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Distributed plasticity approach for time-history analysis of steel frames including nonlinear connections

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ABSTRACT

This paper presents a displacement-based finite element procedure for second-order spread-of-plasticity analysis of plane steel frames with nonlinear beam-to-column connections under dynamic and seismic loadings. A partially strain-hardening elastic–plastic beam-column element, which directly takes into account geometric nonlinearity, gradual yielding of material, and flexibility of nonlinear connections, is proposed. Three major sources of damping are considered at the same time. They are structural viscous damping, hysteretic damping due to inelastic material, and hysteretic damping due to nonlinear connections. A nonlinear solution procedure based on the combination of the Hilber–Hughes–Taylor method and the well-known Newton–Raphson equilibrium iterative algorithm is proposed for solving differential equations of motion. The dynamic behavior predicted by the proposed program compares well with those given by the commercial finite element software ABAQUS and previous studies. Coupling effects of three primary sources of nonlinearity, the bowing effect, geometric imperfections, and residual stress are investigated and discussed in this paper.

1. Introduction

Conventional designs usually assume that beam-to-column connections are fully rigid or ideally pinned. This assumption causes an inaccurate prediction of the seismic response of moment-resisting steel frames because the real moment–rotation relationship of connections is a nonlinear curve, and such connections are called semi-rigid connections. Several dynamic tests were carried out to investigate the ductile and stable hysteretic behavior of steel frames, which is one of the important features of semi-rigid connections under cyclic and seismic loadings [1–6].

In order to predict actual behavior of steel frames, especially in severe loading conditions, advanced analysis methods are employed. An advanced analysis must include key factors of steel frames such as geometric nonlinearities (P-large delta and P-small delta effects), plasticity of material, nonlinear connections, geometric imperfections (out-of-straightness and out-of-plumbness), and residual stress, simultaneously. There are two beam-column approaches for advanced analysis of steel frame structures: (i) the plastic hinge approach (concentrated plasticity) and (ii) the distributed plasticity approach (spread-of-plasticity). In the former approach, once yielding criteria is obtained, a plastic hinge will form at one of monitored points on the member (usually at the two ends). This method is a computationally efficient and simple way to consider the effect of inelastic material. However, the hinge methods overpredict the limit strength of structures [7–9], which can also lead to unsafe designs. What's more, it may inadequately give information as to what is happening inside the member because the member is assumed to remain fully elastic between plastic hinges. On the other hand, by the distributed plasticity approach, yielding spreads throughout the whole length and depth of members. Therefore, the distributed plasticity method is more accurate than plastic hinge methods in capturing the inelastic behavior of frame structures under severe loadings.

In the last two decades, there have not been many analytical studies about the second-order inelastic dynamic behavior of steel frames with nonlinear semi-rigid connections [10–14]. Gao and Haldar [10] presented an efficient and robust finite-element-based method for estimating nonlinear responses of space structures with partially restrained connections under dynamic and seismic loadings. Lui and Lopes [11] proposed an approach for dynamic analysis of semi-rigid frames using stability functions, the tangent modulus concept, and the bilinear model for capturing the effects of geometrical nonlinearities, inelastic behavior, and connection flexibility, respectively. In 1999, Awkar and Lui [12] developed the method of Lui and Lopes [11] for multi-story semi-rigid frames. Chan and Chui [13] published a book about static and dynamic analysis of semi-rigid steel frames, in which they proposed a spring-in-series model for simulating material plasticity and nonlinear connections; both plastic hinge and refined plastic-hinge methods are presented in detail. Recently, Sekulovic and Nefovska-Danilovic [14] applied the refined plastic hinge method and the spring-in-series concept proposed by Chan and Chui [13] for transient analysis of inelastic steel frames with nonlinear connections; however, their study ignored the P-small delta effects. All the above mentioned studies utilized the plastic hinge methods. Thus, analytical researches about the second-order

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distributed plasticity analysis of semi-rigid steel frames under dynamic loadings are uncommon.

In this paper, a sophisticated second-order spread-of-plasticity method proposed by Foley and Vinnakota [15–18] for static analysis is developed for nonlinear inelastic time-history analysis of plane semi-rigid steel frames. An elastic–perfectly plastic model with linear strain hardening is applied to establish a new nonlinear element tangent stiffness matrix based on the principle of stationary potential energy. Accurately, to capture the second-order effects and spread of plasticity, each frame member is divided into many sub-elements along the member length and the cross-section depth. The tangent stiffness matrix of the nonlinear beam-column element directly takes into account the effects of geometric nonlinearity, gradual yielding, and flexibility of nonlinear connections. Nonlinear connections are simulated by zero-length rotational springs. The moving of the strain-hardening and elastic neutral axis, which are due to gradual yielding of the cross-section, is directly included in the element tangent stiffness matrix, and this effect is updated during the analysis process. The bowing effect, geometrical imperfections, and residual stress are also considered in this study. Three major sources of damping are integrated in the same analysis. They are structural viscous damping, hysteretic damping due to nonlinear connections, and hysteretic damping due to material plasticity. A numerical procedure using the Hilber–Hughes–Taylor (HHT) method [19] and the well-known Newton–Raphson iterative algorithm is proposed to solve nonlinear equations of motion. Several numerical examples are performed to illustrate the accuracy, validity, and features of the proposed second-order inelastic dynamic analysis procedure for steel frames with nonlinear flexible connections.

