TABLE OF CONTENTS - Appendix M

LIST OF FIGURES

Appendix M
INTERIM REPORT ON 2-D ANALYSIS OF THE WTC TOWERS UNDER GRAVITY LOAD AND FIRE

M.1 SUMMARY

A two-dimensional (2-D) finite element model is developed to provide insight and evaluate some aspects of a possible collapse sequence for the World Trade Center (WTC) towers. For a prescribed temperature distribution that corresponds to a two-story, quarter-span fire, and for a three-story fire derived from fire dynamics simulation, 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. Because neither the prescribed nor the derived temperature distributions are necessarily representative of the actual fire, and the material properties are approximate, further work is needed to evaluate the collapse sequence and develop findings regarding the actual event.

M.2 INTRODUCTION AND REVIEW

Within days of the collapse of the WTC towers on September 11, 2001, publications postulating the mechanism of the collapse began to circulate. A substantial effort was launched by the Federal Emergency Management Agency (FEMA) and the American Society of Civil Engineers (ASCE), culminating in a preliminary building performance study (McAllister 2002). Quintiere et al. calculated the elastic buckling strength of a single diagonal of a floor truss, assuming pinned end conditions, and suggested that the buckling of such thin members exposed to fire might have initiated the collapse (Quintiere et al. 2002). More recently, Usmani et al. performed a series of 2-D, nonlinear finite element analyses of a 12-story vertical frame that comprises a perimeter column and, at each floor, a truss and floor slab supported by the column and the tower core (Usmani et al. 2003). The temperature distribution in the steel and concrete members was characterized by an assumed time-dependent profile. Usmani et al. concluded that column instability caused by the loss of bracing normally provided by floors led to overall structural collapse (Usmani et al. 2003).

The objective of this report is to present a simplified analysis approach to evaluate some aspects of the collapse sequence of the WTC towers. The analysis is based on a 2-D model that is simple and can be easily used to evaluate a wide variety of conditions. The structural system was modeled independently of connection details. At this stage, connections are the object of a separate analysis that can draw on the results presented here concerning demand upon connections at various stages of fire development.

M.3 STRUCTURAL MODEL

The vertical plane considered in the model includes perimeter column 109 on the North face of WTC 1, and five longitudinal floor trusses and slabs (floors 94 to 98). The center of the airplane impact was at floor 96, and column 109 was the intact column closest to the edge of the initial damage zone (McAllister 2002). 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 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 supported by two perimeter columns, whereas in the present model a single truss supported by a single perimeter column carries a 1 m (40 in.) wide slab. The floor spans 18.3 m (60 ft) from the perimeter column to the core. The model is similar to that of Usmani et al. (2003), except it has fewer floors.

The principal reason for including only five floors in the analysis is to have the simplest model that will still capture salient features of the collapse of the towers. The fire applied to the model only heats two floors, and the remaining floors remain cool and provide lateral restraint to the perimeter column under study. Since the ends of the perimeter column in this simple model are hinged, the model ignores the rotational restraint supplied by the continuous column if additional floors are considered. Thus the short model is less stiff than a taller model (such as the 12-floor model developed by Usmani [2003] and would buckle sooner (or at a lower mechanical or thermal load), if global buckling should occur at all. As far as translational restraint is concerned, only a very small amount of lateral bracing can have a tremendous effect on the buckling strength (Winter 1958), and the cool floors one or two stories away from the fire can be replaced by a support that does not allow horizontal translation. One additional reason for including only five floors is to provide a guide for and allow comparisons with the results of a three-dimensional study, where several full floors are included. The size of the three-dimension model is a concern.

The trusses, slabs and the 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 these elements is the capability of supporting linear temperature gradients across their section and along their length.

