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DYNAMIC RESPONSE OF A REINFORCED CONCRETE FRAME COMPARED WITH OBSERVED EARTHQUAKE DAMAGE

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ABSTRACT

This paper presents the results of an investigation on the seismic performance evaluation of reinforced concrete buildings. As an example, a building, damaged during the 1985 Michoacan Earthquake in Mexico, is evaluated using nonlinear dynamic analyses and an improved version of the Capacity Spectrum Method. The comparison of the performance parameters obtained from both types of analyses with observed damage shows good correlation.

KEYWORDS

Reinforced concrete buildings; structural damage; seismic intensity; damage indices; nonlinear dynamic analysis; capacity spectrum; seismic performance evaluation.

INTRODUCTION

Current seismic design philosophies take measures to prevent the collapse of building structures in the case of severe earthquakes while, at the same time, allowing them to undergo important inelastic deformations and to dissipate seismic energy through hysteresis. Unfortunately, the performance of building structures at such degrees of severity is still evaluated by simplified linear elastic methods based on the assumption that the structure will behave in a similar manner as in the nonlinear range. This assumption has been questioned, even for single degree of freedom, (SDOF), structures, a fact showing the need for reliable evaluation procedures which take into account the relevant aspects of the nonlinear behaviour of structures while keeping simplicity in mind.

During the last decade a variety of methods for seismic evaluation methods has been proposed, ranging from those based on nonlinear dynamic analyses to those based on simplified nonlinear static pushover analyses. Parallel to this, a number of computer programs have been generated to approximate the nonlinear response of building structures with different behaviour models for the structural elements and different types of loading. In most of these attempts, nonlinearity in structural members has been assumed to be entirely due to simple bending, neglecting the fact that observations of damage in existing structures and in experimental and analytical models frequently show that other internal forces, such as axial and shear forces, may play a significant role in the seismic damage phenomenon.

The objective of this investigation is to examine the capabilities of conventional and newly proposed models to reproduce the observed damage in actual buildings, and to present and evaluate an improved version of the Capacity Spectrum Method, (CSM), as a powerful, yet simple, tool for the seismic performance evaluation of regular buildings.

SEISMIC RESPONSE EVALUATION OF A REAL BUILDING

Description of the Structure

The building used in this paper is the administrative offices of the Metro System (STC), a 10-storey, regular building with a basement, located in the old lake bed zone of Mexico City. The detailed description of the structure and the damage caused by the 1985 Michoacan earthquake are reported by Meli and Avila, 1988. Fig. 1a shows the configuration of a typical interior frame and the observed damage.

Description of the Input Motion

At the time of the 1985 Michoacan earthquake there were no recording instruments in the vicinity of the STC building. However, due to its proximity to the SCT site and the similarity in soil conditions, the SCT records are used as seismic excitation in this paper (Ye, 1994).

Nonlinear Dynamic Analysis

In this investigation several nonlinear analysis programs were evaluated by Ayala and Ye, 1995; finding the most adequate program to be CANNY, (Li, 1993). The program offers the possibility of using up to seventeen hysteretic behaviour models for RC and/or steel members. Some of these models are newly developed such as an improved version of the Park, Reinhorn and Kunnath model (Park *et al*, 1987) and a multi-spring model (Li and Otani, 1995) able to reproduce the interaction between bi-directional bending and varying axial load.

The Capacity Spectrum Method

The CSM is an approximate nonlinear static method that falls between dynamic and static analyses as it is a combination of a pushover analysis and a response spectrum evaluation. The method was first introduced by Freeman, 1975. Since then it has been improved and modified making it a simple and reliable method of evaluation. Recently, this method has attracted the attention of structural engineers and researchers and is a subject of debate in code committees for possible inclusion in future performance design codes, NIST, 1994.

One of the main advantages of this method is that the capacity spectrum is a property of the structure once a pattern of seismic equivalent static loading is chosen. It is entirely independent from the seismic demand. Therefore, once computed, the capacity spectrum can be checked against any seismic demand. This is not the case for nonlinear time history analyses where the whole structure needs an independent analysis for each selected earthquake.

