure is strong and stiff therefore presuming full fixity at the pile cap level. Nevertheless, the inclusion of equal soil springs and masses to be able to design soil-structure the interaction is extremely suggested.

REFERENCES

1. Banon, H., Biggs, J. M., and Irvine, H. M., (1981). Seismic damage in reinforced concrete frames. Journal of Structural Engineering, ASCE, Vol. 107, No. ST9, 1713-1729.
2. Bazant, Z. P., and Oh, B. H., (1983). Crack band theory for fracture of concrete. Materials and Structures (RILEM, Paris), Vol. 16, 155-177.
3. Chai, Y. H., Romstad, K. M., and Bird, S. M., (1995). Energy-based linear damage model for high-intensity seismic loading. Journal of Structural Engineering, Vol. 121, No. 5, 857-864.
4. Chopra, A. K., (1995). Dynamics of structures: theory and applications to earthquake engineering. Prentice Hall, New Jersey.
5. Chung, Y.S., Meyer, C. and Shinozuka, M. (1989), Modeling of Concrete Damage, ACI Structural Journal, 86(3), 259-271.
6. Comi, C., and Perego U., (2001). Fracture energy based bi-dissipative damage model for concrete. International Journal of Solids and Structures, Vol. 38, No. 36-37, 6427-6454.
7. Criesfield, M. A., (1982). Local instabilities in non-linear analysis of reinforced concrete beams and slabs. Proceedings of Institute of Civil Engineers, Part 2, Vol. 73, 135-145.
8. Crisfield, M. A., (1996). Nonlinear analysis of solids and structures, Volume 1: Essentials, Willey & Sons, New York.
9. D'Aveni, A. and Muscolino, G. (2001), Improved dynamic correction method in seismic analysis of both classically and non-classically damped structures, Earthquake Engrg. and Struct. Dynamics, 30, 501-517
10. Dikens, J.M., Nakagawa J.M., and Wittbrodt M.J. (1997), A critique of mode acceleration and modal truncation argumentation methods for modal response analysis, Computer & Structures, 62:6, 985-998
11. Fajfar, P., and Gaspersic, P., (1996). N2 method for the seismic damage analysis of RC buildings. Earthquake Engineering and Structural Dynamics, Vol. 25, No. 1, 31-46.
12. Ghobarah, A., Abou-elfath, H., and Biddah, A., (1999). Response-based damage assessment of structures. Earthquake Engineering and Structural Dynamics, Vol. 28, No.1, 79-104.
13. Hillerborg, A., Modeer, M., and Pertersson, P. E., (1976). Analysis of crack formation and crack growth in concrete by means of fracture mechanics and finite element. Cement and Concrete Research, Vol. 6, 773-782.
14. Hindi, R. A., and Sexsmith, R. G., (2001). A proposed damage model for RC bridge columns under cyclic loading. Earthquake Spectra, Vol. 17, No. 2, 261-290.
15. Hindi, R. A., and Sexsmith, R. G., (2004). Inelastic damage analysis of reinforced concrete bridge columns based on degraded monotonic energy. Journal of Bridge Engineering, ASCE, Vol. 9, No. 4, 326-332.
16. Jefferson, A. D., (1999). A multi-crack model for the finite element analysis of concrete. Proceedings of BCA Concrete Conference, 275-286.
17. Jefferson, A. D., (2002a). Local plastic surfaces for cracking and crushing in concrete. Journal of Materials: Design and Application, Vol. 216(L), 257-266.
18. Jefferson, A. D., (2003a). Craft, a plastic-damage-contact model for concrete. Part I – Model theory and thermodynamics. International Journal of Solids and Structures, Vol. 40, No. 22, 5973-5999.
19. Jefferson, A. D., (2003b). Craft, a plastic-damage-contact model for concrete. Part II – Model implementation with implicit return-mapping algorithm and consistent tangent matrix. International Journal of Solids and Structures, Vol. 40, No. 22, 6001-6022.
20. Jeong, G. D., and Iwan, W. D., (1988). The effect of earthquake duration on the damage of structures. Earthquake Engineering and Structural Dynamics, Vol. 16, No. 8, 1201-1211.
21. Jirasek, M., and Bazant, Z. P., (2002). Inelastic analysis of structures. John Willey & Son, New York.
22. Karavasilis, T. L., Seo, C.-Y., and Ricles, J. (2008). HybridFEM: A Program for Dynamic Time History Analysis of 2D Inelastic Framed Structures and Real-Time Hybrid Simulation. Bethlehem, PA.
23. Kim, T. -H., Lee, K. -M., Chung, Y. -S., and Shin, H. M., (2005). Seismic damage assessment of reinforced concrete bridge columns. Engineering Structures, Vol. 27, No. 4, 576-592.
24. Kunnath, Sashi, K.; El-Bahy, Ashraf; Taylor, Andrew; and Stone, William, Cumulative Seismic Damage of Reinforced Concrete Bridge Piers, Technical Report NCEER-97-0006, National Center for Earthquake Engineering Research, September 1997, 147 pages

*

25. Kwan, W. -P., and Billington, S., L., (2003). Unbonded posttensioned concrete bridge piers. Part II - Seismic analyses. Journal of Bridge Engineering, ASCE, Vol. 8, No. 2, 102-111.

