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Nonlinear elastic dynamic analysis of space steel frames with semi-rigid connections

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ABSTRACT

This paper presents a simple effective numerical procedure based on the beam–column method for nonlinear elastic dynamic analysis of three-dimensional semi-rigid steel frames. The geometric nonlinearity is considered by using stability functions and geometric stiffness matrix. An independent zero-length connection element comprising six translational and rotational springs is used to simulate the steel beam-to-column connection. The dynamic behavior of rotational springs is captured through the independent hardening model. The Newmark numerical integration method combined with the Newton–Raphson iterative algorithm is adopted to solve the nonlinear equations. The nonlinear elastic dynamic analysis results are compared with those of previous studies and commercial SAP2000 software to verify the accuracy and efficiency of the proposed analysis.

1. Introduction

Beam-to-column joints of steel frames are usually assumed to be rigid or pinned connections in structural design. This assumption causes an inaccurate estimation of the response of frames since real beam-to-column joints are between fully rigid and pinned connections. A variety of static experiments have been carried out to investigate the nonlinear behavior of semi-rigid connections [1–3]. Several dynamic tests have also been performed to investigate the energy dissipation which is one of the important features of semi-rigid connections under cyclic loadings [3–6].

Several mathematic models for semi-rigid connections were proposed for representing moment–rotation relationship curves, these models can be grouped into two categories: linear semi-rigid connection models and nonlinear semi-rigid connection models. In linear semi-rigid connection models, the stiffness of connections is assumed to be constant and stiffness matrix of a beam–column member is usually modified by using end-fixity factors [7,8]. The advantage of these models is simple in formulation and implementation. However, these models do not consider the nonlinear behavior of semi-rigid connections, and furthermore they ignore the energy dissipation at connections. In nonlinear semi-rigid connection models [9–12], the stiffness of connections varies corresponding to different loading magnitudes and therefore these models can accurately capture the moment–rotation relationship as well as consider the energy dissipation.

In the last two decades, nonlinear dynamic behavior of plane frames with semi-rigid connections is extensively investigated but space frames with semi-rigid connections are rarely concerned. Recently da Silva et al. [13] successfully developed a modeling strategy to represent the dynamic behavior of semi-rigid joints by using the ANSYS finite element software. This analysis method employed approximate shape functions, hence, if beam–column members are divided into a lot of elements, the second-order effects will be predicted more exactly so that it consumes computational time and computer resources intensively. An another method is the beam–column approach using stability functions derived from the closed-form solutions of the differential equation of the beam–column element subjected to the end forces and therefore it is able to evaluate the second-order effects exactly by using only one element per member. It can be seen that the beam–column approach using stability functions saves computer resources and it reduces computational time.

By using stability functions, Lui and Lopes [14] and Awkar and Lui [15] carried out an investigation on the dynamic behavior of plane steel frames, in which the moment–rotation curves of semi-rigid connections were represented by the bilinear model, which do not accurately capture the nonlinear behavior of connections. In the all above mentioned studies, the connections were modeled as single rotational springs attached at beam ends and the stiffness matrix of beam elements is modified to account for the stiffness of connections. The mentioned studies are limited to planar semi-rigid steel frames, but space frames are not analyzed.

In this paper, an independent zero-length connection element with six different translational and rotational springs connecting two different nodes with zero distance is developed. This is efficient because modification of the beam–column stiffness matrix considering the

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semi-rigid connections is unnecessary and the connection is ready to integrate any element types. The dynamic behavior of rotational springs is captured through the independent hardening model employing the well-known Kishi–Chen power model [9], the Richard–Abbott four-parameter model [10], the Chen–Lui exponential model [11], and the Ramberg–Osgood model [12]. Rotational springs with constant stiffness are used to model linear semi-rigid connections.

The second-order effects are considered by using stability functions which Kim and Thai [16–18] have successfully employed for the nonlinear inelastic analysis of steel structures subjected to static and dynamic loadings. The Newmark numerical integration method combined with the Newton–Raphson iterative algorithm is adopted to solve the nonlinear motion equations at each incremental time step. The results of the second-order elastic dynamic response are compared with those of previous studies and commercial SAP2000 software [19] to demonstrate the accuracy and computational efficiency.