The arbitrariness of the method in choosing damping ratios for the demand spectrum and in determining the performance point is one of its drawbacks as the consideration of equivalent responses of a nonlinear and a viscoelastic SDF structures has not been proved. A way to eliminate this drawback, introduced by Kunnath *et al*, 1996, and modified slightly by the authors to better reproduce a mean equivalent fundamental inelastic period as opposed to the maximum period given by the original method, is summarized in the following steps:

1. Draw the capacity spectrum in the (S_a,S_d) or (S_a,T) spaces, together with the elastic spectrum (usually with 5% damping).
2. Draw a straight line along the initial "elastic" part of the capacity spectrum until the elastic demand is reached. Record the the fictitious elastic capacity, S_{a_e}, given by the value of S_a

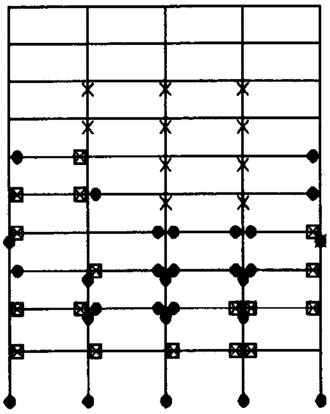

Fig. 1a. Observed damage.

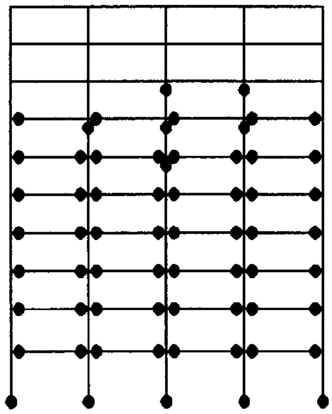

Fig. 1b. Model I.

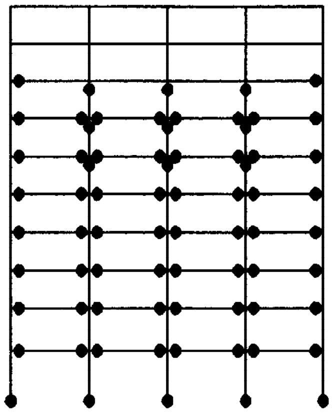

Fig. 1c. Model II.

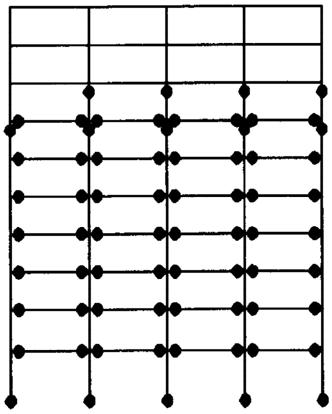

Fig. 1d. Model III.

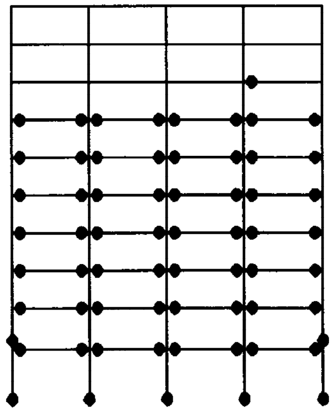

Fig. 1e. Model IV.

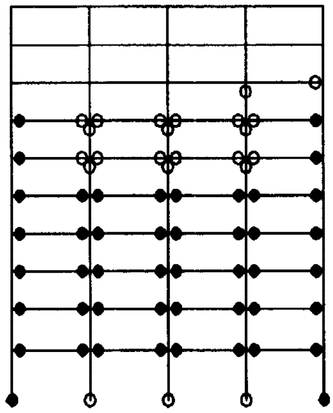

Fig. 1f. CSM.

● Hinge ⊠ Hinge with crushing × Light diagonal cracking O Hinges about to form

at the intersection and the S_a at first yield, S_{ay}, which represents the actual elastic capacity of the structure.

3. Calculate the strength reduction factor, $C_y = \frac{S_{ay}}{S_{ae}}$.
4. Compute the inelastic strength spectrum using C_y as the strength reduction factor for the SDOF structures considered in this process.
5. Find the performance point given by the intersection of the capacity spectrum with the inelastic strength spectrum. Once the (S_{ap},S_{dp}) or (S_{ap},T_p) at the performance point is defined the determination of the equivalent base shear, storey shears, storey displacements, etc, follows the same procedure as the original method.
6. Calculate the inelastic period.

The original method, as presented by Freeman, 1975, ignores the contribution of higher modes, a drawback if these modes have are important as is the case of tall buildings. This topic is under investigation by Tayebi, 1996.

