2-D analysis of a building frame under gravity load and fire

Dat Duthinh[1]

Summary

A two-dimensional finite-element model is developed that provides some insight into the behavior and collapse of high-rise steel buildings with open web floor systems. For one prescribed temperature distribution that corresponds to a two-story, quarter-span fire, the diagonals of the heated trusses buckle inelastically, causing considerable sag in the fire floors. This behavior puts a high tension demand on the truss connections to the perimeter column, which remains at moderate temperatures in this model and does not experience buckling. Our analysis is based on temperatures and material properties that were selected for illustrative purposes. Therefore no claim is made as to its applicability to any specific structure.

Introduction

The objective of this paper is to present an approach to the analysis of a tall steel building frame similar to the analysis by Usmani et al. [1]. For the purpose of comparison with other published work, and to use one specific example to illustrate the analysis method, the data used in this paper are based on information from Reference 2. The choice of a 2-D model is justified by the need to perform simple but informative calculations covering a wide variety of conditions. We modeled the structural system independently of connection details. Connections are outside the scope of the analysis presented in this paper.

Structural model

The vertical plane considered in the model includes a column and five longitudinal floor trusses and slabs at about 4/5 of the height of the building being considered [2]. The column extends 22 m (72 ft) to a height of six floors, and both its upper and lower ends are pinned, with the upper end free to translate vertically. The upper chords of the floor trusses are simply supported at the (right) internal end, and connected to the perimeter column by hinges. In the actual structure, a double floor truss carries a tributary floor slab 2 m (80 in) wide and is in turn supported by two perimeter columns, whereas in our model, a single truss supported by a single column carries a 1 m (40 in) wide slab. The floor spans 18.3 m (60 ft) from the perimeter column to the core.

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[1] National Institute of Standards and Technology, Gaithersburg, MD. 20899, USA

The trusses, slabs, and column that supports them are simulated by three-node beam finite elements, capable of modeling a wide variety of cross sections, with a mesh density and number of integration points specified by the user. One particularly attractive feature of this element is its capability of supporting linear temperature gradients across its section and along its length.

Material properties

The various steels range in nominal yield strength from 248 MPa (36 ksi) in the floor trusses to 448 MPa (65 ksi) in the column [2]. They are all modeled by bilinear stress-strain curves, with a tangent modulus about 0.5 % of the elastic modulus. Steel properties above 1100°K are relatively scarce, but indicate a stabilization of yield and ultimate strengths between 1100°K and 1300°K.

The lightweight concrete slab is also modeled as a bilinear material, with compressive strength of 21 MPa or 3000 psi [2]. For simplicity, the top chord of the floor truss is assumed to act in a perfect composite way with the slab and allow the tensile strength at the bottom of the slab to be equal in magnitude to the compressive strength at the top. The simplification inherent in transforming the steel top chord into an equivalent concrete section disregards the differential thermal expansion between steel and concrete.

Loading

The floor slabs are acted upon by dead and live loads consistent with [2]. The column end loads are determined by a separate analysis of the damaged structure estimated from [2]. In addition, the column dead load is applied along its length.

The behavior of the structure is greatly influenced by thermal loads. This paper uses a strictly conventional fire. For comparison with [1], a single temperature distribution T represented by an exponential function of time t, with a reference temperature T_0 = 300°K, is used. The time rate of change of the temperature, represented by coefficient a = 0.005, depends, among other factors, on the location and intensity of the fire, and the quality of the insulation.

$$T(t) = T_0 + (T_{\max} - T_0)\,(1 - e^{-at}) \tag{1}$$

A two-floor fire, with maximum temperature $T_{\max}$ = 1273°K, heats the structure over the quarter-span closest to the perimeter column. Over that span, the slab of the second floor is uniformly heated, whereas the slabs of the floors below and above it have linear temperature gradients across their thickness, with the bottom of the lower slab and the top of the upper slab remaining at 300°K at all times. In the three-quarters of the span not directly under fire, the temperature decreases linearly from the maximum at quarter-span to normal room temperature at the core. Between the first and the third floors, the column temperature is also described by Eq. (1), with T_{max} = 400°K, whereas the rest of the column remains at 300°K at all times.

Results

Nonlinear, static, large deformation analysis accounting for P-delta effects and buckling was performed. The analysis proceeded in a number of load steps, the first corresponding to gravity loads at the start of the fire (normal room temperature). Subsequent steps occurred at 200 s intervals, with the maximum temperature attained, to within 1°K, at 1400 s. Results are shown in Figs. 1-3.