2. Element formulation

2.1. Nonlinear beam–column element

To capture the effect of axial force acting through the lateral displacement of the beam–column element ($P-\delta$ effect), the stability functions reported by Chen and Lui [20] are used to minimize the modeling and solution time. Only one element per member is generally needed to accurately capture the $P-\delta$ effect. The material nonlinearity including gradual yielding of a steel beam–column member under axial force and bending moments is beyond the scope of this study. The incremental force–displacement equation of a space beam–column element can be expressed in accordance with Kim and Thai [16]:

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

where E and G are the elastic and shear modulus of material; A and L are the area and length of beam–column element; J is the torsional constant; I_n is the moment of inertia with respect to the n axes ($n = y,z$); ΔP, ΔM_{yA}, ΔM_{yB}, ΔM_{zA}, ΔM_{zB}, and ΔT are the incremental axial force, A and B 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; S_{1n} and S_{2n} are the stability functions with respect to the n axis; ($n = y, z$), and are expressed as:

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

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

where $k_n = \sqrt{|P|/EI_n}$.

The element force–deformation relationship of Eq. (1) is expressed in symbolic form as follows:

$$\{\Delta F\} = [K_e]\{\Delta d\} \quad (3)$$

in which

$$\{\Delta F\} = \begin{bmatrix} \Delta P & \Delta M_{yA} & \Delta M_{yB} & \Delta M_{zA} & \Delta M_{zB} & \Delta T \end{bmatrix}^T \quad (4)$$

$$\{\Delta d\} = \begin{bmatrix} \Delta\delta & \Delta\theta_{yA} & \Delta\theta_{yB} & \Delta\theta_{zA} & \Delta\theta_{zB} & \Delta\phi \end{bmatrix}^T. \quad (5)$$

The P–Δ effect is the influence of axial force P acting through the relative transverse displacement of the member ends. This effect can be considered by using the geometric stiffness matrix $[K_g]$ as

$$\left[K_g\right]_{12\times12} = \begin{bmatrix} [K_s] & -[K_s] \\ -[K_s]^T & [K_s] \end{bmatrix} \quad (6)$$

where

$$[K_s] = \begin{bmatrix} 0 & a & -b & 0 & 0 & 0 \\ a & c & 0 & 0 & 0 & 0 \\ -b & 0 & c & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \quad (7)$$

and

$$a = \frac{M_{zA}+M_{zB}}{L^2},\ b\frac{M_{yA}+M_{yB}}{L^2},\ c = \frac{P}{L}. \quad (8)$$

The displacement of a beam–column element can be decomposed into two parts: the element deformation and rigid displacement. The element deformation increment $\{\Delta d\}$ in Eq. (3) can be obtained from the element incremental displacement $\{\Delta D\}$ as

$$\{\Delta d\} = [T]_{6\times12}\{\Delta D\} \quad (9)$$

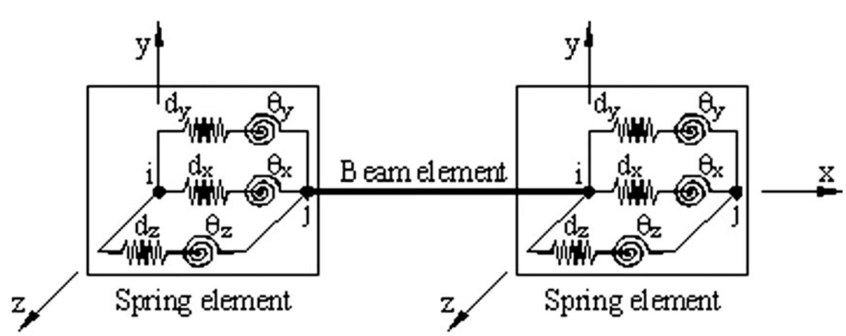

Fig. 1. Spring element model with zero-length.