Fire-Induced Progressive Collapse Analyses of High-Rise Towers

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Abstract

Isobe and his collaborators are seeking for the true cause of the total collapse of the WTC towers back in 2001 by conducting several numerical simulations, including the full model aircraft impact analysis of the WTC tower. Isobe suggested in the past papers that the spring-back phenomenon due to rapid unloading caused by aircraft impact possibly have destructed member joints in the core structure, which might have caused the towers to become brittle and unstable. However, it does not give answers to the total collapse of WTC tower 7, which collapsed, reportedly, only by fire. In this study, the investigation is made from a different point of view and the influence of fire and heat is taken into account. The main objective of this study is to conduct some fire-induced collapse analyses to investigate how fire patterns, structural weakness of member joints, and outrigger trusses on roof tops give influence on the collapse behaviors of high-rise towers. The analytical results obtained using the ASI-Gauss finite element code show a clear difference between each fire pattern, between the models with strong and weak member-joint strengths, and between the models with and without the outrigger trusses. In general, the strongly designed models withstand the total collapse, whereas the weak models initiate their collapse when the temperature reaches to the highest, and ends, eventually in a total collapse. The models with outrigger trusses tend to withstand longer in time by catenary action only if their load paths are protected.

Keywords: progressive collapse, high-rise towers, 9/11, ASI-Gauss technique, finite element analysis.

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1.0 Introduction

Isobe and his collaborators are seeking for the true cause of the total collapse of the WTC towers back in 2001 by conducting several numerical simulations. Isobe and Sasaki [1] first performed the full model aircraft impact analysis of the WTC tower and suggested regarding the obtained results that the spring-back phenomenon due to rapid unloading caused by aircraft impact possibly have destructed member joints in the core structure, which might have caused the towers to become brittle and unstable. However, it does not give answers to the total collapse of WTC tower 7, which collapsed, reportedly, only by fire. In this study, the investigation is made from a different point of view and the influence of fire and heat is taken into account. The main objective of this study is to conduct some fire-induced collapse analyses to investigate how fire patterns, structural weakness of member joints, and outrigger trusses on roof tops give influence on the collapse behaviors of high-rise towers. The analyses are carried out using the ASI-Gauss finite element code which provides higher computational efficiency than the conventional code. It can also cope with dynamic behavior with strong nonlinearities including phenomena such as member fracture. A general outline of the ASI-Gauss technique is described and several numerical simulations on the fire-induced progressive collapse behaviors of high-rise towers are discussed in this paper.

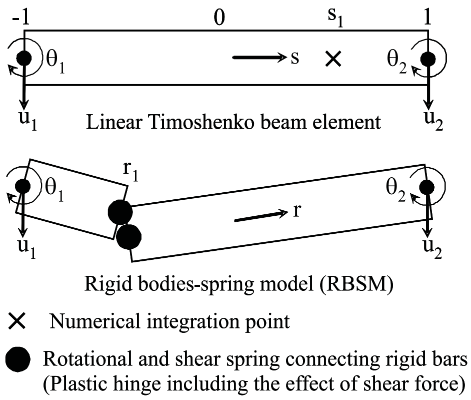


Figure 1. Linear Timoshenko beam element and its physical equivalent.

2.0 Numerical Methods

2.1 ASI-Gauss Technique

In this paper, the numerical analyses are carried out using a finite element code based upon the ASI-Gauss technique [1,2] with the linear Timoshenko beam elements. Figure 1 shows a linear Timoshenko beam element and its physical equivalence to the RBSM. As shown in the figure, the relationship between the locations of the numerical integration point and the stress evaluation point where a plastic hinge is formed is expressed as [3]:

$$r = -s \tag{1}$$

Here, s is the location of the numerical integration point and r is the location where stresses and strains are evaluated. s and r are nondimensional quantities that take values between -1 and 1.

In the ASI-Gauss technique, the numerical integration point is shifted adaptively according to Eq. (1) when a fully plastic section is formed within an element, to form a plastic hinge exactly at that section. By doing so, the plastic behavior of the element is simulated appropriately, and the converged solution is achieved with minimum number of elements per member. When the plastic hinge is unloaded, the corresponding numerical integration point is shifted back to its normal position. Here, the normal position means the location where the numerical integration point is placed when the element acts elastically. Two consecutive elements forming a member are considered as a subset, as shown in Fig. 2, and the numerical integration points of an elastically deformed member are placed such that the stress evaluation points coincide with the Gaussian integration points of the member. This means that stresses and strains are evaluated at the Gaussian integration points of elastically deformed members. Gaussian integration points are optimal for two-point integration and the accuracy of bending deformation is mathematically guaranteed [4]. The scheme takes advantage of two-point integration while using one-point integration in actual calculations. More details on the code can be found in previously published papers [1,2].