M.4 MATERIAL PROPERTIES

The various steels range in nominal yield strength from 250 MPa (36 ksi) in the floor trusses to 450 MPa (65 ksi) in the column (McAllister 2002). They are all modeled by bilinear stress-strain curves, with a tangent modulus about 0.5 percent of the elastic modulus. Figures M–1 and M–2 show the steel properties for the temperature range used in the analysis. Usmani et al. (2003) used similar steel properties.

The lightweight concrete slab is also modeled as a bilinear, ductile material (Fig. M–3), with compressive strength of 20 MPa or 3,000 psi (McAllister 2002). The top chord of the floor truss is assumed to act in a perfectly 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. This choice of a simple, bilinear material overestimates the tensile capacity of the slab. As well, the simplification inherent in transforming the steel top chord into an equivalent concrete section disregards the differential thermal expansion between steel and concrete. A more accurate concrete model (currently being developed) may show slab failure or a smaller horizontal tension at the connection between floor and column than the present results.

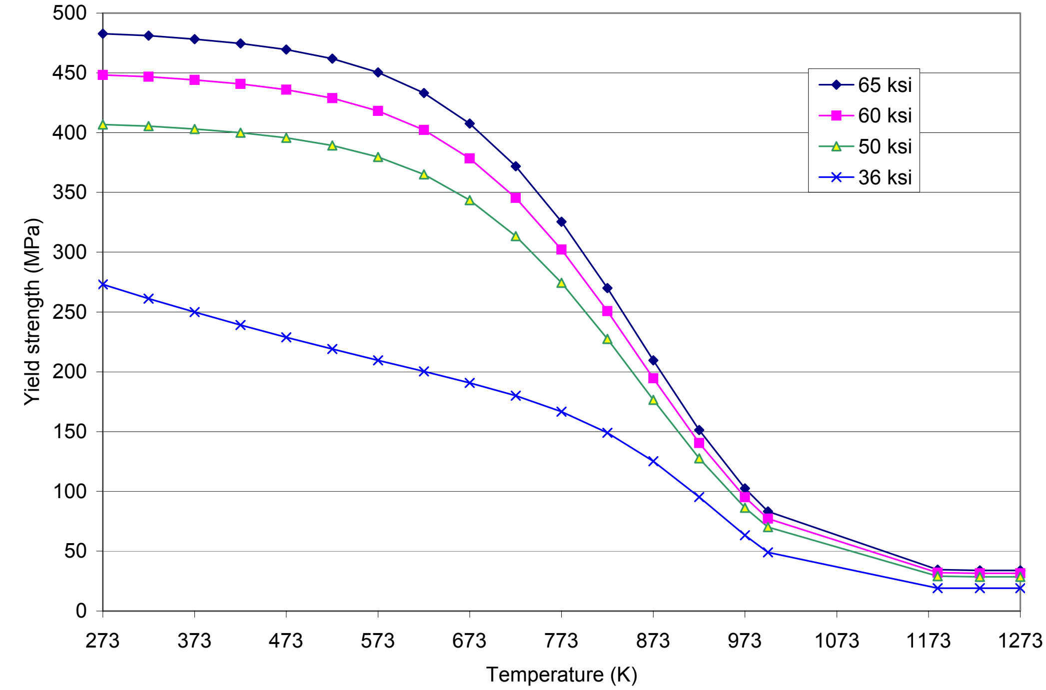

Figure M–1. Yield strength of steels used in model.

