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An advanced analysis method for three-dimensional steel frames with semi-rigid connections

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ABSTRACT

This paper presents an advanced analysis method for three-dimensional semi-rigid steel frames accounting for three main nonlinear sources. The second-order effects are considered by the use of stability functions obtained from the solution of beam–columns under axial force and bending moments at two ends. 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 3D multi-spring element. The generalized displacement control method is applied to solve the nonlinear equilibrium equations in an incremental-iterative scheme. The nonlinear load–displacement responses and ultimate load results compare well with those of previous studies. It is concluded that using only one element per member with monitoring the end sections accurately likely predict the nonlinear responses of three-dimensional semi-rigid steel frames.

1. Introduction

There are two common finite element approaches for advanced analysis of three-dimensional (3D) steel framed structures: the plastic zone methods (spread-of-plasticity) [1–5] and the plastic hinge methods (concentrated plasticity) [6–13]. The former methods based on geometric stiffness matrices requires member to be discretized into several elements to accurately predict the second-order effects and inelastic behavior of steel structures. It is generally recognized to be computationally expensive (computer resources, computational time) because numerous discretizations of elements are used in analysis modeling. Clarke [1], and Teh [5] presented finite element formulations using Hermitian cubic polynomial functions for plastic-zone analysis of 3D framed structures. Foley and Vinnakota [3,4] developed a nonlinear finite element program for second-order spread-of-yielding analysis of 2D multi-storey semi-rigid steel frames under static loading. In order to save computer resources and analysis time, Foley [2] proposed a parallel processing and vectorization for advanced analysis of multi-storey steel frames on a multi-core computer. Structures are divided into some sub-structures for reducing the unknown of nonlinear equilibrium equations. The plastic hinge methods [6,8–12,14,15] based on stability functions obtained from the closed-form solution of the beam–columns under end forces can capture the second-order effects using only one or two elements per member. The inelastic behavior of material is usually considered by the lumped hinges at the two ends of the members. The effects of spread of plasticity and residual stress are considered by using the reduced tangent modulus approach [6,8–10,14]. This method is called the "practical advanced analysis method" because it can consider all key factors, influencing the ultimate strength of the steel framed structures in an effective way, and is then suitable for adoption in office design [6–11,13–15]. However, the plastic hinge analysis is limited due to its 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 the structure [16]. Furthermore, the hinge methods have shown to over-estimate the limit strength when structural behavior is dominated by the instability of a few members [14]. 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 recent years, Ziemian and McGuire [13] proposed a modified tangent modulus approach for the second-order inelastic hinge method that can produce the accuracy of more sophisticated plastic zone methods in analyzing the in-plane behavior of compact doubly symmetric sections. Researchers recommended that further systematic research be undertaken for the purpose of establishing the terms, constants, and limits of applicability for

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additional groupings of sections, imperfections, and residual stresses. Ngo-Huu and Kim [12] improved the common plastic hinge method to become the fiber-hinge method for considering the initial residual stress directly. However, the length of fiber hinges was not adequately investigated and it also cannot capture the effects of distributed plasticity along the member, so it is difficult to apply it towards practical designs. Chiorean [17] 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 that it is able 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. Recently, Thai and Kim [18] presented a fiber beam–column element which considers both geometric and material nonlinearities. The material nonlinearities are included by tracing the uniaxial stress–strain relationship of each fiber on the cross sections. However, updating of the elemental tangent stiffness matrix based on the tangent modulus of each fiber calculated from the ratio of the incremental fiber stress and incremental fiber strain is not rational for the elastic-perfectly plastic material model of steel. This is not rational because with the elastic-perfectly plastic material, the tangent modulus of steel fibers are only equal to the initial Young's modulus, or zero as yielded fibers. To overcome the limitations of the above mentioned studies, this study will develop a fiber beam–column element for advanced analysis of 3D steel frames with nonlinear semi-rigid connections.

A different important nonlinear source in advanced analysis of steel frames is beam-to-column connections. In reality, beam-to-column connections are not fully rigid or ideally pinned joints, they play a role in transfering a part of the forces from some elements to other ones, and the rest is absorbed by connections. The real behavior of connections is nonlinear and usually presented by the moment–rotation relationship of rotational springs with zero-lengths. In the above-mentioned studies, the connections are usually modeled as rotational springs attached at the member ends, then, the elemental tangent stiffness matrix will be modified for considering the effects of connection flexibility [3,4,8,9,17,19].

