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Progressive collapse analysis of steel structures under fire conditions

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ABSTRACT

In this paper a robust static-dynamic procedure has been developed. The development extends the capability of the *Vulcan* software to model the dynamic and static behaviour of steel buildings during both local and global progressive collapse of the structures under fire conditions. The explicit integration method was adopted in the dynamic procedure. This model can be utilized to allow a structural analysis to continue beyond the temporary instabilities which would cause singularities in the full static analyses. The automatic switch between static and dynamic analysis makes the *Vulcan* a powerful tool to investigate the mechanism of the progressive collapse of the structures generated by the local failure of components. The procedure was validated against several practical cases. Some preliminary studies of the collapse mechanism of steel frame due to columns' failure under fire conditions are also presented. It is concluded that for un-braced frame the lower loading ratio and bigger beam section can give higher failure temperature in which the global structural collapse happens. However, the localised collapse of the frame with the higher loading ratio and smaller beam section can more easily be generated. The bracing system is helpful to prevent the frame from progressive collapse. The higher lateral stiffness of the frame can generate the smaller vertical deformation of the failed column at the re-stable position. However, the global failure temperature of the frame is not sensitive to the lateral stiffness of the frame.

1. Introduction

Structural engineers have a responsibility for incorporating fire safety into their building designs in order to minimize loss of life and property. The collapse of the twin towers of the World Trade Centre in New York was a reminder of the potential of fire to cause devastating failures of high-rise buildings by initiating progressive collapse. At present, steel structures have been widely used in the multi-story buildings because they are ideally suited to the current drive for improved construction efficiency as labour costs increase. However, the material properties of steel reduce significantly at elevated temperatures. For example, at 700 °C the strength of steel is only 23% of ambient-temperature strength. At 800 °C this has reduced to 11% and at 900 °C to 6%. Therefore, fire resistance design of steel buildings is a major concern to the structural engineers.

Currently, for structural fire engineering design, there is a trend that more designers will adopt the performance-based design approach. That means structures are treated integrally in structural fire safety design. For last two decades, extensive research has been carried out on the behaviour of steel-framed buildings under fire conditions. The Cardington full-scale fire tests [1] demonstrate that the real behaviour of structural elements can be very different from that indicated by standard furnace tests. In real buildings structural elements form part of a continuous assembly, and building fires often remain localised, with the fire-affected structure receiving significant restraint from cooler areas surrounding it. If such interactions are to be used by designers in specifying fire protection strategies as part of a performance-based structural design approach, then this cannot practically be based on large-scale testing because of the extremely high implicit costs. It is therefore becoming increasingly important that software models be developed to enable the behaviour of such structures to be predicted with sufficient accuracy under fire conditions. In recent years many researchers have developed numerical models to simulate the behaviour of steel or steel-composite frame in fire. For example, Wang and Moore [2] built a three dimensional model of a steel frame with semi-rigid connection to study the structural behaviour in fire. A computer program *Vulcan* has been developed at the University of Sheffield for three-dimensional modelling of steel, steel-framed composite and reinforced concrete buildings in fire [3–7]. The computer program FEMFAN from the Fire Engineering Research Group at Nanyang Technological University has been used by Tan et al. [8–11], to study the behaviour of a number of steel frames under fire conditions. Franssen et al. [12] developed a computer program SAFIR, which was used by many researchers [13–15]. Also a number of researchers [16–24] used commercial FEA software ABAQUS to carry out the structural analysis of steel

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frames at elevated temperatures. The most of above mentioned analyses are based on static analysis. It is clear that static analysis is computational effective for modelling structural behaviour in fire in which the loading time is longer (from 0.5 to 4 h). However, a shortcoming of static analysis is that the analyses would be terminated by numerical singularity or structural instability due to any localised members' failure.

Progressive collapse occurs when an initial local failure spreads from element to element, eventually resulting in collapse of a disproportionately large or entire part of a structure. Tan and Astaneh-Asl [25] experimentally studied the effective tying of steel structure subject to failure of key members and proposed a method to prevent progressive collapse using steel cables. Izzuddin et al. [26,27] investigated the progressive collapse of multi-storey composite buildings modelled by a two-dimensional model. Liew [28] built a mix-element model to study three dimensional steel frames subject to blast load and fire attack. The model is capable of capturing detailed behaviour of member and frame instability associated with the effects of high-strain rate and fire temperature. Lien et al. [29] proposed the vector form intrinsic finite element analysis of nonlinear behaviour of steel frame. They studied the behaviour of steel frame under fire induced by earthquake and concluded that the deformation of structure is significantly affected by the aftershock, fire and fracture of structural element.