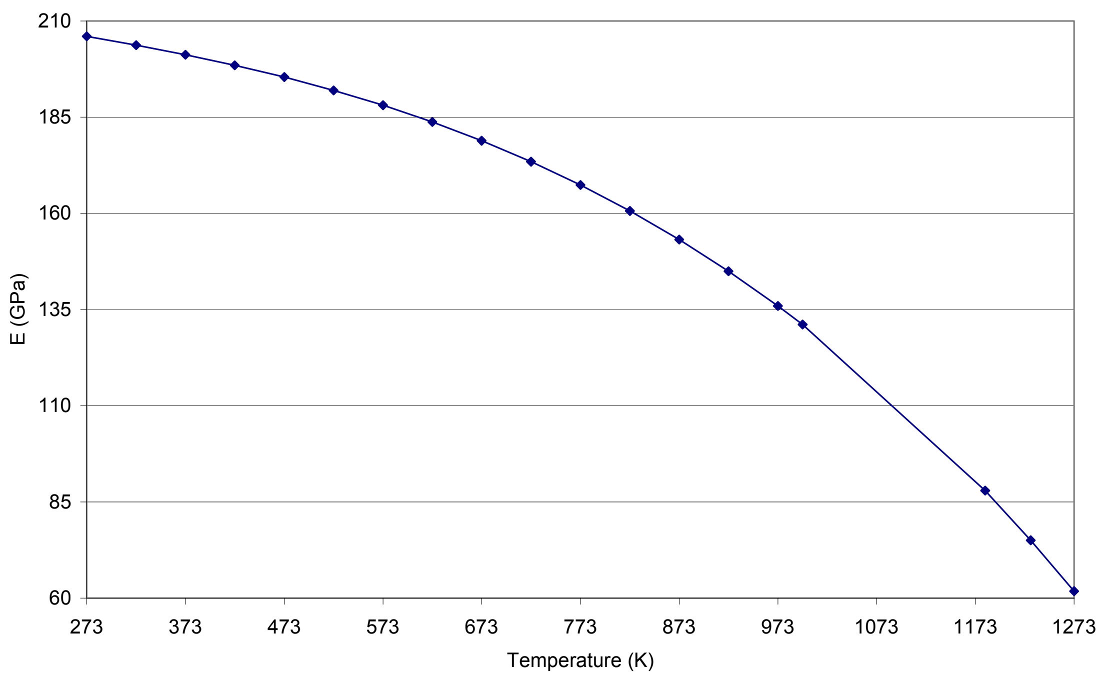

Figure M–2. Modulus of elasticity of steels used in model.

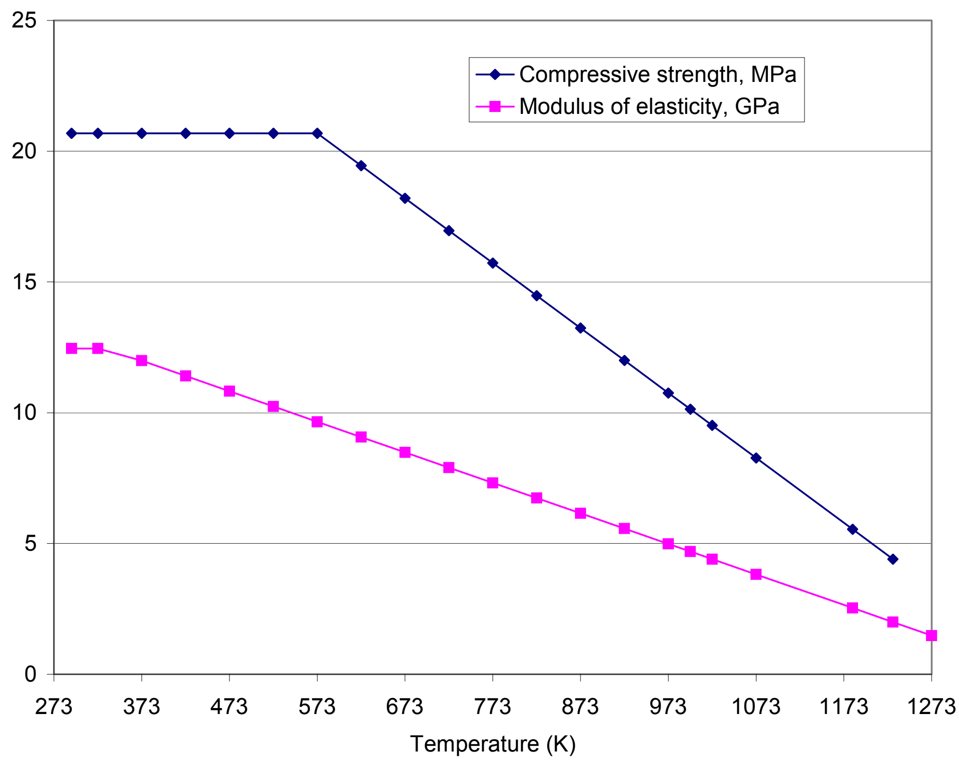

Figure M–3. Mechanical properties of concrete slab as modeled.

