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Nonlinear dynamic collapse analysis of semi-rigid steel frames based on the finite particle method

Ying Yu [a], Xingyi Zhu [b,*]

[a] Department of Civil Engineering, Shantou University, 515063 Shantou, China
[b] Key Laboratory of Road and Traffic Engineering of Ministry of Education, Tongji University, Shanghai 200092, China

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ABSTRACT

This paper presents a numerical procedure based on the finite particle method (FPM) for the nonlinear dynamic collapse analysis of plane steel frames with semi-rigid connections. The FPM can be used to model the plane frame with finite separated particles; in particular, geometric nonlinearity and dynamic fracture. Fictitious motion was used to consider the geometrical nonlinearity, and the explicit time integration method was used for solving the dynamic equilibrium equations. In the FPM, particles are free to separate from each other, which is advantageous in the simulation of member fracture. To simulate member fracture, the fracture criterion and fracture model of the FPM were developed. Furthermore, to simulate the rigidity of a steel beam–column connection, an independent zero-length element was adopted, which can consider the influence of nonlinear and hysteretic connection stiffness. The nonlinear response results were compared with those of existing studies to verify the accuracy of the proposed numerical procedure. Additionally, the collapse analysis of a Vogel six-story frame showed that frames with semi-rigid connections have greater anti-collapse capacity than those with rigid connections do.

1. Introduction

Beam–column connections of steel frames are neither fully rigid nor ideally hinged. In reality, the behavior of connections is more complicated than in the two aforementioned simple extreme cases. The rigidity of a semi-rigid connection affects the strength and displacement response of steel frames considerably, and therefore, over the past 20 years, several numerical studies [1–5] and experiments [6–8] have been conducted to address the connection rigidity.

To construct a mathematical model of the connection behavior on the basis of experimental observations, both linear and nonlinear semi-rigid connection models are used in the numerical simulations. The stiffness of linear semi-rigid connections is assumed to be constant [9,10], whereas that of nonlinear semi-rigid connections varies with the loading magnitudes [11–14]. Therefore, although the linear model can be easily implemented, it does not consider the nonlinear behavior of semi-rigid connections. By contrast, the nonlinear model is more accurate in capturing the nonlinear moment–rotation relationship. In addition to the relationship between the bending moment and rotation, physical models of semi-rigid joins, including the zero-length spring element [15] and the multi-degree of freedom spring system [16,17], have also attracted the attention of numerous researchers, and so on.

In all the aforementioned studies, the nonlinear dynamic behavior of plane frames with semirigid connections has been extensively investigated, but the collapse behavior of these semi-rigid frames has rarely been examined. The main reason for this is that the numerical models in the studies were based on the conventional finite element method (FEM). The collapse of structures involves strong nonlinearities and discontinuities, and therefore, viable alternatives to the standard FEM must be considered. Thus, collapse behavior is not easy to investigate without special treatments and modifications.

Unlike the Finite Element Method and other mesh-free methods, the Finite Particle Method (FPM) [18–21] is derived from vector mechanics [22,23]. This method can be used to model a domain consisting of finite particles instead of a continuous mathematical body and involves the use of Newton's second law to describe the motions of all particles. In this method, unlike the FEM, the equilibrium equations for stress are not derived from variational principles. Equilibrium is instead enforced on each particle, resulting in the nodal internal force and external force being constantly balanced. In the basic numerical method used in the present study, strategies for addressing geometric nonlinearity and dynamic fracture are simple and straightforward.

* Corresponding author.
E-mail address: zhuxingyi66@aliyun.com (X. Zhu).

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The FPM is superior for the nonlinear dynamic collapse analysis of semi-rigid steel frames for the following reasons: (1) Fictitious motion is used to separate rigid body motion and pure deformation of the system, and therefor, geometrical nonlinearity can be addressed naturally in this formulation. (2) The motion equation of each particle is a vector equation and every term of the equation is clearly expressed. Because explicit time integration is used, the nonlinear dynamic response of steel frames can be obtained without any iteration. (3) Because the FPM is a particle method, it is possible to add or delete particles and elements in the analysis domain, which is crucial for the simulation of member fracture. Recently, the FPM has been successfully applied to the mechanism analysis of kinematically indeterminate bar assemblies [18], motion analysis of deployable structures containing beam and rod hinge element [19], post-buckling analysis of space structures [20], and progressive failure analysis of a cantilever truss structure [21].

One of the main objectives of this study was to propose a modeling strategy involving the FPM for appropriately representing the semirigid connection response under dynamic loads, considering geometric and material nonlinearity. Another crucial investigation was related to member fracture resulting from accidental loading or an earthquake, which introduces structural discontinuity and changes the structural topology.

The remainder of this paper is organized as follows. The fundamentals of the FPM, including the discretization of the structure, the basic particle motion equations, and solution procedures, are briefly described. FPM techniques for analyzing geometrical nonlinearities and member fracture are then illustrated. In this study, an independent zero-length element considering with nonlinear and hysteretic connection stiffness was adopted for representing the rigidity of a steel beam–column connection. The nonlinear response results are compared with those of existing studies to verify the accuracy of the proposed numerical procedure. Finally an analysis framework, which was proposed in this study, was used for the collapse analysis of a Vogel six-story semi-rigid frame under dynamic loads.

2. Finite particle method

2.1. Structural discretization

The FPM can be used to model the analyzed domain consisting of finite particles. The structural mass is assumed to be represented by each particle. Particles in the structure are connected by elements, which have no mass. Therefore, they are in static equilibrium. The deformation of elements can represent the force relationship and position variations between the particles connected to them. According to these assumptions, an analysis domain can be modeled using a set of particles and connected elements, as shown in Fig. 1.

