Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VII)

12-2-11

ASSESSMENT OF ASEISMIC CAPACITY FOR STEEL MULTISTORY FRAME BY PSEUDO-ELASTIC APPROXIMATE METHOD

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SUMMARY

The presented method would be emphasized on that the multistory steel frame can be analyzed for nonlinearity basically by direct stiffness method even under a severe earthquake. The elasto-plastic model for steel member is adopted, and the elasto-plastic equivalent strain energy relationship is based to determine the plastic displacements of the members in plastic stage. The steel frame after first plastic hinge appeared will perform inelastically. Furthermore, subjected to the seismic loads which may amplified from a given earthquake, the frame can be analyzed nonlinearly for aseismic capacity and ductility and be associated with proposed pseudo-elastic approach.

INTRODUCTION

In designing a building against severe earthquakes, the large seismic energy input to the building shall be obsorbed and dissipated through large but controllable inelastic deformations of its frame. However, these deformations should be limited to values which would avoid inducing severe damage to either structural and nonstructural elements. Specially for highrise steel frames, which will perform more flexible than reinforced concrete structures, should be carefully controlled and assessed their aseismic capacity and ductility as well.

The steel members are assumed to perform in elasto-plastic behavior (1,2,3,4,5), and their plastic deformations can be figured out by means of energy equivalent model (2,3,4,5).

The inelastic performation of steel frame subjected to dynamic input had been investigated by response spectrum approach (4,5). Here, the nonlinear frame performation associated with time history earthquake input will be figured out by introducing the pseudo-elastic approximate method which presents the equivalent elastic analytical model through out the nonlinear analysis to the steel frame under severe earthquakes.

BEHAVIOR OF STEEL MEMBERS

Steel beams are usually used in rolled or built up I-shape, while induced moment and shear are the major forces on a beam section, therefore axial force and torsion will be ignored in this paper. The elasto-plastic behavior is assumed as dealing with the relationship between moments and curvatures for steel beams and girders, also the energy conservation is assumed to be held when the plastic deformations are transfered to the equivalent elastic deformations as shown in Figure 1, (2,3,4,5).

Steel columns are considered in H-shape or box type, while moment, shear and axial force are the major forces on a column section, therefore buckling effect shall not be ignored. The values of plastic moments for a column section will be reduced as the axial force is increased(6), and one may take three dimensional yield surface into consideration(7).

LINEAR ANALYSIS FOR STEEL FRAME

In the deterministic analysis of earthquake response, it is usually assumed that the structure is a linear system which the stiffness of the frame does not change during the earthquake, and the frame may be analyzed under the associated assumptions those are: 1. Floors are rigid in-plan, 2. Axial deformations of beams may be ignored, 3. Torsional effects of members may be neglected, 4. Steel members of frame are performing in ideal elasto-plastic behavior, and 5. Seismic equivalent loads are acting on the levels of the corresponding floors, and are spread through floors by means of diaphram shear transfer.

The steps of analysis for the frame are followings:

1. To establish stiffness matrices for members,
2. To assemble the stiffness matrix for whole frame,
3. To operate the direct stiffness matrix analysis such as

$$\begin{Bmatrix} \bar{R}_n \\ \bar{R}_l \end{Bmatrix} = \begin{bmatrix} K_{nn} & K_{nl} \\ K_{ln} & K_{ll} \end{bmatrix} \begin{Bmatrix} r_n \\ r_l \end{Bmatrix} \tag{1}$$

and to carry through the static condensation(8) by

$$K_l = K_{ll} - K_{ln} K_{nn}^{-1} K_{nl} \tag{2}$$

$$R_1 = \bar{R}_1 - K_{1n} K_{nn}^{-1} \bar{R}_n \tag{3}$$

$$R_1 = K_1 r_1 \tag{4}$$

where R being acting forces, K being stiffnesses and r being displacements.
4. To solve the lateral displacement vector r for frame by Gaussian elimination.
5. To find out the end forces of members on the frame.

The linear analysis for the frame expressed herein is adopt for ordinary building design according to typical seismic code requirements.

PSEUDO-ELASTIC APPROACH FOR NONLINEAR ANALYSIS

It can easily be demonstrated that an earthquake of only moderate intensity will cause significant overstress and structural damage and the structure stifness must undergo changes so as to perform nonlinearly. However, nonlinear analysis usually requires a large expenditure of computing effort to evaluate the behavior even rather simple structures since the stiffness matrix must be reformed and decomposed for each incremental process.

In order to obtain a reasonable measure of the nonlinear earthquake behavior of a structure with simple process similar to ductility factor method(9), the writer would borrow this idea to treat the members on steel frame. However, the displacement of elastic and elasto-plastic model for same strain energy in corresponding to the structures with short fundamental period(1)(5). Even the stiffness for overstressed member need not be reformed, one can carry out the plastic displacements transformed from pseudo-elastic displacements, Figure 1.

