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Nonlinear inelastic response history analysis of steel frame structures using plastic-zone method

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ABSTRACT

A beam–column element formulation and solution procedure for nonlinear inelastic analysis of planar steel frame structures under dynamic loadings is presented. The spread of plasticity is considered by tracing the uniaxial stress–strain relationship of each fiber on the cross section of sub-elements. An elastic perfectly-plastic material model with linear strain hardening is employed for deriving a nonlinear elemental stiffness matrix, which directly takes into account geometric nonlinearity and gradual yielding. A solution procedure based on the combination of the Hilber–Hughes–Taylor method and the Newton–Raphson method is proposed for solving nonlinear equations of motion. The nonlinear inelastic time-history responses predicted by the proposed program compare well with those given by the commercial finite element package known as ABAQUS. Shaking table tests of a two-story steel frame were carried out with an aim to clarify the inelastic behavior of the frame subjected to earthquakes generated by the proposed program. A more practical analysis method for seismic design can be developed by comparing it with the presented frames for verification.

1. Introduction

Steel frame structures are usually designed for industrial and commercial buildings in earthquake-prone areas because of their ductility. In order to predict the exact behavior of steel frames, especially in severe loading conditions, advanced analysis methods are employed. An advanced analysis must simultaneously include all key factors of steel frames such as geometric nonlinearities (P—large delta and P—small delta effects), inelastic material, semi-rigid connections, imperfection geometry, residual stress, etc. There are two methods for advanced analysis of steel frame structures: (i) the "plastic-nodes" or "plastic-hinges" approach (concentrated plasticity), (ii) the plastic-zone approach (distributed plasticity). The basic difference between these two approaches lies in the manner that yielding within members is modeled. In the former case, assuming that plastic hinges form at two ends or monitored points of members while the yielding is distributed throughout the whole length and depth of members with the plastic-zone approach, the plastic-zone approach is a more exact model to predict the behavior of frame structures. However, it consumes many computer sources and computational time. Nowadays, with the ongoing development of computer sciences, the personal computer has developed a great capacity for restoring data and performing computations. Therefore, computation time is not a great obstacle for the nonlinear finite element analysis, so the plastic-zone method is more and more feasible for practical office design.

Research is generally lacking in regard to second-order distributed plasticity analysis of steel frame structures under earthquake excitations. A fair amount of studies concentrate on obtaining the optimal accurate finite elements so that static problems are employed. There are three finite element types that can be developed for capturing the gradual yielding of steel frames. The accuracy and complexity levels are respectively listed as follows: (i) solid elements, (ii) shell elements and (iii) beam–column elements. In order to trace local buckling and warping effects in steel profiles, there are a few ABAQUS simulation studies using shell elements [1–3], recently, Rigobello et al. [4] present a solid-like finite element. Although finite element solutions using shell-type [1–3] and solid-type [4] elements are more accurate than those of beam-type elements, it is complicated and expensive in terms of computer sources and computational time for performing multi-story frames. Therefore, the spread-of-plasticity analysis using beam-type elements is more favorable and common [5–15]. Foley and Vinnakota [7–9] developed a nonlinear finite

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element program for the second-order distributed plasticity analysis of multi-storey planar steel frames under static loadings. In order to improve computational performance, Foley [11] proposed parallel processing and vectorization, in which a main structure is separated into several sub-structures for reducing unknowns of system equations. Alemdar and White [12] presented different procedures based on displacement, flexibility, and mixed beam–column formulations using a total Lagrangian corotational approach for the distributed plasticity analysis of frame structures.

In recent years, Chiorean [13] derived a complicated beam–column method for the second-order inelastic analysis of space steel frames with semi-rigid connections. The advantage of this study is that it is able to trace the spread of plasticity along the member length by using only one beam–column element per frame member in analytical modeling. However, it seems that the shape parameters a and n of the Ramberg–Osgood model and α and p of the proposed modified Albermani model for the force–strain relationship of the cross-section, which consider the effect of the gradual yielding behavior of members, need to be investigated more adequately. Thai and Kim [14,15] presented a second-order spread-of-plasticity analysis method for steel frames using only one beam–column element per frame member by employing stability functions, in which the elemental axial stiffness and bending stiffness are sensitive to the number of integration points due to the important influence of weight factors (i.e. if plastic hinges only form at two ends of the member, using two integration points gives a smaller stiffness than using ten integration points for a member due to the weight factors being distributed more uniformly for ten monitoring points). Therefore, choosing the number of integration points acts on the final response of structures. Also in this way, the effect of the shift of the elastic core during the yielding processing is difficult to consider exactly. Such proposed formulations can underestimate the load-carrying capacity and performance of frame structures. To obtain a high accuracy, it is necessary that members should be completely divided into a lot of sub-elements along their length. After that, stiffness matrices of sub-elements are assembled to form a stiffness matrix at the member level.