26. Lai, S.S., Will, G.T. and Otani, S. (1984) "Model for inelastic biaxial bending of concrete members", ASCE Journal of Struct. Engrg., V.110, 11, 2563-2584.

27. Lee, K.J. and Yun, C.B. (2008), Parameter identification for nonlinear behavior of RC bridge piers using sequential modified extended Kalman filter, Smart Structures and Systems, 4:3, 319-342.

28. Ma, G., Hao, H., and Lu, Y., (2003). Modelling damage potential of high-frequency ground motions. Earthquake Engineering and Structural Dynamics, Vol. 32, No. 10, 1483-1503.

29. Meyer, B., Armijo, R., De Chabalier, J., Delacourt, C., Ruegg, J., Acache, J., Brioke, P., Papanastassiou, D., 1996. The 1995 Grevena (Northern Greece) earthquake: fault model constrained with tectonic observations and SAR interferometry. Geophys. Res. Lett. 23, 2677 – 2680.

30. Mork, K.J. (1991), "Response Analysis of Reinforced Concrete Structures under Seismic Excitation", Earthquake Engineering and Structural Dynamics, 23(1), 33-48.

31. Oliver, J., (1989). A consistent characteristic length for smeared crack models. International Journal for Numerical Methods in Engineering, Vol. 28, 461-474.

32. Panda S K and Ramachandra L S 2010 Buckling of rectangular plates with various boundary conditions loaded by non-uniform inplane loads. Int. J. Mech. Sci. 52: 819–828

33. Park, J.Y., and Ang, A.H.S., (1985). Mechanistic seismic damage model for reinforced concrete. Journal of Structural Engineering, ASCE, Vol. 111, No. 4, 722-739.

34. Popov, E.P., Bertero, V.V. and Krawinkler, H. (1972), "Cyclic behavior of three reinforced concrete flexural members with high shear", Earthquake Engrg. Research Center Report No. EERC 72-5, Univ. of California, Berkeley, Calif

35. Roufaiel, M.S.L. and Meyer, C. (1987), Analytical modeling of hysteretic behavior of R.C. Frames, J. Struct. Engrg., ASCE, 113:3, 429-443

36. Rahnama, M. and Krawinkler, H. (1993), "Effect of Soft Soils and Hysteresis Models on Seismic Design Spectra", John A. Blume Earthquake Engineering Research Center Report No. 108, Department of Civil Engineering, Stanford University.

37. Saaidi, M. And Sozen, M.A. 1979, "Simple and Complex Models for Nonlinear Seismic Response of reinforced Concrete Structures," SRS No.465, U of Illionis, Urbana.

38. Stephens, J. E., and Yao, J. T. P., (1987). Damage assessment using response measurements. Journal of Structural Engineering, Vol. 113, No. 4, 787-801.

39. Sucuoglu, H. and Erberik, A. (2004), "Energy-based Hysteresis and Damage Models for Deteriorating Systems", Earthquake Engineering and Structural Dynamics, 33(1), 69-88.

40. Takeda, T., Sozen, M. A., and Nielsen, N. N., (1970). Reinforced concrete response to simulated earthquakes. Journal of Structural Engineering, ASCE, Vol. 96, No. 12, 2557-2573.

41. Williams, M. S., and Sexsmith, R. G., (1995). Seismic damage indices for concrete structures: a state-of-the-art review. Earthquake Spectra, Vol. 11, No. 2, 319-349.

42. Williams, M. S., Villemure, I., and Sexsmith, R. G., (1997). Evaluation of seismic damage indices for concrete elements loaded in combined shear and flexure. ACI Structural Journal, Vol. 94, No. 3, 315-322.

43. Youssef, M., and Ghobarah, A., "Modelling of RC Beam-Column Joints and Structural Walls," Journal of Earthquake Engineering, V.5, No. 1, 2001, pp. 93-111.

44. Zienkiewicz, O. C., and Taylor, R. L., (1991). The finite element method. 4th edition. Vols. 1 and 2, Mc.Graw-Hill, London.