ASEISMIC ASSESSMENT FOR LATERAL STABILITY OF MULTISTORY STEEL FRAME BY PSEUDO-ELASTIC METHOD

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SUMMARY

This paper presents a pseudo-elastic method to assess the maximum seismic resisting capacity for the lateral stability of a multistory steel frame. The steel frame is analyzed basically by direct stiffness operation, however the frame can be dealt with the inelastic behavior of members on which ends the plastic hinges may occur and the possible member buckling models are considered. Based on that the energy conservation concept, the total strain energy conserved in a member on the frame at pseudo-elastic stage is equal to the energy conserved in same member at inelastic stage. The elasto-plastic models for the steel members are established firstly, and several models of sub-structural stability caused by plastic hinges forming are studied. The steel frame is also dynamically analyzed, thus the pseudo-elastic response of the frame can be transformed to the inelastic response simply by presented pseudo-elastic method while the frame is subjected to lateral loads combined with vertical loading. Finally the maximum seismic resistant capacity and drift of the frame can be determined in acceptable accuracy by presented pseudo-elastic method associated with buckling modes of the steel frame.

INTRODUCTION

In considering a high-rise steel frame against severe earthquake, the seismic energy input to the frame will be absorbed and dissipated through inelastic deformations of the members of frame. However, these deformations should be limited to avoid severe damage to structural elements. The members on steel frame are assumed to perform in elasto-plastic behaviors in accordance with their acting forces. The drift in inelastic stage of frame can be traced by means of energy equivalent relationship from imagined pseudo-elastic deformations. Basing on the total strain energy conserved in all members on the frame in inelastic stage is equal to the total strain energy conserved in all members on the frame in imagined pseudo-elastic stage of the frame. The inelastic response of a steel frame subjected to dynamic time history earthquake had been investigated and associated with pseudo-elastic method of equal energy approach or equal drift approach herein.

PSEUDO-ELASTIC ANALYSIS FOR MEMBERS ON FRAME

The moment-curvature response of a steel beam which is subjected vertical load and earthquake load, is able to be expressed in elasto-plastic bilinear curve as shown in Figure 1; while the bilinear curves for moment-curvature relationship of column are expressed in Figure 2. The basic transformation assumption is that the area

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of shaded triangle is equal to the shaded rectangular area, this means that the equivalent energy conserved, the area which is integrated beneath the corresponding moment-slope curve; for small deflection members, in the member should keep constant for assuming no energy dissipated.

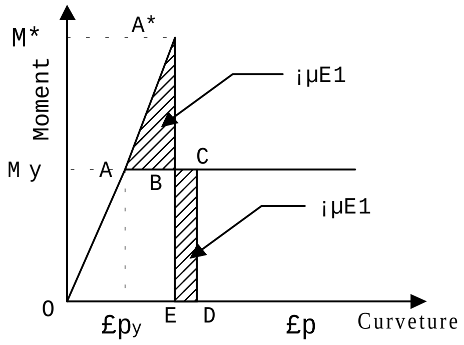

Figure 1 Beam Moment-Curveture Curve

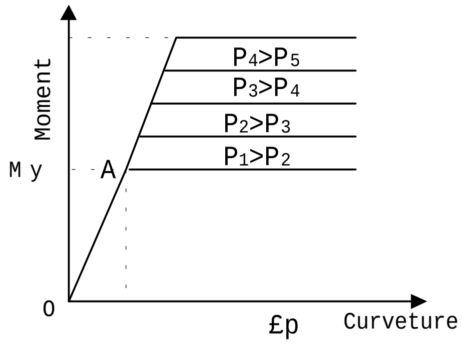

Figure 2 Column Moment-Curveture Curve

The total strain energy restored in the member can be expressed as:

$$\Pi = \sum C_m \int M d\phi + \sum C_p \int P d\delta + \sum C_s \int S d\nu + \sum C_t \int M_t d\varphi + ... \qquad (1)$$

where M, P, S, and M_t are denoted for moment, axial force, shear force, and twisting moment respectively; C_m, C_p, C_s, and C_t are denoted for coefficients of the strain energy due to moment , axial force, shear force, and twisting moment respectively. The pseudo point A^* is located in accordance with that the strain energy covered in area of triangle AA^*B is equal to the area of rectangular BCDE, this makes that the area of triangle OA^*E equals to the area of OACD.