2. Nonlinear finite element formulation

2.1. Beam-column element including the second-order effects and distributed plasticity

Investigation of a typical beam-column member subjected to loads is plotted in Fig. 1. In order to capture the distributed plasticity, the beam-column member is divided into n elements along the member length as illustrated in Fig. 2; each element is divided into m small fibers within its cross section as illustrated in Fig. 3; and, each fiber is represented by its material properties, geometric characteristic, area A_j, and its coordinate location (y_j, z_j) corresponding to its centroid. This way, residual stress is directly considered in assigning an initial stress value for each fiber. The second-order effects are included by the use of several sub-elements per member through updating of the element stiffness matrix and nodal coordinates at each iterative step.

To reduce the computational time when assembling the structural stiffness matrix and solving the system of nonlinear equations, n sub-elements are condensed into a typical beam-column member with the six degrees of freedom at the two ends by using the static condensation algorithm derived by Wilson [20]. A reverse condensation algorithm is used to find the displacements along the member length for evaluating the effects of distributed plasticity and the second-order effects. The Appendix C presents the static condensation procedure in detail.

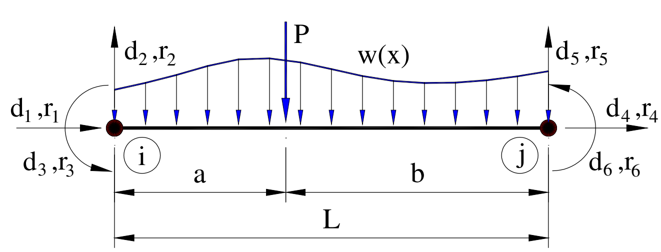

Fig. 1. Beam-column element modeling under arbitrary loads.

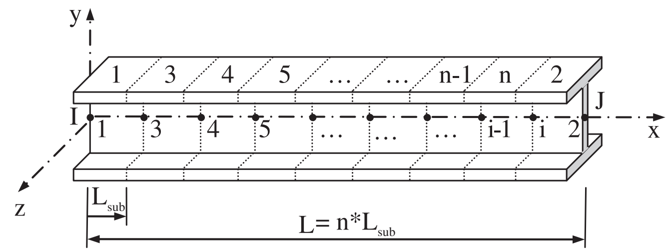

Fig. 2. Meshing of beam-column element into n sub-elements.

In the development of the second-order spread-of-plasticity beam-column element, the following assumptions are made: (1) the element is initially straight and prismatic; (2) plane cross-sections remain plane after deformation and normal to the deformed axis of the element; (3) out-of-plane deformations and the effect of Poisson are neglected; (4) shear strains are negligible; (5) member deformations are small, but overall structure displacements may be large; (6) residual stress is uniformly distributed along the member length; (7) yielding of the cross-section is governed by normal stress alone; (8) the material model is linearly strain-hardening elastic–perfectly plastic; and, (9) local buckling of the fiber elements does not occur. In this study, an elastic–perfectly plastic stress–strain relationship with linearly strain hardening used by Toma and Chen [21] is adopted as shown in Fig. 4. 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 beam-column element can be expressed as follows:

$$U = \int_V \int_\varepsilon \sigma d\varepsilon dV. \tag{1}$$

The normal stresses corresponding to the strain state of fibers are calculated as follows:

$$\begin{aligned} &\sigma = E\varepsilon && \text{for elastic fibers} \\ &\sigma = E\varepsilon_y = \sigma_y && \text{for yielding fibers} \\ &\sigma = E\varepsilon_y + E_{sh}(\varepsilon - \varepsilon_{sh}) = \sigma_{sh} && \text{for hardening fibers.} \end{aligned} \tag{2}$$

The total internal strain energy of a partially strain-hardening elastic–plastic beam-column element can be expanded as

$$\begin{aligned} U = &\int_{V_e}\int_0^\varepsilon E\varepsilon d\varepsilon dV_e + \int_{V_p}\left(\int_0^\varepsilon \sigma_y d\varepsilon - \int_0^{\varepsilon_y} E\varepsilon d\varepsilon\right) dV_p \\ &+ \int_{V_{sh}}\left\{\int_0^\varepsilon \sigma_y d\varepsilon - \int_0^{\varepsilon_y} E\varepsilon d\varepsilon + \int_{\varepsilon_{sh}}^\varepsilon E_{sh}(\varepsilon - \varepsilon_{sh}) d\varepsilon\right\} dV_{sh} \end{aligned} \tag{3}$$

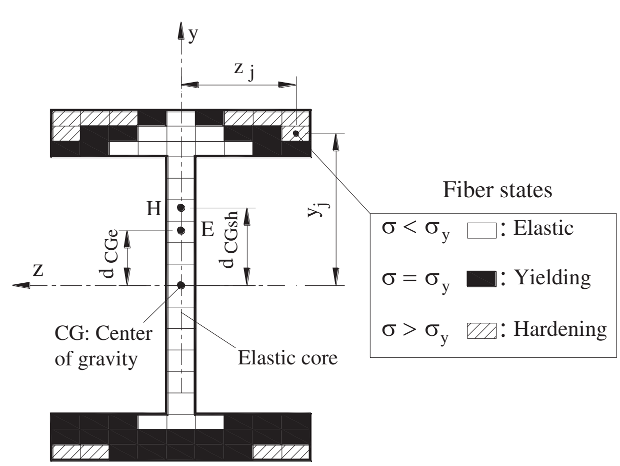

Fig. 3. Illustration of meshing of element cross-section and states of fibers.