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Second-order spread-of-plasticity approach for nonlinear time-history analysis of space semi-rigid steel frames

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ABSTRACT

In this paper, a nonlinear inelastic time-history analysis procedure for space semi-rigid steel frames subjected to dynamic loadings is presented. Geometric nonlinearities are taken into account by using stability functions and the geometric stiffness matrix. The spread of plasticity over the cross section and along the member length is captured by monitoring the uniaxial stress–strain relation of each fiber on selected sections. The nonlinear semi-rigid beam-to-column connection is simulated by a multi-spring space element. Three main sources of inelastic hysteretic, nonlinear connecting, and structural viscous damping are considered. The differential equation of motion is solved by the Hilber–Hughes–Taylor method combined with the Newton–Raphson method for the equilibrium iterative procedure. Using only one element per member in the structure modeling, the nonlinear time-history responses which are predicted by the proposed program compare well with those given by commercial finite element packages and other available results. Numerical examples are presented to verify the accuracy and efficiency of the proposed method.

1. Introduction

In conventional analysis and design, beam-to-column connections are usually assumed to be fully rigid or ideally pinned joints. The real behavior of beam-to-column connections is a nonlinear curve which depends on the configuration of connections. Such connections are called semi-rigid connections which play a role in transfering a part of moments from elements to other ones while the rest is resisted by themselves. The experimental studies showed that semi-rigid steel frames feature ductile and stable hysteretic behavior when the connections are designed appropriately [1–4]. The energy is dissipated through hysteretic loops of semi-rigid connections, which are one of the important damping sources of structures.

There are two common nonlinear analytical approaches for space steel framed structures: the plastic hinge methods (concentrated plasticity) [5–12] and the plastic zone methods (distributed plasticity) [13–17]. The plastic hinge methods [7–12] using stability functions obtained from the closed-form solution of the beam–column element subjected to end forces can accurately capture the second-order effects using only one or two elements per member. Material nonlinearity is considered by the lumped plastic hinges at the two ends of the member. The effects of distributed plasticity and residual stress are indirectly taken into account by using the reduced tangent modulus approach. However, the plastic hinge methods are limited due to their incapability of capturing the more complex member behaviors that involve torsional–flexural buckling, local buckling, and severe yielding under the combined action of compression and bi-axial bending, which may significantly reduce the load-carrying capacity of a structure [14]. Furthermore, the hinge methods have shown to over-estimate the limit strength when structural behavior is dominated by the instability of a few members [18]. Also, it may inadequately give information as to what is happening inside the member because the member is assumed to remain fully elastic between its ends. In the meanwhile, the plastic zone methods based on interpolation shape functions requires members to be divided into several elements to accurately predict the second-order effects and spread-of-plasticity behavior of steel framed structures. It is generally recognized to be an "exact" and computationally expensive solution compared with the plastic hinge methods.

In recent years, Alemdar and White [15] presented several beam–column finite element formulations for full nonlinear distributed plasticity analysis of planar frame structures. The fundamental processes within the derivation of displacement-based,

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flexibility-based, and mixed element methods using Hermitian cubic polynomial functions are summarized. These formulations are presented using a total Lagrangian corotational approach and are also applicable to general beam–column elements for space structural analysis. Chiorean [16] proposed a beam–column method for nonlinear inelastic analysis of 3D semi-rigid steel frames. The nonlinear inelastic force–strain relationship and stability functions are used in representing the inelastic behavior and second-order effects, respectively. The advantage of this study is its ability to trace the spread of plasticity along the member length by using only one beam–column element per framed member in analysis 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 considerably affect the inelastic behavior of the steel frames, are not consistently used. Mazza [19] proposed a distributed plasticity model which a frame member are simulated by two plastic-zone segments with lengths l_{Pi}, l_{Pj} at the ends of the member and one elastic segment with length l at the middle. This method cannot accurately simulate spread of plasticity along the length of the member. To overcome the limitations of the above mentioned studies, this study will develop a fiber beam–colum element based on stability functions for nonlinear inelastic time-history analysis of space steel frames with semi-rigid connections.

This study presents a second-order spread-of-plasticity approach for nonlinear time-history analysis of space semi-rigid steel frames. The second-order effects are considered by the use of stability functions obtained from the closed-form solution of the beam–column element subjected to axial force and bending moments at the two ends. The spread of plasticity over the cross section and along the member length is captured by tracing uniaxial stress–strain relations of each fiber on the cross sections located at the selected integration points along the member length. Warping torsion and lateral–torsional buckling are ignored. An independent zero-length connection element with six translational and rotational springs is developed for beam-to-column joints with various connection types. This is efficient because modification of the beam–column stiffness matrix considering semi-rigid connections is unnecessary and the connection is ready to integrate with any element type. The Kishi–Chen three-parameter power model [20] and the Richard–Abbott four-parameter model [21] are applied for representing the moment–rotation relationship and predicting the instantaneous stiffness of connections. A numerical procedure based on the Hilber–Hughes–Taylor method combined with the Newton–Raphson method is developed to solve nonlinear differential equations of motion. Several numerical examples are presented to verify the accuracy, efficiency, and applicability of the proposed proceduce in predicting nonlinear inelastic time-history responses of space steel frames with semi-rigid connections.

