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An efficient compound-element for potential progressive collapse analysis of steel frames with semi-rigid connections

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ABSTRACT

In this paper, the formulation of a novel 1D frame compound-element for the materially and geometrically non-linear analysis of steel frames with flexible connections is outlined. The element is formulated based on the force interpolation concept and the total secant stiffness approach, and implemented in a FORTRAN computer code. The accuracy and efficiency of the formulation are verified through some numerical examples. For steel frames with bolted flush end-plate and extended end-plate connections, a static and dynamic progressive collapse assessment based on the alternate load path (ALP) method is undertaken by employing the developed analytical tool and dynamic load factor (DLF) is estimated. Furthermore, the implications of analyzing semi-rigid steel frames based on the assumption of fixed connections and the effects of the connection details on the global response of a frame during different progressive collapse scenarios are investigated.

1. Introduction

Steel frames are usually designed based on the assumption of either idealized rigid or pinned beam-to-column connections (or joints) [1]. It has been well-established that the majority of beam-to-column joints, however, do not show such idealized behavior. Connections of this type are known flexible (semi-rigid connections), and their rigorous inclusion in the analysis and design of steel framed structures is fraught with difficulty.

Over the last two decades, a large number of studies have been devoted to steel and steel–concrete composite frames with semi-rigid connections [2–6]. Studies covering various aspects of steel and composite frames with flexible connections, such as the global behavior of the frame including material and geometrical non-linearities [7–10], theoretical and experimental investigations of behavioral models for the connections [11–15], the global buckling and stability of steel frames [16,17] and analyses of semi-rigid steel frames under cyclic, seismic and blast loads [18–20].

For the non-linear finite element analysis of steel frames, two different classes of model are available, viz. continuum-based formulations and discrete 1-D frame elements, which have different domains of applicability [4,11]. Non-linear continuum-based models offer good versatility and accuracy which are required for the detailed study of local effects, but they are very time-demanding from a computational point of view for the analysis of multiple storey–multiple bay frames with large numbers of degrees of freedom. Such demands on computational resources make the continuum-based FE modeling of large structures inefficient and inapplicable. Discrete 1-D frame models, however, are a good compromise between accuracy and efficiency for predicting the global response of framed structures [2,7,8,21–25].

Progressive collapse is an important issue in structural failure, and has been so since the well-reported partial collapse of the Ronan point apartment building in London in 1968. Since this time, progressive collapse analysis has been the subject of much research endeavor with regard to the global response of members [26–31], however, less attention has been paid to the effect of the stiffness and strength of the joints and their behavior on the global response [31–35]. One procedure for investigating the potential for progressive collapse is based on the so-called alternative path method (APM), which has been integrated into several building codes [36,37], and in different forms has been adopted by researchers for the numerical modeling of reinforced concrete and steel frames [22,28,29,38,39]. In the APM approach, one or more columns are assumed to fail and are removed from the structural model with the remaining structure analyzed to determine whether other members (or the structure) will fail or not.

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In this paper, the force interpolation concept is employed to formulate an efficient non-linear compound-element, comprising of a frame element with nodal springs. The geometrical non-linearity is taken into account by decomposing the element displacements into a rigid body rotation and deformations [40]. The effect of the transverse displacement on the axial strain is taken into account; however, the strains are taken to be small. The element formulation is implemented in a FORTRAN computer code and the numerical tool developed is employed for static and dynamic progressive collapse assessment of steel frames with bolted flush end-plate and extended end-plate connections and the dynamic load factor (DLF) is estimated. Furthermore, the effects of connection details (i.e. the position of the bolts and the end-plate thickness) on the global response of steel frames with semi-rigid connections during scenarios of progressive collapse are investigated.

2. Element formulation

2.1. Compatibility equations

Adopting the Navier–Bernoulli assumptions, section compatibility requirements produce

$$\varepsilon_x = \varepsilon_r - y\kappa, \tag{1}$$

where ε_x denotes the total strain at an integration point in the local x–x direction (along the element axis; Fig. 1(a)), ε_r is the section axial strain, κ is the total curvature of section and y is the distance of the integration point (fiber) from the mid-plane of the element (Fig. 1(a)).

Fig. 1(a) shows a 2-node plane frame element AB with three degrees of freedom at each node. Furthermore, at each nodal point the element is attached to a rotational and a translational spring, which represent the flexural and axial stiffness of the connections at this point respectively (Fig. 1(a)). The generalized nodal displacement and force vectors (with rigid body modes included) are denoted by $\mathbf{q}$ and $\mathbf{Q}$, respectively. Using the principle of virtual force and integration by parts for the simply supported configuration shown in Fig. 1(b), the strain-deformation compatibility equation for the element (without nodal springs) is obtained as

$$\overline{\mathbf{q}}' = \int_0^l \overline{\mathbf{b}}^{\mathrm{T}}[x,w(x)]\mathbf{d}(x)\mathrm{d}x, \tag{2}$$

where

$$\overline{\mathbf{b}}[x,w(x)] = \begin{bmatrix} -1 & 0 & 0 \\ -w(x)/2 & x/l-1 & x/l \end{bmatrix}, \tag{3}$$

$\overline{\mathbf{q}}' = \begin{bmatrix} \overline{q}_1' - \overline{q}_1'' & \overline{q}_2' & \overline{q}_3' \end{bmatrix}^{\mathrm{T}}$ is the generalized nodal deformation vector of the frame element excluding the nodal springs (without rigid body modes) and $\mathbf{d}(x) = \begin{bmatrix} \varepsilon_r & \kappa \end{bmatrix}^{\mathrm{T}}$ is the section generalized strain vector.