M.5 LOADING

The floor slabs are acted upon by a dead load of 3.3 kPa (70 psf) and a live load of 720 Pa (15 psf). The column load, determined by a linear, static finite element analysis of the global, damaged structure, includes the weight of the floors above and a surcharge due to load transfer from the columns damaged or missing after the airplane impact (Appendix D, Section D.2.4 of this report). The top of the column is loaded by a 1,100 kN (250 kip) axial compressive force and a 2,000 N·m (18 kip·in.) clockwise moment (compared to 540 kN or 120 kip, and 1,800 N·m or 16 kip·in. counter clockwise moment before damage). In addition, the column self-weight is applied along its length. For comparison, Usmani et al. (2003) used loading consistent with the FEMA report (McAllister 2002) and applied 40 percent of the gravity loads of the tributary floor strips above the model to the top of the perimeter column.

The behavior of the structure and its eventual collapse are greatly influenced by thermal loads. This report first performs an analysis based on a conventional fire, which provides a useful first approximation to the behavior of the building in fire. For comparison with the work of Usmani et al. (2003), 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} = 1,273 K, heats the structure on floors 95 and 96, over the quarter-span closest to the perimeter column. Over that span, the slab of floor 95 is uniformly heated, whereas the slabs of floors 94 and 96 have linear temperature gradients across their thickness, with the bottom of slab 94 and the top of slab 96 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 room temperature at the core. Between floors 94 and 96, 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.

The finite element model was used further by applying to it a second temperature distribution (Fig. M–4) that corresponds to a more realistic, physics-based fire, generated by NIST's Fire Dynamics Simulator in a manner consistent with initial conditions appropriate for the WTC towers following the aircraft impacts. The gas temperatures associated with the fire were used to calculate structural member temperatures and temperature gradients, assuming an insulation thickness of 19 mm (3/4 in.) for truss members and 36 mm (1.4 in.) for the perimeter column. The fire considered in this application is more widely spread than the conventional fire, and covers four floors, with the entire floor span heated. Floor 94 (lowest) remains unheated, and the column is only moderately heated. Linear temperature gradients are modeled across the column section and the slab thickness of heated floors. The peak temperature of 1,230 K is obtained at the end of 25 temperature load steps, each 200 s apart.