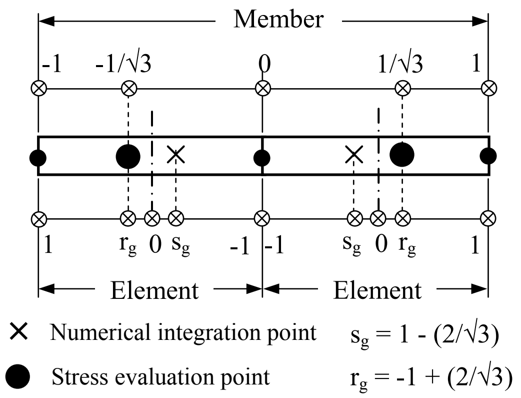


Figure 2. Locations of numerical integration and stress evaluation points.

2.2 Consideration of Member Joint Strength and Fracture

We define the joint strength ratio C_M which indicates the ratio of member joint strength against the member strength itself. The value is directly used in the function as follows, to explicitly express the weakness of member joint strength:

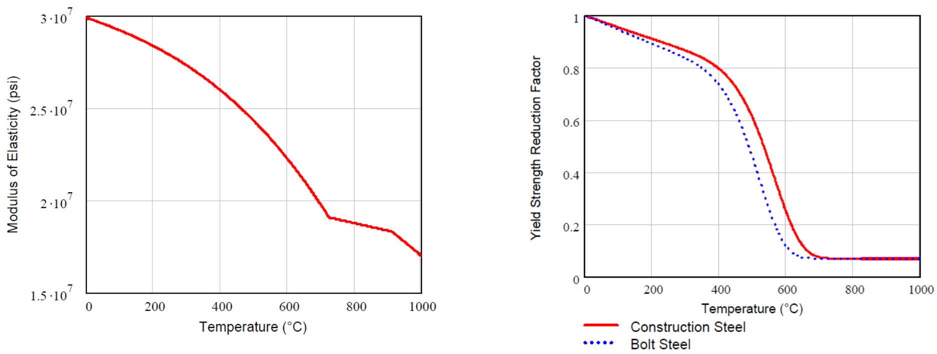
$$f = \left(\frac{M_x}{C_M M_{x0}}\right)^2 + \left(\frac{M_y}{C_M M_{y0}}\right)^2 + \left(\frac{N}{N_0}\right)^2 - 1 = f_y - 1 = 0 \tag{2}$$

Here, f_y is the yield function, and M_x , M_y and N are the bending moments around the x- and y-axes and axial force, respectively. Those with the subscript 0 are values that result in a fully plastic section in an element when they act on a cross section independently. The ratio value C_M is assumed on the basis of the fact that the member joint strengths of typical core columns in the WTC towers were about 20 to 30 % of those of the members [5]. The values used in this paper are C_M =1.0 for strong member-joint model and C_M = 0.3 for weak member-joint model.

Structurally discontinuous problems also become easily handled using the ASI-Gauss technique by shifting the numerical integration point of the linear Timoshenko beam element to an appropriate position and by releasing the resultant forces in the element simultaneously. Here, the fracture condition of member joints are considered by examining the strain values in a member, as follows:

$$\left|\frac{\kappa_x}{\kappa_{fx}}\right| - 1 \geq 0 \text{ or } \left|\frac{\kappa_y}{\kappa_{fy}}\right| - 1 \geq 0 \text{ or } \left(\frac{\epsilon_z}{\epsilon_{fz}}\right) - 1 \geq 0 \text{ or } \left|\frac{\gamma_{xz}}{\gamma_{fxz}}\right| - 1 \geq 0 \text{ or } \left|\frac{\gamma_{yz}}{\gamma_{fyz}}\right| - 1 \geq 0 \tag{3}$$

where κ_x, κ_y, ε_z, γ_{xz}, and γ_{yz} are the bending strains around the x- and y-axes, the axial tensile strain and the shear strains for the x- and y-axes, respectively. The critical values for these strains are indicated by the subscript *f*. We used the critical strain values that are actually obtained from some experiments on member joints concerning high strength joint bolts [6]. Elemental contact is considered in the code by introducing gap elements between pairs of elements determined to be in contact by geometrical relations.



(a) Elastic modulus (b) Yield strength

Figure 3. Strength reduction curves of steel due to elevated temperatures.