2.2. Equilibrium equations and constitutive material law for steel

Adopting the small slope assumption ($\sin\theta \cong \tan\theta \cong \theta = w'$), the equilibrium equations for the free body of Ax (Fig. 2) produce

$$N(x) - V(x)\theta(x) + \overline{Q}_1 = 0, \tag{4}$$

$$N(x)\theta(x) + V(x) + (\overline{Q}_2 + \overline{Q}_3)/l = 0, \tag{5}$$

$$M(x) + \overline{Q}_1 w(x) + (1 - x/l)\overline{Q}_2 - (x/l)\overline{Q}_3 = 0, \tag{6}$$

where $N(x)$ and $V(x)$ represent the section normal (axial) and tangential (shear) forces respectively, and $\theta(x)$ is the section rotation.

Rearranging Eqs. (4)–(6), leads to the matrix representation

$$\overline{\mathbf{D}}(x) = \mathbf{b}[x,w(x),\theta(x)]\overline{\mathbf{Q}} + \overline{\mathbf{D}}^*(x), \tag{7}$$

where

$$\mathbf{b}[x,w(x),\theta(x)] = \begin{bmatrix} -1 & -\theta(x)/l & -\theta(x)/l \\ -w(x) & x/l-1 & x/l \end{bmatrix}, \tag{8}$$

$\overline{\mathbf{Q}} = \begin{bmatrix} \overline{Q}_1 & \overline{Q}_2 & \overline{Q}_3 \end{bmatrix}^{\mathrm{T}}$ denotes the nodal force vector (in the system without rigid body modes), $\overline{\mathbf{D}}(x) = \begin{bmatrix} N(x) & M(x) \end{bmatrix}^{\mathrm{T}}$ is the section internal force vector and $\overline{\mathbf{D}}^*(x)$ is the section internal force vector due to the element load. It is noteworthy that $\overline{\mathbf{D}}^*(x)$ is a function of the element rigid body rotation when gravity loads act on the element.

Equilibrium across the section requires that

$$\overline{\mathbf{D}}(x) = \begin{bmatrix} \int_\Omega \sigma_x \,\mathrm{d}A & -\int_\Omega y\sigma_x \,\mathrm{d}A \end{bmatrix}^{\mathrm{T}}, \tag{9}$$

where y is the distance of the integration point from the element mid-plane, σ_x is the total x–x stress component at the monitoring points and Ω represents the cross-sectional domain.

Decomposing the total strain ε_x into its elastic ε_{ex} and inelastic ε_{px} components, the total stress and strain in the x–x direction can

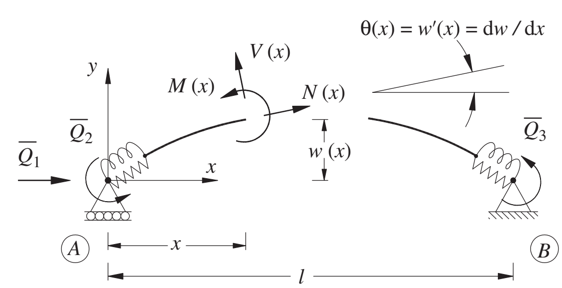

Fig. 2. Equilibrium in the simply supported configuration and free body diagram of Ax, after deformation (system without rigid body modes).

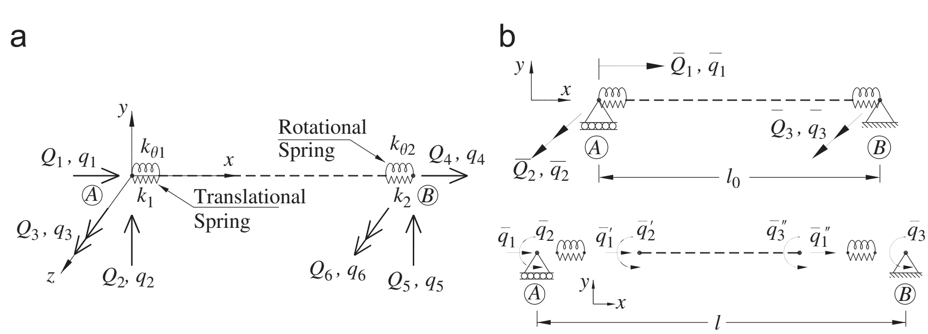

Fig. 1. (a) 2-node frame element AB in x–y plane and (b) outline of the simply supported configuration (system without rigid body modes).