The robustness of structure is the ability of the structure to prevent from disproportional failure after the local damage arisen by accidental actions. Hence, in order to assess of the robustness of structure in fire conditions, it is necessary to make sure that the analysis can go further after local instability taking place. Some researchers have tried to overcome this shortcoming of static analysis by carrying out full dynamic analysis for the whole duration of fire. Because the time of fire loading is relative long, hence the computation is very expansive. Therefore, the main objective of this paper is to develop a robust simplified numerical procedure in which the whole behaviour of a load-controlled structure can be modelled effectively. The model developed combines the static and dynamic analysis together to make full use of their advantages. Static analysis can be used to trace the behaviour of the structures at elevated temperature until the instability happened. After the instability of the analyses is identified, the dynamic procedure will be activated to continue the analysis. In this paper, an explicit dynamic procedure has been developed to allow modelling of the collapse of structural frames in fire. The model developed can be used to overcome the instabilities encountered in previous static analyses, and any re-stabilization of the frame at high deflections can be identified. After the re-stabilization of the frame gained the procedure will switch to static analysis again. The procedure developed was comprehensively validated. A series of parametric studies was conducted to investigate the mechanism of progressive collapse of planar steel frame due to the individual column failure.

2. Non-linear procedure

2.1. Dynamic Procedure

The general equation of a body motion can be expressed as:

$$\boldsymbol{M}\ddot{\boldsymbol{u}} + \boldsymbol{C}\dot{\boldsymbol{u}} + \boldsymbol{F}(\boldsymbol{u}) = \boldsymbol{Q}(t) \tag{1}$$

where $\boldsymbol{M}$ is the mass matrix, $\boldsymbol{C}$ is damping matrix, $\boldsymbol{F}(\boldsymbol{u})$ is the internal force vector and $\boldsymbol{Q}(t)$ is external force vector; t, $\mathbf{u}$, $\dot{\mathbf{u}}$ and $\ddot{\mathbf{u}}$ are time, displacement, velocity and acceleration vectors, respectively.

In order to solve Eq. (1) the direct-integration dynamic procedure provides two general operators: the implicit integration and explicit integration methods. In implicit dynamic analysis the integration operator matrix must be inverted and a set of nonlinear equilibrium equations must be solved at each time increment. But, for explicit one, no global mass or stiffness matrices need to be formed and inverted because displacements and velocities are calculated in terms of quantities that are known at the beginning of a time increment, thus, the calculation at each increment is relatively inexpensive compared to an implicit integration scheme.

Since implicit dynamic procedure requires forming and inverse the global stiffness matrix, hence, more disk space and memory are needed compared to explicit dynamic. Thus, for large scale problem explicit dynamic will be more effective than implicit one. Moreover, for problems with high nonlinearity or material complexity, the implicit dynamic would have difficulty to get a converged solution, resulting in either a large number of iterations needed or numerical failure of the analysis. Since the high nonlinearity due to material degradation, failure of members and the local and global instability presented in the collapse of the structural frame, in this research explicit method is adopted as integration method for dynamic analysis.

2.2. Time integration

In the developed explicit dynamic procedure, central difference integration is used to integrate the equation of motion explicitly through the time, using the kinematic conditions at the current increment i to calculate the kinematic conditions at the next increment, $i+1$. That is,

$$\dot{u}^n_{i+1/2} = \dot{u}^n_{i+1/2} + \Delta t_i\ \ddot{\boldsymbol{u}}^n_i \tag{2}$$

$$u^n_{i+1} = u^n_i + \Delta t_{i+1/2}\dot{\boldsymbol{u}}^n_{i+1/2} \tag{3}$$

where u^n_i and u^n_i are the displacement and velocity of degrees of freedom (DOF) n at ith time step, Δt_i is the time step and the subscript i refers to the current increment number of dynamic steps, $\Delta t_{i+1/2} = (\Delta t_i + \Delta t_{i+1})/2$. The key to the computational efficiency of the explicit procedure is the use of diagonal elements of mass matrices because the acceleration at the beginning of the increment is computed by:

$$\ddot{u}^n_i = (\boldsymbol{M}^n)^{-1}\left(\boldsymbol{Q}^n_i - \boldsymbol{F}^n_i - \boldsymbol{D}^n_i\right) \tag{4}$$

where, $\ddot{u}^n_i$ is the acceleration of DOF n at ith time step, $M^{\,n}$ is the mass of DOF n, Q^n_i is the applied load, and F^n_i and D^n_i are the internal and damping force vectors, respectively. In this procedure the time increments must be quite small so that the accelerations are nearly constant during an increment. Since the time increments are small, analyses typically require many thousands of increments. Fortunately, each increment is computationally inexpensive because there are no simultaneous equations needed to be solved. Table 1 gives a summary and flowchart of the explicit dynamics algorithm developed.

2.3. Mass matrix

A robust beam-column element has been developed in *Vulcan* [4]. The cross section of the beam column is divided into a matrix of segments and each segment may have different material, temperature, and mechanical properties. For beam-column element in *Vulcan* (see Fig. 1), the configuration of the beam is characterized using global coordinate (x–y–z) and a local coordinate (x'–y'–z') which is located at the neutral axis of the beam. In this case, an effective way to form lumped mass matrix is to measure the translational displacements in global coordinates (x–y–z), but to measure the angular velocity referenced to the natural coordinate.

The motion of the finite element model is described by the displacements $u^{n,j}$, velocities $\dot{u}^{n,j}$ and accelerations $\ddot{u}^{n,j}$ of the node referenced to the global co-ordinate system ($j = x, y, z$) and n is the number of nodes). The rotational motion of the node is de-