LINEAR DYNAMIC ANALYSIS FOR RIGID FRAME

In analysis of earthquake response, the rigid multistory steel frame is assumed to be a linear system which stiffness does not change during the pseudo-elastic analysis. Therefore the frame response can be carried out by linear elastic analysis under the basic essential assumptions as followings: (1) Floors are in-plan rigid. (2) Axial deformation of beam may be ignored. (3) Torsion effect of member may be neglected. (4) All cross sections of steel member are uniform. (5) Seismic equivalent loads are acting at the levels of the corresponding floors of building, also the floor masses are lumped at the same levels, and seismic forces are spread through floors by means of diaphragm shear transfer. Equation of motion for a linear rigid high-rise steel frame is given as usual as:

$$M\ddot{X} + C\dot{X} + KX = -M\ddot{X}_g \qquad (2)$$

in which, coefficient matrices *M,C,* and *K* are mass matrix, damping matrix and stiffness matrix; $\ddot{X}_g$ denotes the earthquake acceleration input, *X* is the relative floor displacement vector. This equation can be solved by mode superposition method and time history dynamic analysis associated with numerical integration operations. However, to establish the stiffness matrices for members and to assemble the stiffness matrix for whole frame shall be worked out by the application of matrix static condensation, used by [Wilson and Dovey]. The responses of the frame and member forces can be obtained by combining the forces induced from gravity load and the largest member forces from dynamic load through out the duration of the earthquake time history input.

PSEUDO-ELASTIC EQUAL ENERGY APPROACH FOR NONLINEARITY

Considering on the seismic loading frame for its imaginary pseudo-response which will be determined by applying the constant structural stiffness matrix to the system in equation (2). That pseudo-stiffness does not change even after the plastic hinges appeared on the true frame, and the pseudo-relationship of base shear *V* and drift Δ is shown as linearly as OAB in Figure 4, while the true frame will perform in inelastic behavior as OACF. In Figure 4, once the first plastic hinge occurs on the frame, thus point A is located at the division tip for elastic and inelastic response curves of frame. The process to trace the inelastic response curve ACF from elastic response straight line AB, also it is called as the pseudo-elastic response, had been expressed by [Chern and Liao 1989]. Mainly the equal energy approach is followed that

$$W = area\ OABCD = area\ OACFE \tag{3}$$

$$\Delta W = area\ ABC = area\ CFED \tag{4}$$

thus

$$\frac{\Delta W}{W} = R = \frac{area\ ABC}{area\ OABCD} = \frac{area\ CFED}{area\ OACFE} = \frac{\sum \Delta E_i}{\sum E_i} \tag{5}$$

where *W* and ΔW denote work down due to external forces and their increment work down, E_i and ΔE_i denote member strength energy and its increment for member ***i***. The relation curve for elasto-plastic behavior of steel members are followed from [Blume, Newmark and Corning, 1961], shown in Figure 1, in dealing with moment and curvature.

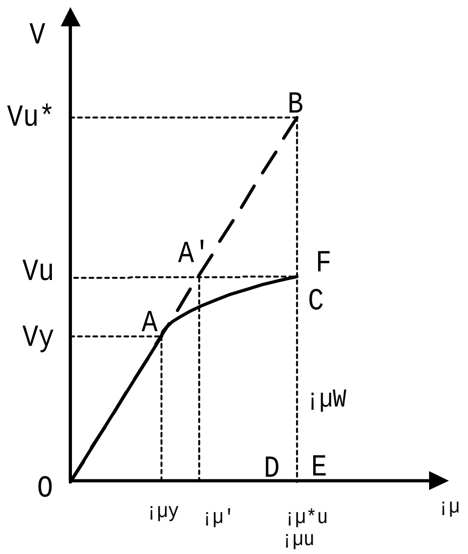

Figure 3 Pseudo-elastic equal drift approach

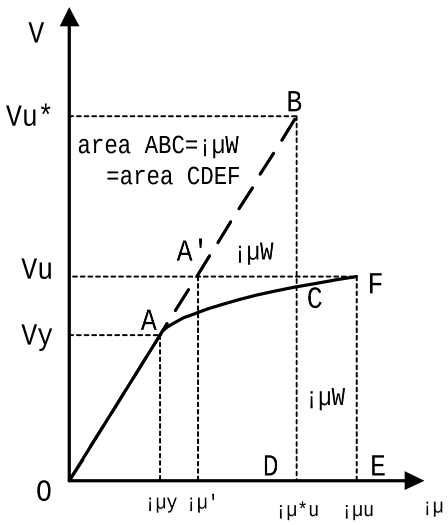

Figure 4 Pseudo-elastic equal energy approach

In this study the failure of frame is defined as one of followings occurred: (1) Mechanism failure for the structure, or plastic hinges appear on both ends of all columns on same floor. (2) The total inclined angle, the ratio of maximum drift of frame top to total frame height, is larger than 0.015, local code upper limit for it is 0.005. (3) The inclined angle for any one of stories is larger than 0.015, local code upper limit for it is 0.005. (4). The acting axial load or shear force on any column is greater than the axial capacity or shear capacity of that column respectively. (5) Column buckling is happened, while the buckling modes are described as followings section, or axial force of column is equal or larger than buckling axial force P_{cr}. (6) Column axial force is larger than allowable axial capacity P_a.