2.3 Consideration of Strength Reduction due to Elevated Temperature

Thermal stress effect is applied to the above mentioned finite element code. The reduction curves of elastic modulus and yield strength of steel due to elevated temperatures [7], as shown in Figs. 3(a) and 3(b), are also adopted. The elastic modulus of the raw columns without thermal insulation is reduced to 60 % of the original value, and the yield strength to 10 % of the original strength, at 700 ℃, which is a natural temperature in normal fire. We assumed the temperature to rise up to 700 ℃ in 7 minutes as shown in Fig. 4. A time increment control is also applied to reduce calculation time in the seamless analysis from static to dynamic phenomena.

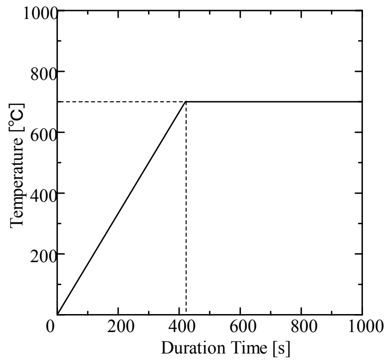


Figure 4. Assumed time history of temperature.

3.0 Numerical Model

The numerical model used in the fire-induced collapse analyses is a 25-story 3-span tower as shown in Fig. 5. An outrigger truss system is placed on the roof top of the model and the influence of the system against structural vulnerability of the tower is verified in the analyses. The tower is modeled with 2056 linear Timoshenko beam elements and contains 1488 nodes and 8928 degrees of freedom. For example, the sectional sizes of the columns used on the 1st floor are □-430×430×13×13. We provided two cases of member joint strengths as mentioned; strong type (C_M = 1.0) and weak type (C_M = 0.3), each with and without outrigger truss system, and assumed two fire patterns; middle three blocks and outer three blocks from 17th to 19th floors as shown in Fig. 5.

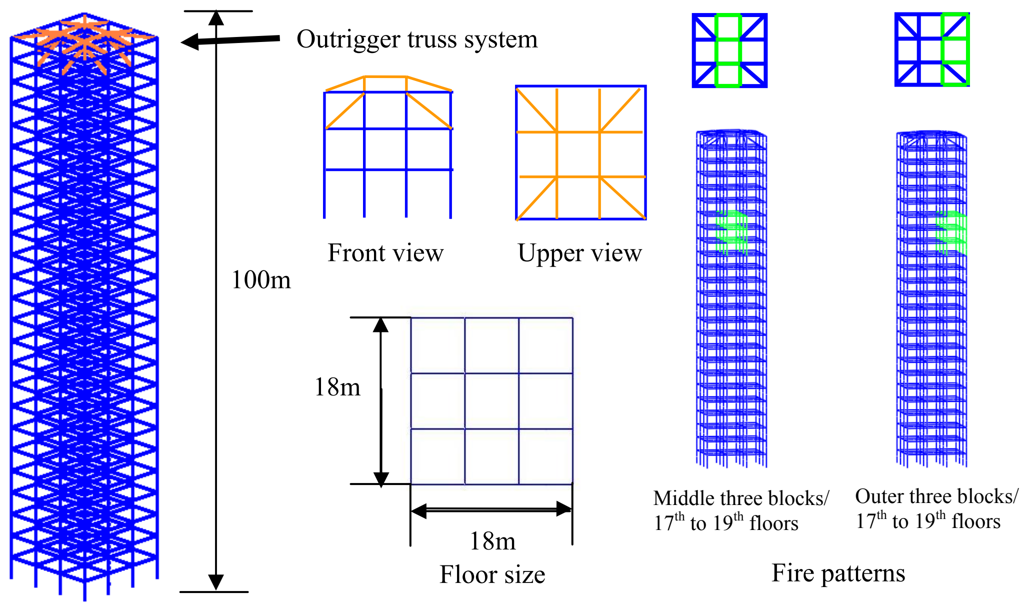


Figure 5. 25-story 3-span tower model.

4.0 Numerical Results

Two of the numerical results are shown in Figs. 6 and 7. Both are performed with the weak member-joint type ($C_M = 0.3$) with outrigger truss systems placed on the roof tops. Figure 6 shows the result with the middle three blocks on fire and Fig. 7 with the outer three blocks on fire, respectively. Both cases initiate to collapse in different times and continue to collapse progressively in different modes. The collapse initiation time in the former case is longer than in the latter case and the upper structure drops vertically to the ground. The collapse initiation time is short in the latter case and the upper structure slants to the right where the blocks are on fire.