At room temperature, even under the severe load redistribution due to initial damage to the building, non-linear effects are negligible, i.e., the structure still behaves linearly. The maximum floor sag is 35 mm (1.4 in), causing the horizontal span to decrease and the column to pull in slightly. Approaching 200 s and a temperature of 915°K, the heated trusses begin to show distress, especially in the compressed diagonal and vertical members, which buckle inelastically. (The temperatures referred to in these results are the hottest temperatures in the structure at any given time.) At 200 s the maximum floor sag increases to 335 mm (13.2 in), and the column is pushed out (peak of 38 mm or 1.5 in) by the thermal expansion of the second floor. At that time the connection of the second floor to the perimeter column experiences its maximum compression of 125 kN (28 kips). Because the third floor slab has a thermal gradient with its top surface at room temperature, its lateral expansion is much smaller than for the second floor slab, and its sag is larger. The connection between the third floor slab and the column is always in tension (Fig. 2). As expected, the bottom floor slab, heated at the top and cool at the bottom, bows upward. As the temperature continues to rise, more of the second floor truss fails, and the increasing sag begins to pull the column in. The horizontal deflection of the column becomes positive (inward), and the connection force between the column and the second floor turns to tension. This inward movement of the column relieves the tension in the connection between the column and the third floor. Further temperature rise causes further weakening in the third floor truss, which eventually becomes active in pulling the column in. At the peak temperature of 1273°K, the maximum lateral deflection in the column (183 mm or 3.3 in) occurs at the third floor, inward, and the connection between the column and the third floor experiences a tension of 185 kN (41.6 kips). The connection forces supplied by the interior column to the floors are similar to that of the external column.

NIST analysis of connections is ongoing and will indicate whether this or other connections can supply the calculated demand, and if not, at what temperatures connection failures will occur. In this regard, the composite behavior of the concrete slab with the steel truss may become questionable at a temperature and strain level yet to be determined.

For comparison with Quintiere et al. [3], our results show that the truss diagonals buckle *inelastically*, and there is considerable reserve strength after the first diagonal buckles. At the highest temperatures analyzed, seven diagonals and vertical had buckled in each of the heated floors. This conclusion assumes that the various structural connections maintain their integrity throughout the fire.

Conclusions

A model is developed that provides some insight into the behavior and collapse of high-rise steel buildings with open web floor systems. For the particular temperature distribution we selected from among those assumed by Usmani et al. [1], the diagonals of the heated trusses buckle inelastically, causing considerable sag in the fire floors. This behavior puts a high tension demand on the truss connections to the perimeter column, which remains at moderate temperatures in this model and *does not experience buckling*. This is the major difference between our results and Usmani's, even though the heated trusses in our model are exposed to a much higher temperature and the column to a more severe load that reflects load redistribution in the damaged structure. One possible explanation for the difference is that failure modes may be very sensitive to material properties. Similar high tension demands are also imposed on the floor connections to the internal column

References

1 Usmani, A.S., Chung, Y.C. and Torero, J.L. (2003): "How did the WTC towers collapse: a new theory," *Fire Safety Journal*, vol. 38, no. 6, Oct. pp. 501-533.

2 FEMA (2002): "World Trade Center Building Performance Study", Federal Emergency Management Agency Report 403, Sep.

3 Quintiere, J.G., di Marzo, M. and Becker, R. (2002): "A suggested cause of the fire-induced collapse of the World Trade Towers," *Fire Safety Journal*, vol. 37, no. 7, Oct. pp. 707-716.

Acknowledgment

Discussion with Michael Riley produced useful insight on FEA. The guidance provided by project leader Emil Simiu is also gratefully acknowledged.