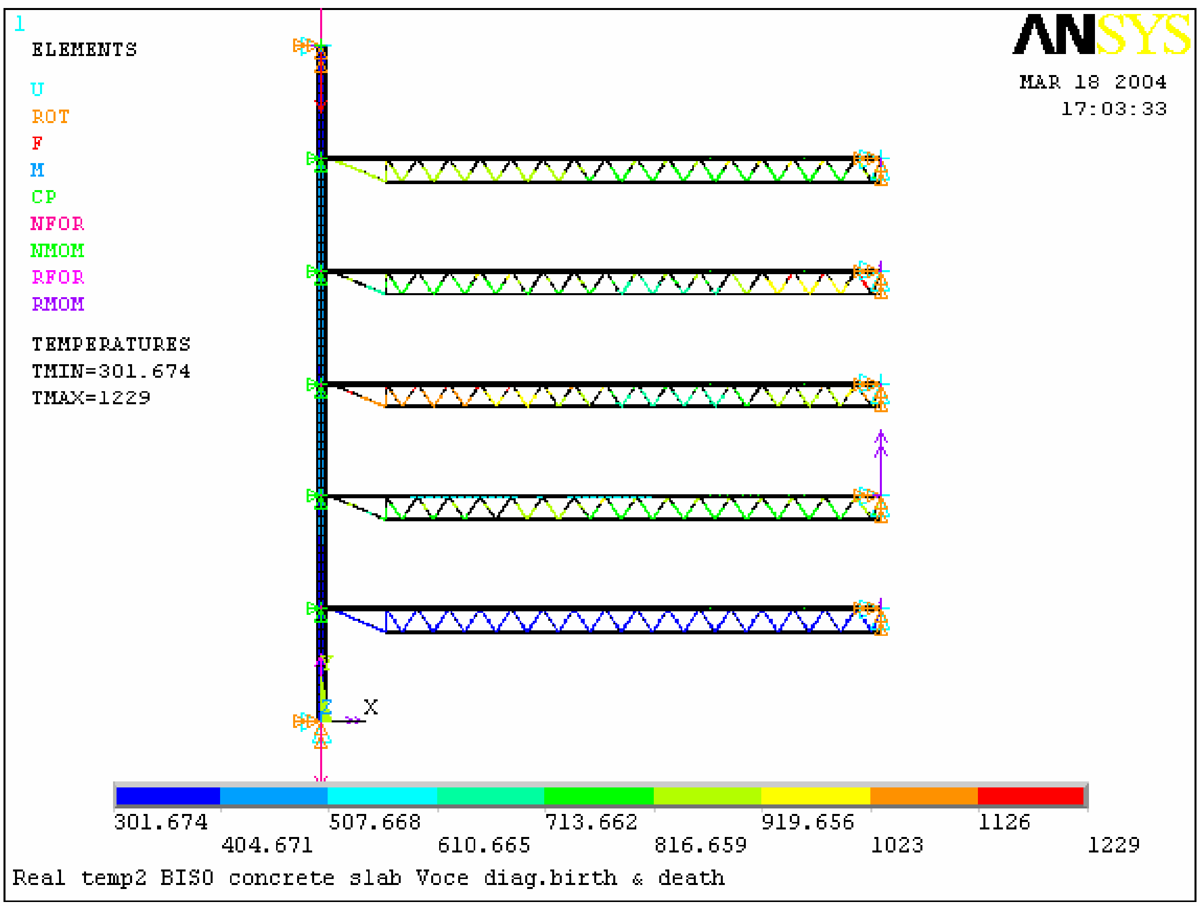

Figure M–4 Temperature distribution (K) of "real fire" scenario 2-04.

M.6 RESULTS

Nonlinear, static, large deformation analysis accounting for the magnification of flexural deflections due to axial load (P-delta effect) was performed. Member stiffness matrices are updated during the analysis to account for the P-delta effect, and when a member stiffness gets close to zero, excessive lateral deflection occurs and the member is considered to have buckled. The first analysis proceeded in eight 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 1,400 s. As required in the computation, the load steps are further divided into substeps (up to several hundreds). Results for the conventional fire are shown in Figs. M–5, M–6, and M–7.

At room temperature, even under the severe load redistribution due to the damage caused by the airplane impact, 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 temperatures referred to in these results are the hottest temperatures in the structure at any given time), the heated truss begins to show distress, especially in the compressed diagonal and vertical web members, which buckle inelastically. This means these heated steel members do not buckle elastically, but rather reach yielding in compression. Buckling is then governed by the tangent modulus of steel, which is about 0.5 percent of the elastic modulus, and the members immediately buckle after yielding, in the inelastic range. 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 floor 95. At that time, the connection of floor 95 to the perimeter column experiences its maximum compression of 125 kN (28 kip). Because slab 96 has a thermal gradient with its top surface at room temperature, its lateral expansion is much smaller than for slab 95, and its sag is larger. The connection between slab 96 and the column is always in tension (Fig. M–6). As expected, slab 94, heated at the top and cool at the bottom, bows upward. As the temperature continues to rise, more of truss 95 web members buckle inelastically, and the increasing sag begins to pull the column in. The horizontal deflection of the column becomes positive (inward), and the connection force between column and floor 95 turns to tension. This inward movement of the column relieves the tension in the connection between the column and floor 96. Further temperature rise causes further weakening in truss 96, which eventually becomes active in pulling the column in. At the peak temperature of 1,273 K, the maximum lateral deflection in the column (183 mm or 3.3 in.) occurs at floor 96, inward, and the connection between the column and floor 96 experiences a tension of 185 kN (42 kip).