In this paper, a fiber beam–column method is developed for the second-order inelastic analysis of 3D semi-rigid steel frames. The spread of plasticity over the cross section and along the member length is captured by tracing the uniaxial stress–strain relations of each fiber on the cross sections located at the selected integration points along the member length. The Gauss–Lobatto integration rule is adopted herein for numerically evaluating the elemental stiffness matrix instead of the classical Gauss integration rule because it always includes the end sections of the integration field. Stability functions obtained from the closed-form solution of a beam–column subjected to end forces are used to accurately capture the small P-delta effect. A new force interpolation function matrix is developed to consider the effects of moment magnification due to axial force and lateral displacements. Warping torsion and lateral-torsional buckling are not considered in this study. An independent two-node zero-length connection element with six translational and rotational springs is developed for nonlinear beam-to-column joints and various connections. This is efficient because modification of the beam–column stiffness matrix considering the semi-rigid connections is unnecessary and the connection is ready to integrate with any element types (e.g. truss joints, bridge joints, etc.). The Kishi–Chen three-parameter power model [20] and the Chen–Lui exponential model [21] are applied for representing the moment–rotation relationship and predicting the instantaneous stiffness of connections. The generalized displacement control (GDC) method proposed by Yang and Shieh [22] is employed for solving nonlinear equilibrium equations. Several numerical examples are presented to verify the accuracy, efficiency, and applicability of the proposed proceduce in predicting the second-order inelastic response of 3D steel frames with semi-rigid connections.

2. Formulation

2.1. Nonlinear beam–column element

2.1.1. Small P-delta and shear deformation effects

To capture the effect of axial force acting through the lateral displacement of the beam–column element (small P-delta effect), the stability functions reported by Chen and Lui [23] are used to minimize modeling and solution time. Generally only one element per member is needed to accurately capture the small P-delta effect. From Kim and Choi [9], the incremental force–displacement equation of 3D beam–column element that accounts for transverse shear deformation effects can be expressed as

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

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; C_{1y}, C_{2y}, C_{1z}, and C_{2z} are bending stiffness coefficients accounting for the transverse shear deformation effects, and are defined as

$$C_{1y} = \frac{k_{1y}^2 - k_{2y}^2 + k_{1y}A_{sz}GL}{2k_{1y} + 2k_{2y} + A_{sz}GL} \quad (2a)$$

$$C_{2y} = \frac{-k_{1y}^2 + k_{2y}^2 + k_{2y}A_{sz}GL}{2k_{1y} + 2k_{2y} + A_{sz}GL} \quad (2b)$$

$$C_{1z} = \frac{k_{1z}^2 - k_{2z}^2 + k_{1z}A_{sy}GL}{2k_{1z} + 2k_{2z} + A_{sy}GL} \quad (2c)$$

$$C_{2z} = \frac{-k_{1z}^2 + k_{2z}^2 + k_{2y}A_{sy}GL}{2k_{1z} + 2k_{2z} + A_{sy}GL} \quad (2d)$$

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 n axis ($n = y$, z), and are expressed as

$$S_{1n} = \begin{cases} \frac{k_nL \ \sin(k_nL) - (k_nL)^2 \ \cos(k_nL)}{2 - 2\cos(k_nL) - k_nL \ \sin(k_nL)} & \text{if } P < 0 \\ \frac{(k_nL)^2 \ \cosh(k_nL) - k_nL \ \sinh(k_nL)}{2 - 2\cosh(k_nL) + k_nL \ \sinh(k_nL)} & \text{if } P > 0 \end{cases} \quad (3a)$$

$$S_{2n} = \begin{cases} \frac{(k_nL)^2 - k_nL \ \sin(k_nL)}{2 - 2\cos(k_nL) - k_nL \ \sin(k_nL)} & \text{if } P < 0 \\ \frac{k_nL \ \sin(k_nL) - (k_nL)^2}{2 - 2\cosh(k_nL) + k_nL \ \sinh(k_nL)} & \text{if } P > 0 \end{cases} \quad (3b)$$

where $k_n^2 = |P|/EI_n$. EA, EI_n, and GJ denote the axial, bending and torsional stiffness of the beam–column element, and are