Moreover, the pseudo-elastic response for the frame subjected to an earthquake can be figured out easily since the stiffness of the frame need not be changed. Further more, the inelastic response for the frame after first plastic hinge appeared may be traced by applying the following approximations: 1. The strain energy conserved for pseudo-elastic performance of the frame beyong the inelastic stage Δw is shown to be proportional to the summation of the energy conserved for pseudo-elastic performance of all members beyong the plastic stage E_i, (10)(11), that is

$$\frac{\Delta W}{W} = \frac{\Sigma \Delta E_i}{\Sigma E_i} \tag{5}$$

where, W being workdone due to external loads or total strain energy conserved for the frame and being proportionally appeared as the triangular area oi Δ_i in Figure 2a and 2b, and E_i being strain energy of member i and ΔE_i being the pseudo-elastic strain energy conserved for the member i beyond its real plastic stage such as the area AB'C in Figure 1. 2. Total energy conserved for inelastic performance of the frame will be same amount of the total energy conserved for pseudo-elastic performance of the frame. The inelastic response can be traced piecewise continuously by using these two relations to determine the slope and end coordinate of the next response segment linearly.

ILLUSTRATIONS

<u>Example 1</u> There is a two-story in two-bay steel frame, same as used in Ref.(11,12), which is subjected to uniform distributed load 3.58 t/m and concentrated axial loads on column, as well as it is subjected to earthquakes those are amplified from the earthquake as shown in Figure 3. The monotonic inelastic responses, shown in Figure 4, are presented in comparison with static inelastic step by step analysis. The maximum lateral load capacity and drift determined from pseudo-elastic approach are very close to the results obtained from step by step static nonlinear analysis.

<u>Example 2</u> An office building with 19-story steel frame, same as used in Ref.(5, 10,11), is subjected the earthquakes which are amplified from the earthquake as shown in Figure 3. The monotonic inelastic responses determined from different approaches are presented in comparison with this method, as shown in Figure 5. The maximum lateral load (or seismic) capacity and drift determined from the proposed pseudo-elastic approach are acceptable.

CONCLUSION

The pseudo-elastic method presented herein is proposed as an approximate approach for evaluating the aseismic capacity of steel framed structures with acceptable accuracy after this and associated studies (10)(11), the following conclusion remarks can be drawn:

1. During analysis going, neither member stiffness nor structural stiffness should be reformed or recomposed, the direct stiffness matrix method can be operated with same stiffness matrix through out whole analysis, so that good accuracy, time saving and easy operation could be reached.

2. The seismic load may be introduced in lateral static form such as in seismic code requirements, or in dynamic expressions such as in time history earthquake accelerogram or in earthquake response spectrum.

3. The maximum lateral load capacity of the structure can be evaluated, as well as the drift of the frame and the ductility coefficients of members can be obtained. The maximum amplifier (factor of safety) to a given earthquake which

the structure can stand safely, also can be obtained by this presneted mehtod.

ACKNOWLEDGEMENTS

The financial supports coming from the National Science Council of the Republic of China to this study and associated investigations would be gratefully acknowledged. The author wish to express his special thanks to the contributions of Mr. F. S. Yang and Mr. Y. F. Liao for their calculation and computer work efforts to this study.

REFERENCES

1. Veletsos, A. S. and Newmark, N. W., "Effect of Inelastic Behavior on the Response of Simple Systems to Earthquake Motions," Proc. 2nd World Conf. on Earthquake Engineering, Vol. 2, Tokyo, (1960).
2. Veletsos, A. S., "Maximum Deformations of Certain Nonlinear System," Proceedings of 4th World Conference on Earthquake Engineering, Vol. 2, Chile, (1969).
3. Pique, J. R. and Roesset, J. M., "On the Use of Simple Models in Nonlinear Dynamic Analysis," MIT R76-43, (1976).
4. Newmark, N. M. and Hall, W. J., "An Evaluation of A Response Spectrum Approach to Seismic Design of Building," ATC-2, Applied Technology Council, San Francisco, California, (Sept., 1974).
5. Chern, C. C. and Lin, Y. C., "Nonlinear Dynamic Analysis for Aseismic Capacity of Multistory Steel Frame," Earthquake Engineering Research Center, CEER R75-03, Taipei, (August, 1986).
6. American Institute of Steel Construction(AISC), Manual of Steel Construction, 8th Edition, (1980).
7. Porter, F. and Powell, G., "Static and Dynamic Analysis of Inelastic Frame Structures," EERC R71-3, University of California at Berkeley, (June, 1971)
8. Wilson, E. L. and Dovey, H. H., "Three Dimensional Analysis of Building System," Report EERC 72-8, (Dec., 1972).
9. Clough, R. W. and Penzien, J., Dynamic of Structures, McGraw-Hill, (1975), pp602.
10. Yang, F. S., "Nonlinear Time History Dynamic Analysis for Highrise Steel Frame," M.S. thesis directed by Prof. C. C. Chern, Dept. of Civil Engineering, National Taiwan University, (June, 1987).
11. Liao, Y. F., "Nonlinear Dynamic Analysis for Highrise Steel Frame with Nonrigid Joints," M.S. thesis directed by Prof. C. C. Chern, Dept. of Civil Engineering, National Taiwan University, (June, 1987).
12. Alvarev, R. J. and Birnstiel, C., "Inelastic Analysis of Multistory Multibay Frame," ASCE Vol. 95, No. ST 11, (Nov., 1969).