Under the second fire scenario, the structure exhibits similar behavior. Figures M–8, M–9, and M–10 show the resulting deflections. Inelastic buckling of the diagonals causes considerable vertical deflection of the heated floors beyond a maximum temperature of 900 K. At 1,220 K, the sag of floors 96 and 97 overcomes the outward push on the perimeter column due to thermal expansion of floors, and pulls the column inward. This transition causes the column to temporarily straighten up, causing the overall floor deflection to be less. Figures M–11 and M–12 show severe horizontal tension greater than 120 kN (27 kip) at the connection of floors with the internal column (floor 95) and external column (floor 96).

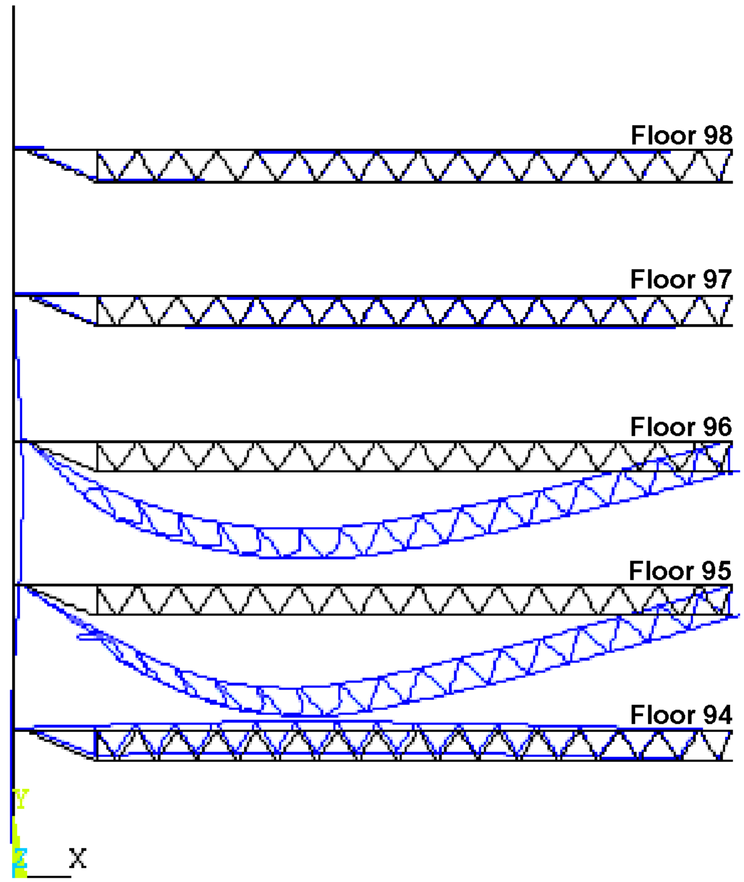

Figure M–5. Deformed shape at 1,273 K (not to scale).