Structural Modelling for Response Analysis

Four different models of behaviour, are evaluated in this research using them in the nonlinear seismic analyses of the STC building: Model I, 2-D element with Takeda hysteretic rule; Model II, 2-D element with a general hysteretic rule; Model III, 2-D nonlinear multi-spring element and Model IV, 3-D nonlinear multi-spring element. Model II uses an improved version of the Park *et al*, 1987; original element which allows for the change in stiffness at unloading. The multi-spring elements used in Models III and IV simulate the interaction between biaxial bending and axial force.

The modelling of the frame and the used properties are described in Ayala and Ye, 1995. In the models, beams and columns are considered bending only elements idealized as linear elements with nonlinear

Table 1. Results of the seismic evaluation.

Model	D_{max} (cm)	D/H (%)	Q_{max} (kN)	Q_{max}/W	T_o (sec)	T_f (sec)	T_{max} (sec)	D_{Park}	D_T
I	60.08	1.54	2572	0.142	1.53	2.48	11.78	0.658	0.800
II	50.89	1.30	2542	0.140	1.44	2.02	9.39	0.643	0.454
III	50.42	1.29	2654	0.146	1.36	1.95	10.29	*	0.434
IV	60.85	1.56	2639	0.145	1.45	2.10	8.89	*	0.512
CSM1	41.06	1.05	2445	0.135	1.44	2.30	6.9	0.390	0.673
CSM2	42.09	1.08	2393	0.132	1.53	2.32	7.1	0.337	0.665

rotational springs at both ends plus axial force for columns. For Models III and IV columns are idealized as multi-spring elements with biaxial bending and axial force interaction. In Model IV, the shear walls in the N-S direction are idealized as linear wide columns including shear deformations, and the analysis is carried out using all three components of the SCT record.

PRESENTATION AND ANALYSIS OF RESULTS

Table 1 summarizes the response parameters of interest in this paper. In it D_{max} is the maximum roof displacement, D/H the roof displacement to total height ratio, Q_{max} the maximum base shear, W the weight of the structure, T_0, T_f and T_{max}, respectively the initial, final and maximum instantaneous period of the structure, D_{Park} and D_T the Park and Ang, 1985, and the Ayala and Ye, 1995, damage indices, respectively. An ultimate period of 2.74 sec. is used in D_T as defined by the point when the design base shear is exceeded.

Figs. 1b-1e show the hinge distribution resulting from the dynamic analysis with the four models and Fig. 1f from the CSM application. When comparing the distributed damage obtained from different models, it appears that Model II (Fig. 1d) is the closest to the observed damage. However, none could predict the hinges in the interior columns of the second to fourth floor. This is explained by the neglection of overstrength in beams and shear-bending interaction in columns.

Figs. 2a-2c depict the distribution of maximum floor displacements, interstorey drift and storey shears for the four models. These figures show that models II and III give similar results in terms of the considered response parameters. This is not the case for Model I which consistently gave larger floor displacements and drifts. However, all three 2-D models gave similar maximum storey shears. Regarding Model IV, the building being regular and fully symmetric, one would expect its response to be similar to that of Model III. This does not happen as initial vertical load forces and bidirectional bending have a strong influence on the response which is found to be larger than that of corresponding 2-D model. It may be argued that the results of Model II, shown in Fig. 2a, reproduce the 3-D behaviour due to their apparent similarity in displacement response. Unfortunately, this is only a coincidence not seen when shear forces are compared.

The analyses results show that Model II is the most adequate for the 2-D frame and is consequently used for the rest of this investigation. For this model the maximum roof displacement was about 1.3% of the total height, a value exceeding the code prescribed limit state of 1.2% by .8%. Fig. 2b shows that the code prescribed interstorey drift of 0.6% was exceeded in all stories except the top one, This may be the reason that most of the structural and mainly nonstructural damage is concentrated in the intermediate floors [2 to 8]. Observed damage corroborates this conclusion.

Additional evidence of the correlation of evaluated performance and observed damage is found in Fig. 2c where the design base shear of 1415 kN (0.0787W) was exceeded by far in all of the first eight stories.