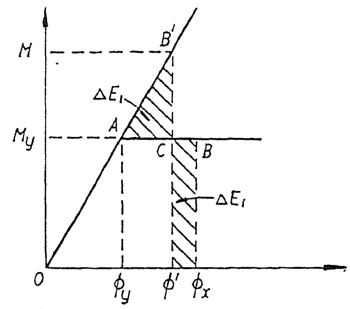

Fig. 1 Pseudo-elastic and Elasto-plastic Behavior Model for Steel Beam

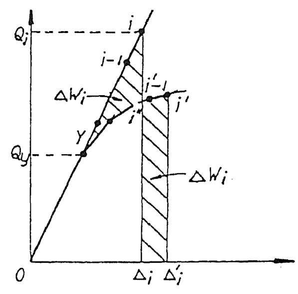

Fig. 2 Pseudo-elastic and Elasto-nonlinear Behavior Model for Steel Frame

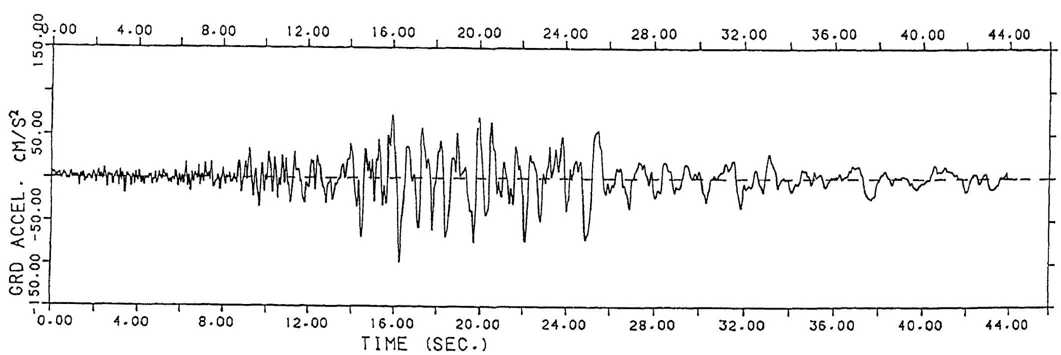

Fig. 3 Taipei Earthquake Accelerogram, November 15, 1986 (NS Component)

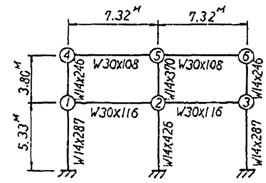

Fig. 4a Frame Size

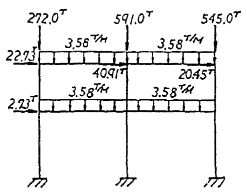

Fig. 4b Frame Loadings

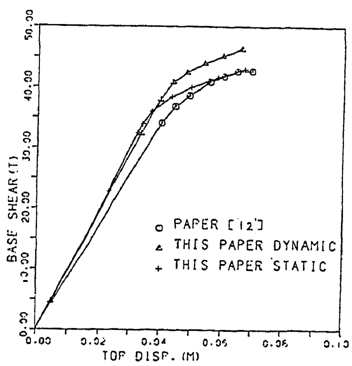

Fig. 4c Shear of Column 1 - 4 vs. Top Displacement

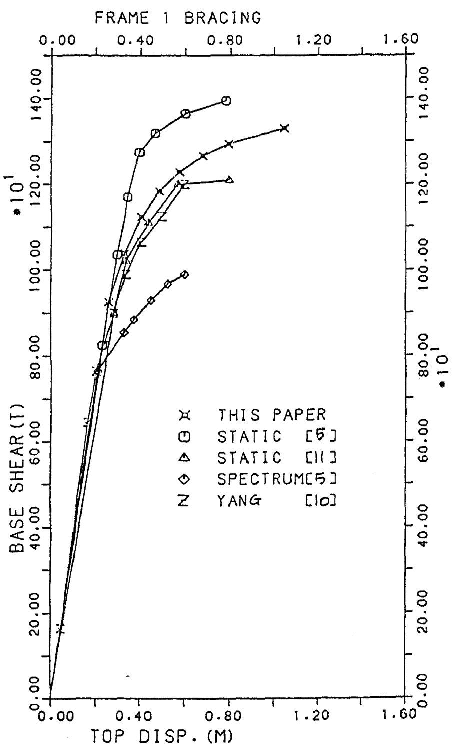

Fig. 5 Base Shear of Frame vs. Drift