2.2. Motion equation

The motion equation of an arbitrary particle α follows Newton's second law,

$$\mathbf{M}_\alpha \ddot{\mathbf{d}}_\alpha = \mathbf{F}_\alpha^{ext} - \mathbf{F}_\alpha^{int} - \mathbf{F}_\alpha^{dmp}. \quad (1)$$

where $\mathbf{M}_\alpha$ is the mass matrix, $\ddot{\mathbf{d}}_\alpha$ is the acceleration vector, and $\mathbf{F}_\alpha^{ext}$ and $\mathbf{F}_\alpha^{int}$are the external and internal force vector of particle α, respectively. The parameter $\mathbf{F}_\alpha^{int}$ equals the summation of the internal nodal forces exerted by the elements connected to the particle α. The formulations for internal forces of the elements are derived later. The parameter $\mathbf{F}_\alpha^{dmp}$ is the damping force vector given by

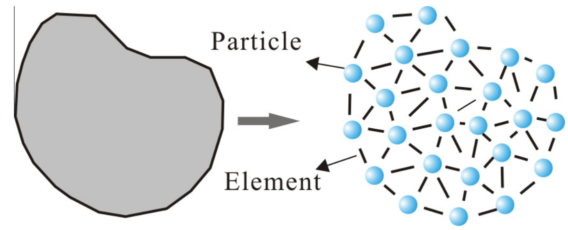

Fig. 1. FPM model of an analysis domain.

$\mathbf{F}_\alpha^{dmp} = \mu \mathbf{M}_\alpha \dot{\mathbf{d}}_\alpha$, where μ is the damping factor and has the same definition as it does in the dynamic relaxation method [24].

Because we focused on planar structures in this study, each particle was considered to have two translational degrees and one rotational degree of freedom. The discrete model of a planar frame is shown in Fig. 2. The motion equations for an arbitrary particle α of the planar frame are

$$\begin{bmatrix} m_\alpha & 0 & 0 \\ 0 & m_\alpha & 0 \\ 0 & 0 & I_\alpha \end{bmatrix} \frac{d^2}{dt^2} \begin{bmatrix} d_x \\ d_y \\ d_\theta \end{bmatrix}_\alpha = \begin{bmatrix} f_x^{ext} \\ f_y^{ext} \\ M^{ext} \end{bmatrix}_\alpha - \begin{bmatrix} f_x^{int} \\ f_y^{int} \\ M^{int} \end{bmatrix}_\alpha - \begin{bmatrix} f_x^{dmp} \\ f_y^{dmp} \\ M^{dmp} \end{bmatrix}_\alpha , \quad (2)$$

where m_α and I_α are the mass and moment of inertia lumped at particle α, respectively. Furthermore, we have $m_\alpha = \frac{1}{2}\sum_{i=1}^{n} m_i^l$, where n is the number of the elements connected to particle α, and m_i^l is the mass of the ith element connected to particle α. We also have $I_\alpha = \frac{1}{2}\sum_{i=1}^{n} I_i^l = \frac{1}{2}\sum_{i=1}^{n} m_i^l (r_i^l)^2$, where r_i^l is the radius of gyration of the ith element connected to particle α.

Because every particle in Eq. (1) is considered to be in dynamic equilibrium under the internal and external force, the static and dynamic analysis can be combined into a single procedure. Numerous methods can be employed to find the solution for Eq. (1). To avoid an iterative solution procedure, explicit time integration was suggested in this study. If a simple central difference is adopted, the velocity and acceleration can be approximated as

$$\dot{\mathbf{d}}_n = \frac{1}{2\Delta t}(\mathbf{d}_{n+1} - \mathbf{d}_{n-1}), \quad (3)$$

$$\ddot{\mathbf{d}}_n = \frac{1}{\Delta t^2}(\mathbf{d}_{n+1} - 2\mathbf{d}_n + \mathbf{d}_{n-1}), \quad (4)$$

where $\mathbf{d}_{n+1}$, $\mathbf{d}_n$, and $\mathbf{d}_{n-1}$ are the displacements of an arbitrary particle at steps $n+1$, n, and $n-1$, respectively, and Δt is a constant time increment. Substituting Eqs. (3) and (4) into Eq. (1) yields

$$\mathbf{d}_{n+1} = \left(\frac{2}{2+\mu\Delta t}\right)\frac{\Delta t^2}{m_\alpha}\left(\mathbf{F}_n^{ext} - \mathbf{F}_n^{int}\right) + \left(\frac{4}{2+\mu\Delta t}\right)\mathbf{d}_n - \left(\frac{2-\mu\Delta t}{2+\mu\Delta t}\right)\mathbf{d}_{n-1}. \quad (5)$$

This equation presents a simple and explicit formula that can be used for determining displacements of structures.

2.3. FPM for modeling geometric nonlinearity

Structural geometric nonlinearities include rigid body motion, large rotation and large deformation. It is crucial to remove the rigid body motion and rotation from the structural displacement. In the FPM, fictitious motion (including fictitious reverse motion and fictitious forward motion) is used to address geometric nonlinear problems in calculating the internal force of the element [19].

The fictitious motion is now briefly explained using a 2D beam element. Element 12 moves from 12 (at time t_a) to 1′2′ (at time t_b) in a time interval Δt. Because the internal forces are related only to