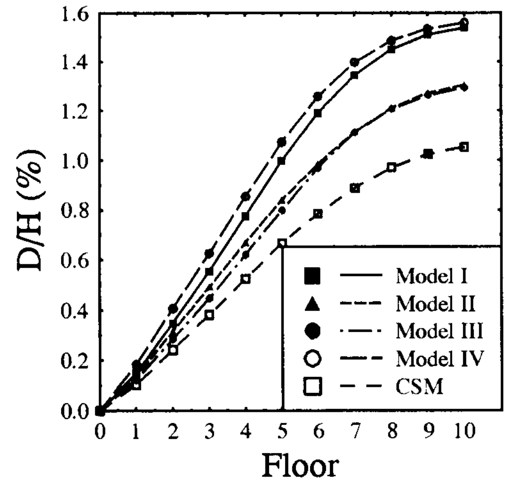

Fig. 2a. Maximum Storey displacements.

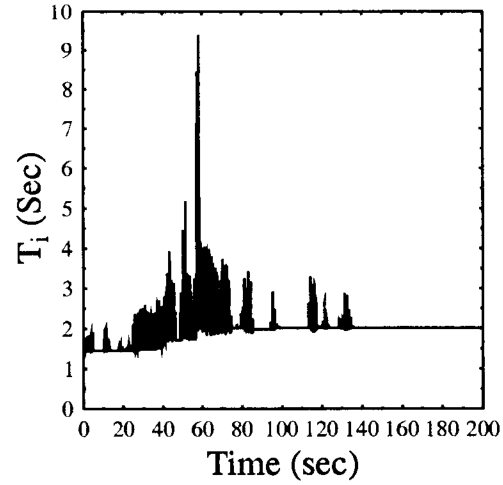

Fig. 3a. T_i time history.

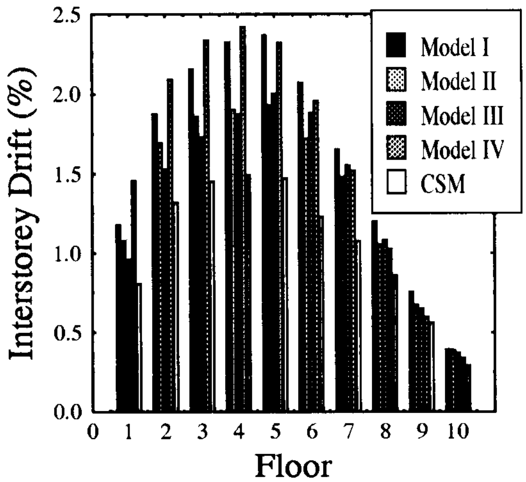

Fig. 2b. Maximum storey drifts.

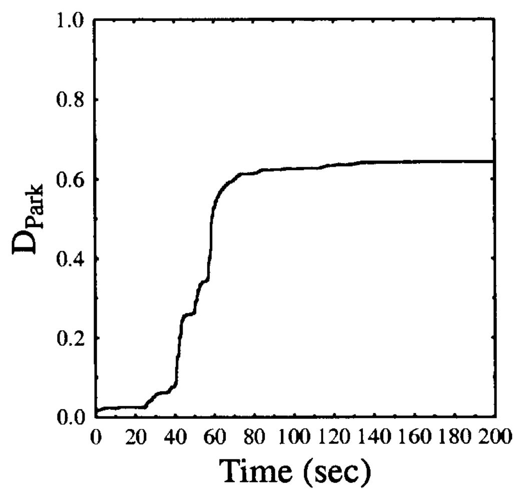

Fig. 3b. D_{Park} time history.

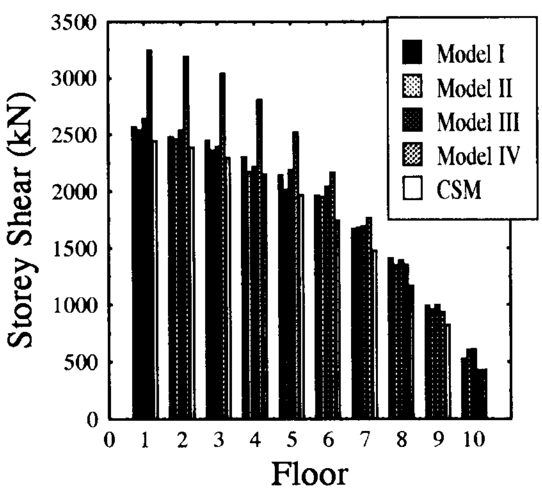

Fig. 2c. Maximum storey shears.