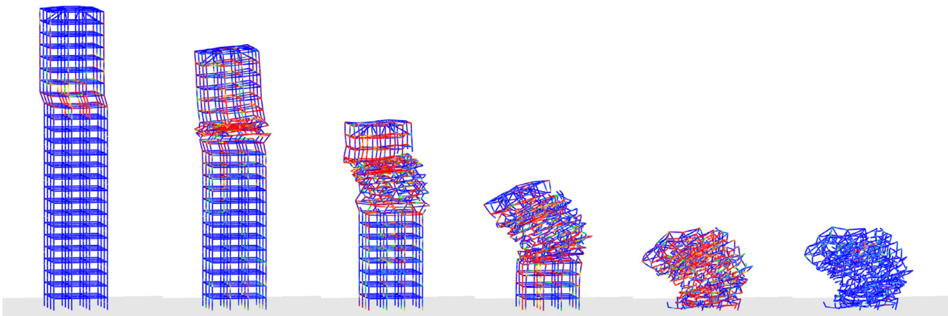


Figure 6. Collapse mode ($C_M = 0.3$, with outrigger truss system, middle three blocks on fire).

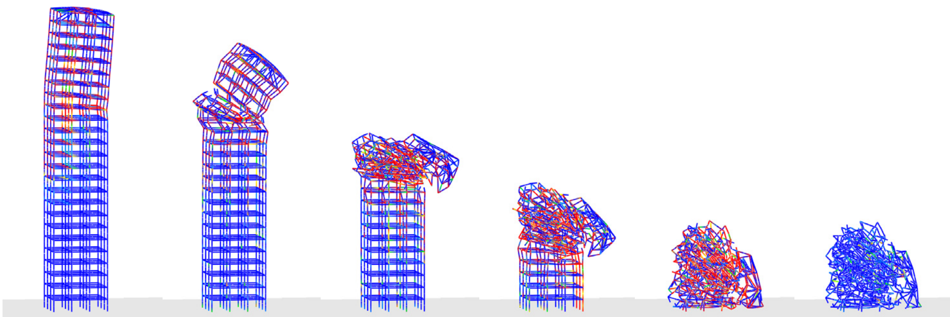


Figure 7. Collapse mode ($C_M = 0.3$, with outrigger truss system, outer three blocks on fire).

Table 1 shows the sum of all the results. Strong member-joint models did not collapse regardless of the arrangement of outrigger truss systems or the fire patterns. The effect of outrigger truss systems can be observed in the collapse initiation times of the weak member-joint models. When the middle three blocks are on fire, the model with an outrigger truss system initiates its collapse earlier than the model without it. The load paths to/from the outrigger truss system are completely intercepted and the system behaves only as an extra mass. The good effect becomes evident when the outer blocks are on fire. The model tends to withstand longer in time by the catenary action of the system. It may well be said that the former case with middle three blocks on fire is stable, and the collapse initiation time tends to delay since the fire pattern is symmetric. However, the outrigger truss system has reverse effect in this case (151s shortening the initiation time) since the load paths are not protected. The latter case with outer three blocks on fire is comparatively unstable than the former case, and the collapse initiation time tends to shorten since the fire pattern is asymmetric. However, the outrigger truss system helps largely to delay the collapse initiation time (121s delayed in time), since the load paths are partially protected in this case.

Table 1: Influences of member-joint strengths, fire patterns and outrigger truss systems

<table>
<tr><th>C_M</th><th>Fire patterns</th><th>Outrigger truss systems</th><th>Collapse initiation time [s]</th></tr>
<tr><td rowspan="4">C_M =0.3</td><td rowspan="2">Middle three blocks/ 17th to 19th floors</td><td>None</td><td>997</td></tr>
<tr><td>Yes</td><td>846</td></tr>
<tr><td rowspan="2">Outer three blocks/ 17th to 19th floors</td><td>None</td><td>577</td></tr>
<tr><td>Yes</td><td>698</td></tr>
<tr><td rowspan="4">C_M =1.0</td><td rowspan="2">Middle three blocks/ 17th to 19th floors</td><td>None</td><td>—</td></tr>
<tr><td>Yes</td><td>—</td></tr>
<tr><td rowspan="2">Outer three blocks/ 17th to 19th floors</td><td>None</td><td>—</td></tr>
<tr><td>Yes</td><td>—</td></tr>
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5.0 Conclusion

The numerical results shown in this paper indicate that the member-joint strengths and fire patterns give large influences on collapse behaviors of high-rise buildings. Moreover, the outrigger truss systems on roof tops tend to help elongating the collapse initiation time, only if the load paths to/from the systems are protected.

6.0 References

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