BUCKLING MODES FOR FRAME COLUMN

There eight basic buckling modes those are shown in Figure 5, are carefully considered for

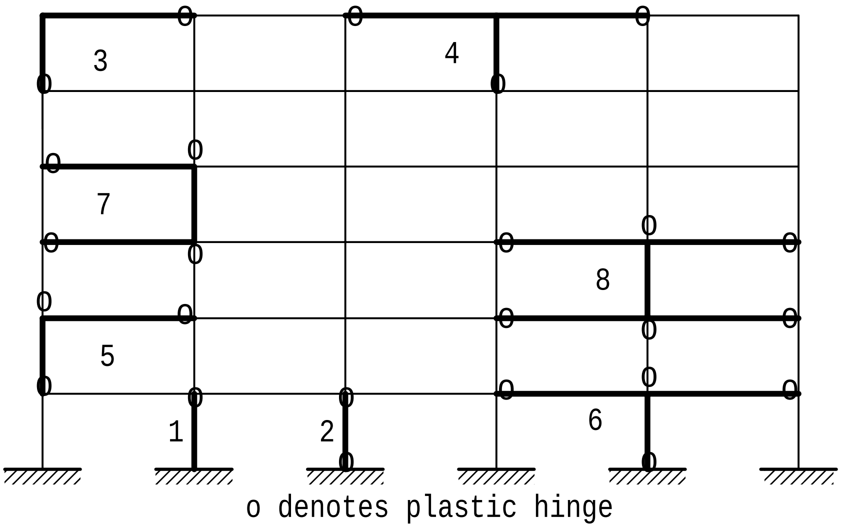

Figure 5 Eight buckling modes considered

including the side-sway of columns and derived the characteristic equations for end columns and for internal columns respectively in this study. The characteristic equation can be formed as a function of eigen-value K as $F(KL)=0$. For example, one deals the first and the 2nd buckling modes. There is one plastic hinge to occur at top of column for the first bucking mode; and there is one plastic hinge to occur at top and bottom of the column respectively, see Figure 5. The side-sway of column is δ , and the characteristic equation in terms of KL is derived as following as:

$$\frac{M_p L}{EI(KL)^2}\left[\cos(KL)-1\right]+\frac{\delta}{L}\left[\frac{\sin(KL)}{KL}-\cos(KL)\right]=0 \tag{5}$$

$$\frac{M_p L}{EI(KL)^2}\left[KL\sin(KL/2)+KL\cot(KL)\cos(KL/2)+0.5KL\csc(KL/2)-2\right]=\frac{\delta}{L} \tag{6}$$

in these equations, E is elastic modulus of member; I is moment of inertia of the cross section of member; M_p is plastic moment of the column; K is defined as $K^2 = P_{cr}/EI$; L is height of column; and the maximum inclined angle of column is defined as a code limit case of frame as:

$$\delta/L = 0.005 \tag{7}$$

The minimum value of (KL) can be solved numerically said β from equation (5) for the first buckling mode and equation (6) for the second buckling mode respectively. One can easily obtain the critical load as $P_{cr} = \beta^2 EIL^{-2}$. Similarly the critical load with respect to other columns can be figured out.

EVALUATION FOR THE ASEISMIC CAPACITY OF THE FRAME

The pseudo-elastic equal drift approach, its response relevant to base shear and drift as shown in Figure 3, is suitable for the frames of natural period in the range of 1.5 to 2.0 second,[Blume, Newmark and Coring 1961]. On the other hand, the pseudo-elastic equal energy approach, as shown in Figure 4, is suitable for the frames of natural period out of the range of 1.5 to 2 second. No matter basing on which approach, a lot of multistory steel frames should be analyzed linearly then nonlinearly.

When dynamic analysis is concerned, the local earthquake ground acceleration records as well as El Centro 1940's record are input for this study. Once the first plastic hinge is occurred, thus the yielding base shear V_y for the frame can be found, see Figures 3 & 4. Then the traditional approach will be the stiffness matrix analysis for the frame, and the stiffness of the members should be changed step by step for every one plastic hinge increased as DRAIN 2D+ doing. However, in this paper the author would try to apply the pseudo-elastic approach for not changing the member stiffness and frame stiffness. Basing on the equal energy approach, the total strain energy conserved in the frame should be the same prior to and after the plastic hinge occurred. Thus imagined pseudo-elastic drift Δ_u^* corresponding to lateral pseudo-load V_u^* can be traced to real inelastic drift Δ_u and real lateral load V_u as shown in Figure 4. On the application of equal drift approach, the real inelastic response should be worked out by DRAIN 2D+, as shown in Figure 3; thus one should find out Δ_u and V_u , then locate V_u^* .