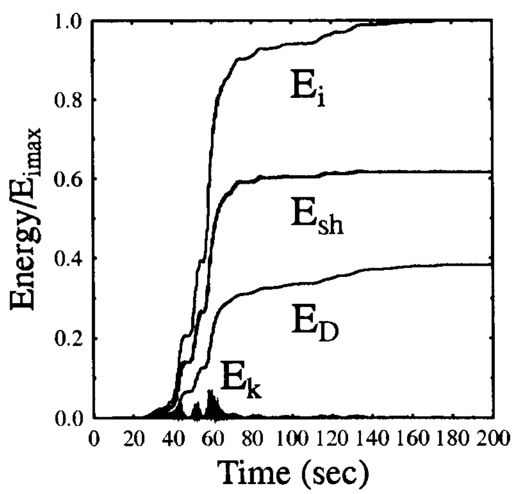

Fig. 3c. Energies time histories.

This fact shows that the hinges formed in floors 2-5 (Fig. 1a) were shear hinges or highly influenced by shear. It is thought that if the used models had incorporated yielding and failure of elements due to shear, the results of the dynamic analysis would have better predicted the observed damage.

Fig. 3a shows the time history of the fundamental instantaneous period of the structure obtained in Model II, T_i. It may be observed that after the intense part of the earthquake action has ceased (i.e. 138 sec), T_i reaches a constant value much smaller than T_{max}, [not relevant as a global damage indicator as it only occurs once] and larger than T_0. This final period is used in this paper as the basis of D_T;

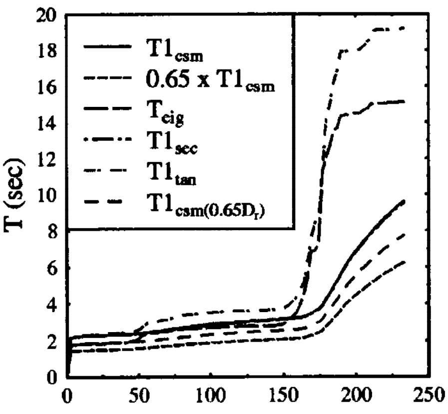

Fig. 4. Different period definitions.

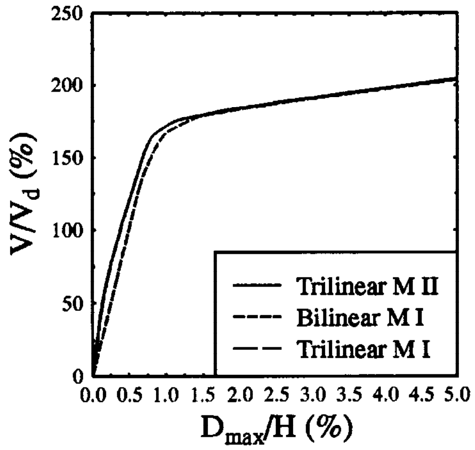

Fig. 6. Pushover curve.

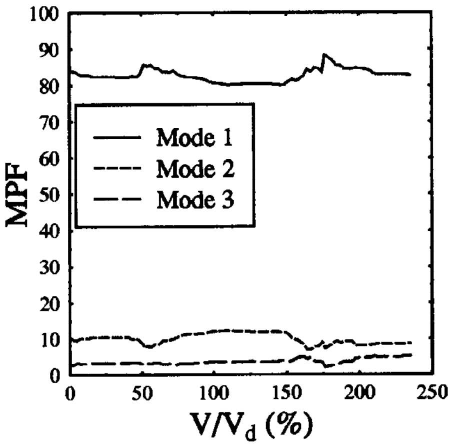

Fig. 5. Evolution of MPF's.

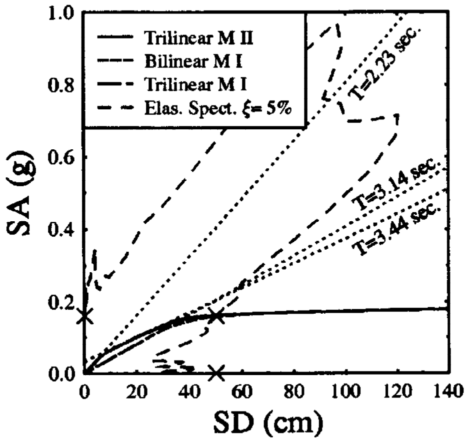

Fig. 7 Performance point determination.

Ayala and Ye, 1995 and 1996.