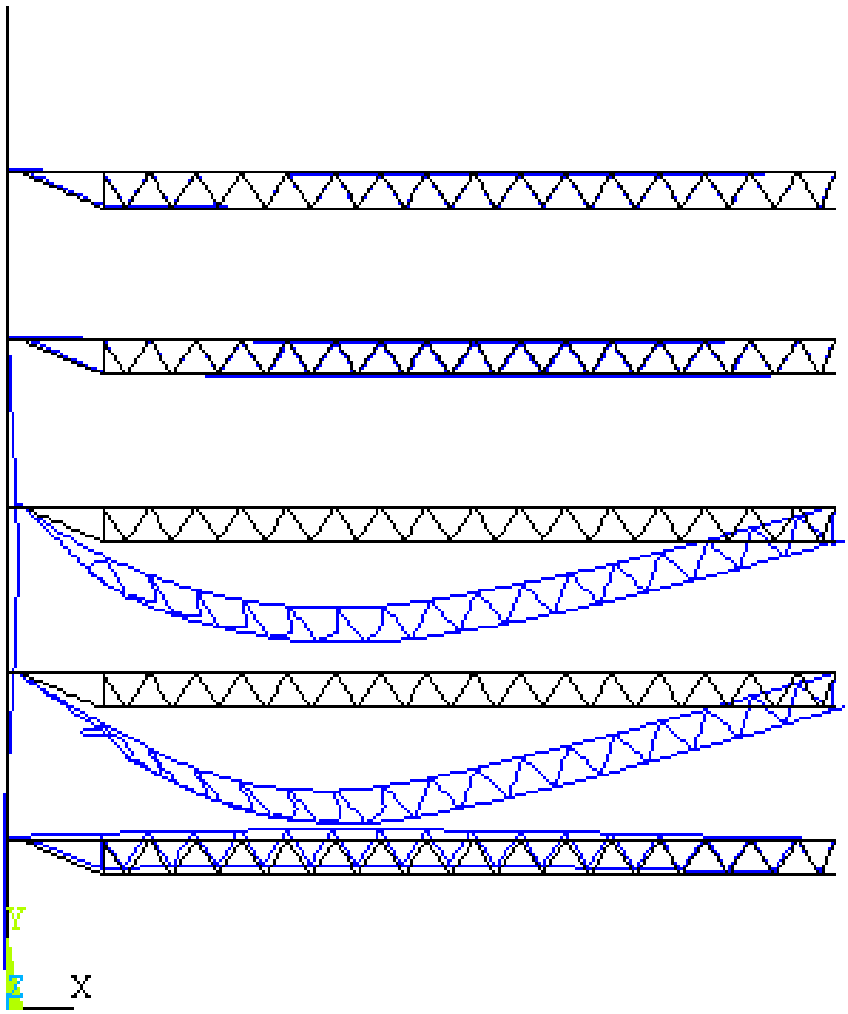

Fig. 1 Deformed shape at 1273°K

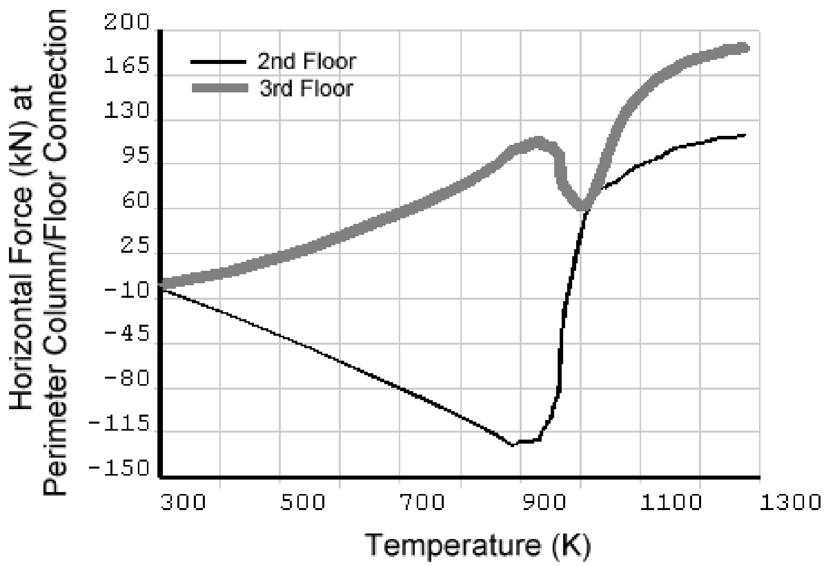

Fig. 2 Horizontal force (kN) between column and second and third floors vs. temperature (°K)

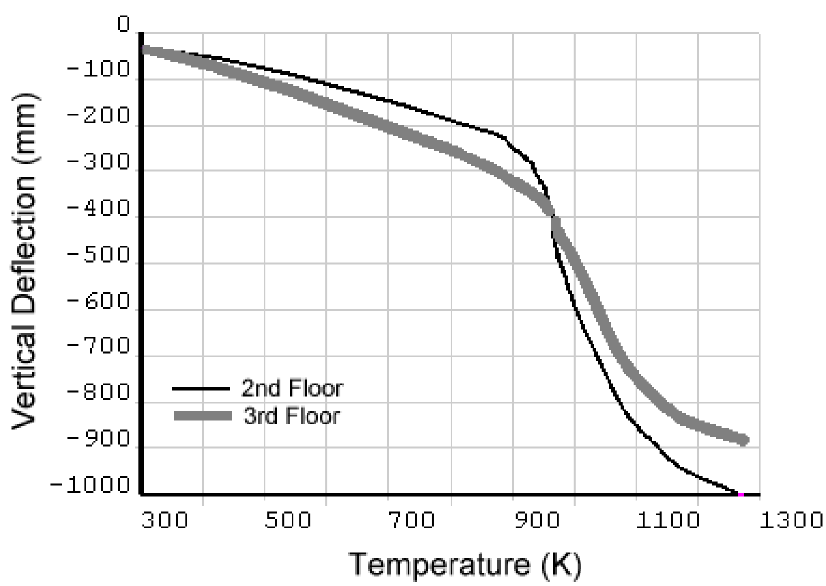

Fig. 3 Vertical deflections (mm) of second and third floors vs. temperature (°K)