The pseudo static pushing over analysis is also applicable by presented pseudo-elastic approach. For this the earthquake forces acting on frame at the levels of floor are determined in accordance with the local seismic code as followings:

$$F_i = \frac{(V - F_t)w_i h_i}{\sum_{j=1}^{j=N} w_j h_j} \tag{8}$$

where V is base shear force for the frame, w_i and h_i are the weight and height of the i-th floor respectively, F_t denotes the additional force at top of building, and F_i denotes the earthquake pseudo static force for i-th floor. N denotes the total story number of the frame. One may increase the value of V to obtain the static response such as shown in Figures 3 & 4.

Finally the maximum base shear V_u for the frame, that is the maximum lateral resistant capacity of the frame in considering the failure modes of the frame including member buckling, could be figured out. And the frame drift reaches its maximum Δ_u, also the lateral pseudo-load V_u^* can be obtained. These loads and drifts are related to the lateral resistant capacity factor α as:

$$\alpha = \frac{V_u}{V_y} = \frac{\Delta^{'}}{\Delta_y} = \frac{\mu\Delta^{'}}{\Delta_u} \tag{10}$$

in which $\mu = \Delta_u / \Delta_y$ denotes the ductility capacity of frame.

For individual case of frame inelastic analysis, one can finally find out yielding lateral base shear V_y and corresponding drift Δ_y, the maximum lateral base shear V_u or the ultimate lateral capacity of frame, and the corresponding ultimate drift of the frame Δ_u, thereafter the ductility capacity $\mu = \Delta_u / \Delta_y$ can be determined.

It is possible for large number of steel frames to be assessed by the presented pseudo-elastic approach for this kind of inelastic analysis. From equation (10), once yielding lateral base shear V_y and corresponding drift Δ_y are found as the first plastic hinge appeared on the frame. The ductility capacity of frame μ and frame lateral resistant capacity factor α are figured out from prior statistic analysis for large number of high-rise steel frames, the approximate aseismic capacity V_u and the corresponding drift Δ_u for the frame can be obtained very quickly. The hard work for this approach is to carry out the nonlinear analysis for real high-rise frames to figure out the empirical formula for capacity factor α and ductility capacity μ of steel frame previously for local frames.

ILLUSTRATIONS FOR ASEISMIC ASSESSMENT

In this study, there are 3 sets of high-rise frames in 15-story, 20-story and 25-story in 5-column with 4-beam. There are 2 types for each set of frame, one type for ductile space moment resisting frame and another type for ductile frame associated with lateral bracing members. The beam sections are H350×350×10×16mm to H350×350×19×19mm. The column sections are H800×300×14×22 mm to H800×300×22×40mm the spans are 900cm, and story heights are 500cm to 1000cm, these sizes are commonly used in Taiwan. The thickness of R.C. floor slabs is assumed in 12cm. The live load on each floor is given in 300 kgf/m^2, and the accelerogram from EL Centro earthquake in 1940 was adopted for time history nonlinear dynamic analysis, associated the software DRAIN 2D+ version 1.1, which was revised by [Tsai and Li 1994]. The preliminary empirical values [Lee and

Chern, 1996] for frame capacity factor α =1.768 and frame ductility capacity μ = 5.803. More accuracy values could be worked out by applying the presented method for more inelastic analyses of more high-rise steel frames. The control buckling modes in lower stories are to be mode 2 and 6; in middle stories to be mode 5, 6, 7, and 8; in top story to be mode 3 and 4. [Wang and Chern 1999]

CONCLUSION

After this study the following conclusion remarks could be drawn: (1) The presented pseudo-elastic method could be applied to assess the aseismic capacity, or ultimate lateral shear capacity, for the steel frames. (2) The preliminary value for frame lateral resistant capacity factor α is 1.768 and the value for frame ductility capacity μ is 5.803. (3) For more precise assessment, the further studies and analyses covering a large number of the existed steel frames should be carried out, in order to get more accurate values of lateral resistant capacity factor α and the ductility ratio μ for steel frames. (4) The proposed substructure buckling in eight modes herein this study can be extended. (5) The presented pseudo-elastic method can be applied to assess on the ultimate lateral capacity of reinforced concrete frames in good condition, however none of the substructure buckling modes may control the frame lateral capacity.

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