It is interesting to point out that the most important peaks on the T_i time history occur at the same times as the sudden jumps in the time history of D_{Park} shown in Fig. 3b. After those peaks, T_i remains almost constant indicating that the structure has stopped accumulating any further significant damage. The behaviour of D_{Park} is found somehow misleading as it presents sudden jumps every time the ductility in the structure reaches a value larger than preceding ones, remaining almost constant between consecutive jumps with no apparent contribution of the energy dissipated through hysteresis . This fact has been referred as an inappropriate weight given to this effect in the definition of the index, i.e. β factor (Ye, 1996).

Fig. 3c shows the evolution of the different components of energy in the structure, i.e. Input (E_i), Kinetic (E_k), Damping (E_D), and Strain+Hysteretic (E_{sh}) energies, Uang and Bertero, 1988. All energies are normalized to the maximum input energy $E_{i_{max}}$. The similarity of the time histories of D_{Park} and normalized E_{sh} is evident. D_{Park}, however, is almost constant immediately after T_{max} while there are still some changes in T_i and in (E_{sh}) as shown in Figs. 3a and 3b.

Fig. 4 shows the variation of six different definitions of period for increasing base shear intensity: (1) the original CSM period, T_{csm}, (2) the eigen period, T_{eig}, two definitions of the RMS period, (3) $0.65T_{csm}$ and (4) $T_{csm0.65D_{max}}$, (5) the "secant" period $T_{sec} = 2\pi\sqrt{\frac{m}{K_{sec}}}$ and (6) the "tangent" period $T_{tan} = 2\pi\sqrt{\frac{m}{K_{tan}}}$, where m is the total mass of the structure and K_{sec} and K_{tan} the "equivalent" secant and tangent stiffnesses of the structure if idealized as a SDOF with the pushover curve as its backbone curve, $K_{sec} = \frac{V}{D_{max}}$ and $K_{tan} = \frac{\Delta V}{\Delta D_{max}}$. The coincidence of T_{sec} and T_{csm} shows that the CSM is equivalent to the secant averaging method The difference between T_{tan} and T_{eig} shows the importance of higher modes indicating that the structure cannot be idealized as a SDOF system which is the case of low rise buildings, where T_{tan} and T_{eig} proved to be very similar (Tayebi 1996).

Fig. 5 depicts the mode participation factors (MPF's) for different values of base shear. It shows that higher modes must be considered, especially the second and third. The changes in MPF's also indicate new damage incurred by the structure due to significant deterioration of stiffness. As higher modes contribute up to 15% of the total response they should be included in the seismic response evaluation of medium height to tall buildings. As this paper covers only the first part of this investigation, only first mode was considered, nevertheless, the results are promising.

Fig. 6 shows the static pushover curve of the structure for three different models, two of which include cracking (trilinear primary curve) and a simplified bilinear model where cracking is ignored. Interestingly, all models give very close elastic branches and exactly similar plastic branches. This is due to the fact that loading is only in one direction and the effects of unloading which make the difference between models are not present. Hence, for the and CSM only a simple bilinear model is needed which can be easily simulated by most structural engineers via a sequence of elastic analyses with a simple frame static elastic analysis program.

Fig. 7 shows the determination of the performance point according to the proposed method. The similarities of the models results are also reflected in the S_a-S_d curves. In the (S_a,S_d) space, straight lines from the origin represent constant period lines. For the trilinear models one should not be misled by the initial cracking period (period at the origin) as it is of no practical value. The period of interest is obtained by drawing a tangent to the second branch that comes before the plastic branch, which in similar cases to this one will not pass by the origin. This is clarified when bilinear models are used where there is no cracking part and where there are only two branches and the period of interest starts from the origin as it is shown for the Takeda model (I). All three models give approximately the same point of intersection with the elastic spectrum at S_a=0.24 g and all three models predict an S_{ay} of 0.152 g resulting in a C_y of 0.63. The computer program NONSPEC (Mahin and Lin, 1983) is used to generate the inelastic strength spectrum using the equivalent strength parameter $\eta=\frac{F_y}{mPGA}=\frac{C_y}{PGA(g)}$ which is in this case equal to 3.79. For this case η=3.79 produces a spectrum identical to the elastic spectrum which simplifies the task even more. The performance point, PP, (50 cm,0.16 g) gives the results shown in Table 1 (where CSM1 and CSM2 represent the results of trilinear and bilinear models respectively) and Figs. 1f, 2a, 2b and 2c which are very close to those predicted by the dynamic analyses and those inferred from observed earthquake damage. It may be noted that the hinge distribution is almost identical to that of Models II and III.