2. Formulation

2.1. Nonlinear beam–column element

2.1.1. The effects of P-small delta and shear deformation

To capture the effect of axial force acting on bending moment through the lateral displacement of the beam–column element (P-small delta effect), the stability functions reported by Chen and Lui [22] are used to minimize modeling and solution time. Generally, only one element per member is needed to accurately capture the P-small delta effect. From Kim and Choi [8], the incremental force–displacement equation of space beam–column element accounting for transverse shear deformation effects can be expressed as

$$\begin{Bmatrix} \Delta P \\ \Delta M_{yA} \\ \Delta M_{yB} \\ \Delta M_{zA} \\ \Delta M_{zB} \\ \Delta T \end{Bmatrix} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & C_{1y} & C_{2y} & 0 & 0 & 0 \\ 0 & C_{2y} & C_{1y} & 0 & 0 & 0 \\ 0 & 0 & 0 & C_{1z} & C_{2z} & 0 \\ 0 & 0 & 0 & C_{2z} & C_{1z} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{GJ}{L} \end{bmatrix} \begin{Bmatrix} \Delta\delta \\ \Delta\theta_{yA} \\ \Delta\theta_{yB} \\ \Delta\theta_{zA} \\ \Delta\theta_{zB} \\ \Delta\phi \end{Bmatrix} \tag{1}$$

where ΔP, ΔM_{yA}, ΔM_{yB}, ΔM_{zA}, ΔM_{zB}, and ΔT are the incremental axial force, end moments with respect to y and z axes, and torsion, respectively; $\Delta\delta$, $\Delta\theta_{yA}$, $\Delta\theta_{yB}$, $\Delta\theta_{zA}$, $\Delta\theta_{zB}$, and $\Delta\phi$ are the incremental axial displacement, joint rotations, and angle of twist, respectively; E, G and J are the elastic modulus and shear modulus of a material and the torsional constant of a cross section respectively; C_{1y}, C_{2y}, C_{1z}, and C_{2z} are bending stiffness coefficients accounting for the transverse shear deformation effects and are defined as

$$C_{1y} = \frac{k_{1y}^2 - k_{2y}^2 + k_{1y}A_{sz}GL}{2k_{1y} + 2k_{2y} + A_{sz}GL} \tag{2}$$

$$C_{2y} = \frac{-k_{1y}^2 + k_{2y}^2 + k_{2y}A_{sz}GL}{2k_{1y} + 2k_{2y} + A_{sz}GL} \tag{3}$$

$$C_{1z} = \frac{k_{1z}^2 - k_{2z}^2 + k_{1z}A_{sy}GL}{2k_{1z} + 2k_{2z} + A_{sy}GL} \tag{4}$$

$$C_{2z} = \frac{-k_{1z}^2 + k_{2z}^2 + k_{2y}A_{sy}GL}{2k_{1z} + 2k_{2z} + A_{sy}GL} \tag{5}$$

where $k_{1n} = S_{1n}(EI_n/L)$ and $k_{2n} = S_{2n}(EI_n/L)$; S_{1n} and S_{2n} are stability functions with respect to the axis of n $(n = y, z)$ and are expressed as

$$S_{1n} = \begin{cases} \frac{k_nL\ \sin(k_nL) - (k_nL)^2\ \cos(k_nL)}{2 - 2\ \cos(k_nL) - k_nL\ \sin(k_nL)} & \text{if } P < 0 \\ \frac{(k_nL)^2\ \cosh(k_nL) - k_nL\ \sinh(k_nL)}{2 - 2\ \cosh(k_nL) + k_nL\ \sinh(k_nL)} & \text{if } P > 0 \end{cases} \tag{6}$$

$$S_{2n} = \begin{cases} \frac{(k_nL)^2 - k_nL\ \sin(k_nL)}{2 - 2\ \cos(k_nL) - k_nL\ \sin(k_nL)} & \text{if } P < 0 \\ \frac{k_nL\ \sin(k_nL) - (k_nL)^2}{2 - 2\ \cosh(k_nL) + k_nL\ \sinh(k_nL)} & \text{if } P > 0 \end{cases} \tag{7}$$

where $k_n^2 = |P|/EI_n$. EA and EI_n denote the axial and bending stiffness of the beam–column element and are integrated as follows:

$$EA = \sum_{j=1}^{s} w_j \left(\sum_{i=1}^{m} E_i A_i \right)_j \tag{8}$$

$$EI_y = \sum_{j=1}^{s} w_j \left[\sum_{i=1}^{m} E_i (A_i z_i^2 + I_{yi}) \right]_j \tag{9}$$

$$EI_z = \sum_{j=1}^{s} w_j \left[\sum_{i=1}^{m} E_i (A_i y_i^2 + I_{zi}) \right]_j \tag{10}$$

in which s is the total number of monitored sections along an element; m is the total number of fibers divided on the monitored cross-section; w_j is the weight coefficient for Lobatto quadrature at the jth section [23]; E_i and A_i are the elastic modulus of the material and the area of ith fiber, respectively; I_{yi} and I_{zi} are the y- and z-axis moment of inertia of ith fiber around its centroid; y_i and z_i are the coordinates of ith fiber to the centroidal bending axis of the cross-section as shown in Fig. 1. The element force–deformation relationship of Eq. (1) can be expressed in symbolic form as