In the context of this study, the distributed plasticity method developed by Foley and Vinnakota [7–9,11] for nonlinear static analysis is upgraded in terms of its applications for nonlinear dynamic analysis. A numerical procedure is proposed to perform the second-order inelastic time-history analysis of planar steel frames subjected to dynamic loadings and earthquake excitations. Frame members are divided into a lot of sub-elements along the member length and each sub-element cross-section is divided into a lot of fibers to trace the progress of the spread of plasticity and assign initial residual stress distributions for steel sections. An elastic perfectly plastic material model with strain hardening is used. The tangent stiffness matrix of the nonlinear beam–column element directly taken into account for the effects of geometric nonlinearity and gradual yielding of the material is derived using the Rayleigh–Ritz method and the principle of stationary potential energy. The moving of the strain-hardening and elastic neutral axis, which, due to yielding of some fibers in the cross-section, is directly considered in the tangent stiffness matrix during the analysis process. A computer program using the Hilber–Hughes–Taylor (HHT) method [16] is developed for solving the governing differential equations of equilibrium because its numerical dissipation is necessary to obtain convergent solutions with regard to some complicated nonlinear problems. Shaking table tests for a one-bay two-story steel frame carried out the aim to clarify the inelastic behavior of steel frames subjected to earthquake excitations and its results are used to verify the validity of the second-order inelastic dynamic analysis techniques of the proposed program. The results of nonlinear inelastic responses are also compared with those of ABAQUS to verify the accuracy of the proposed numerical procedure.

2. Nonlinear beam–column element formulation

In order to capture the distributed plasticity, a beam–column member is divided into n elements along the member length as illustrated in Fig. 1a, each element is divided into small fibers within its cross section as illustrated in Fig. 1b, and each fiber is represented by its material properties, geometric characteristic, area A_i, and its coordinate location corresponding to its centroid $\left(y_j, z_j\right)$. By this way, residual stress is directly considered in assigning an initial stress value for each fiber. The P-delta effects are included by the use of several sub-elements per member through continuous updating of the element stiffness matrix and nodal coordinates.

In the formulation of the nonlinear finite element, the following assumptions are made: all elements are initially straight and prismatic; plane sections remain plane after deformation; lateral–torsional buckling is prevented; local buckling of cross-sections is not considered; residual stress is uniformly distributed along the member length; the shear effect on the yielding of materials is ignored; the Bauchinger effect is neglected since the isotropic hardening model is used for steel material. The Rayleigh–Ritz method and the principle of minimum potential energy are used to derive the stiffness matrix of a nonlinear beam–column element under loads as illustrated in Fig. 2. An elastic plastic stress–strain relationship with linearly strain hardening presented by Toma and Chen [17] is adopted as shown in Fig. 3. Strain hardening starts at the strain of $\varepsilon_{sh} = 10\varepsilon_y$, and its modulus E_{sh} is assumed to be equal to 2% of the elastic modulus E. The total internal strain energy of a nonlinear elastic–plastic beam–column element considering the effect of strain hardening can be given as follows:

$$U = \int_{V_e}\int_0^{\varepsilon}\sigma d\varepsilon dV_e + \int_{V_p}\left(\int_0^{\varepsilon}\sigma_y d\varepsilon - \int_0^{\varepsilon_y}\sigma d\varepsilon\right)dV_p + \int_{V_{sh}}\left\{\int_0^{\varepsilon}\sigma_y d\varepsilon - \int_0^{\varepsilon_y}\sigma d\varepsilon + \int_{\varepsilon_{sh}}^{\varepsilon}E_{sh}(\varepsilon-\varepsilon_{sh})d\varepsilon\right\}dV_{sh} \tag{1}$$

since

$\sigma = E\varepsilon$	for elastic fibers
$\sigma = E\varepsilon_y = \sigma_y$	for yielding fibers
$\sigma = E\varepsilon_y + E_{sh}(\varepsilon-\varepsilon_{sh}) = \sigma_{sh}$	for hardening fibers

where ε is the normal strain at any fiber within a cross section, σ is the normal stress at any fiber within a cross section, V is the volume of fibers corresponding to their states within a cross section of an element, and subscripts $e, p(y), sh$ stand for elastic, plastic, and strain hardening states of fiber elements. Fig. 1b illustrates a cross-section discretization with fiber states, in which d_{CGe} and d_{CGsh} are the shift of the center of the initial neutral axis and the distance from the initial neutral axis to the strain-hardening neutral axis created by fibers in the strain-hardening regime, respectively.

Using Hook's law for elastic fibers and replacing the integrations over the volume of the element in Eq. (1) by integrating along the length and throughout the cross section of the element, Eq. (1) is expressed as

$$U = \frac{E}{2}\int_L\int_{A_e}\varepsilon^2 dA_e dx + \sigma_y\int_L\int_{A_p}\varepsilon dA_p dx - \frac{1}{2}\sigma_y\varepsilon_y\int_L\int_{A_p}dA_p dx + \frac{E_{sh}}{2}\int_L\int_{A_{sh}}\varepsilon^2 dA_{sh}dx + (\sigma_y - E_{sh}\varepsilon_{sh})\int_L\int_{A_{sh}}\varepsilon dA_{sh}dx + \frac{1}{2}(E_{sh}\varepsilon_{sh}^2 - \sigma_y\varepsilon_y)\int_L\int_{A_{sh}}dA_{sh}dx \tag{2}$$