CONCLUSIONS

It is a general conclusion that the use of different models in dynamic response calculations may lead to significant differences in the prediction of lateral displacements. In this investigation, the results obtained from Models II and III are considered the most reliable reliable. Model II is recommended due to its relative simplicity.

For CSM calculations, all models gave similar results as unloading is not present. Thus, for applications of the CSM it is recommended to use only a simple model.

Regarding the performance evaluation, $\frac{E_{sh}}{E_{max}}$ is found to be a very promising global damage index to be used for seismic evaluation avoiding the uncertainties involved in the global D_{Park} definition. For the evaluated building, it can be concluded that seismic performance can be adequately predicted using either nonlinear dynamic analysis or the CSM with the suggested modifications. The CSM remains, however, the best option among existing methods of seismic performance evaluation requiring only simple static analyses with a bilinear model or a series of elastic analyses.

The results of this paper, regarding the CSM, are preliminary, as the contribution of higher modes was not considered. Further research is needed to include this and to improve the determination of the performance point. These last two topics are the subject of current research by the authors.

This paper shows the importance of the use of engineering judgement in processing results from nonlinear analyses. It is the authors' opinion that the issue of choosing the "best" model for nonlinear dynamic analysis is still a topic of further research.

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REFERENCES

Ayala, A. G. and X.G. Ye (1995). Analytical evaluation of the structural damage of reinforced concrete frames.*Proc. of the VII Canadian Conf. on Earthq. Engrg.*, Montreal, Canada, June, 389-396.

Ayala, A.G. and X.G. Ye (1996). Analytical evaluation of seismic intensities with a period-based damage index for RC buildings. to be published in the *Proc. of the I Int. Symp. Earthq. Resistant Engrg. Structures*, CMP Publications, Southampton, October.

Freeman S.A., J.P. Nicoletti and J.V. Tyrrell (1975). *Evaluation of existing buildings for seismic risk. A case study of Puget Sound Naval Shipyard, Bremerton, Washington.* Proc. of the US Nat. Conf. on Earthq. Engrg., EERI, Ann Arbor, MI, 113-122.

Kunnath S.K., R.E. Valles-Matox and A.M. Reinhorn (1996). Evaluation of seismic damageability of typical R/C buildings in midwest United States. *Proc. of the XI WCEE*, Elsevier, Amsterdam.

Li, K.N. and Otani S. (1993). Multi-spring model for 3-dimensional analysis of RC members. *Journal of Structural Engineering and Mechanics*, **1**, No.1, 17-30.

Li K.N. (1993). CANNY-C, A computer program for 3D non-linear dynamic analysis of building structures. Technical report 004, National University of Singapore.

Mahin S.A. and Lai J. (1983). Construction of inelastic response spectra for single-degree-of-freedom systems. Report No. UCB/EERC-83/17, University of California, Berkeley, CA.

Meli, R. and J.A. Avila (1988). Analysis of building response. *Earthq. Spectra*, **5**, No.1, 1-18.

NIST (1994). Standards of seismic safety for existing federally owned or leased buildings, U.S. Department of commerce, National Institute of Standards and Technology, NISTIR 5382, ICSSC RP-4.

Park, Y.J. and A.H.S. Ang (1985). Mechanistic seismic damage model for reinforced concrete. *Journal of structural engineering, ASCE*, **113**, No.ST8, 722-739.

Park, Y.J., A.M. Reinhorn and S.K. Kunnath (1987). IDARC: Inelastic damage analysis of reinforced concrete frame-shear wall structures. Technical Report NCEER-87-0008, National Center for Earthquake Engineering Research, Buffalo, NY.

Tayebi A.K. (1996). Development and evaluation of enelastic design approaches for earthquake resistant buildings. *PhD thesis in process*, Dept. Str. Engrg., Cornell University, Ithaca, NY.

Uang C.M. and V.V. Bertero (1988). Use of energy as a design criterion in earthquake-resistant design. Report No. UCB/EERC-88/18, University of California, Berkeley, CA.

Ye X.G. (1996). Evaluation of structural damage in RC buildings subjected to earthquake. *PhD thesis in process*, DEPFI, UNAM, Mexico (in Spanish).