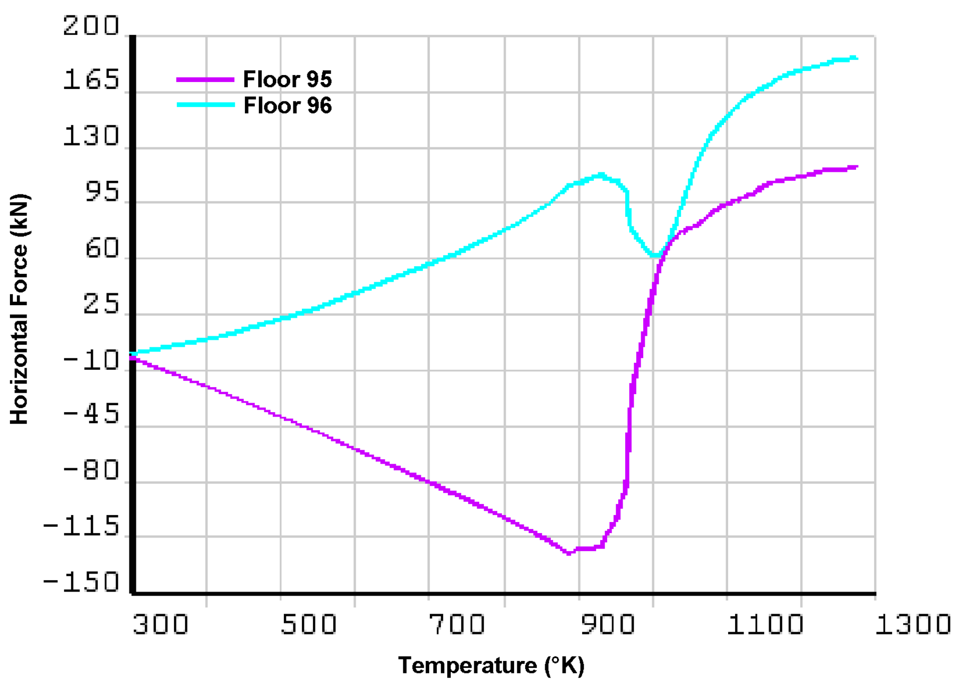

Figure M–6. Horizontal force (kN) between column and floors versus temperature (K).

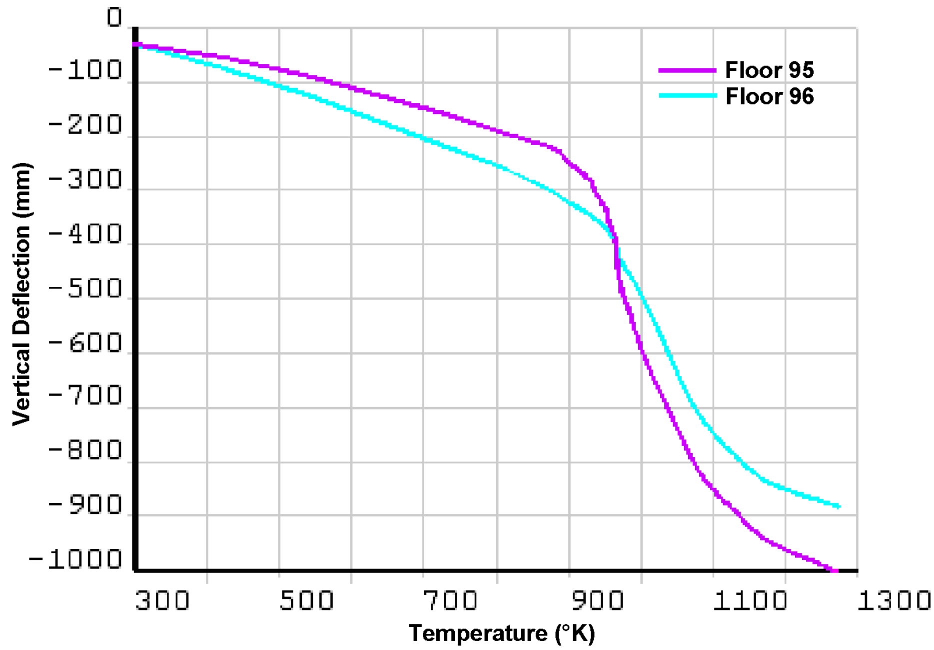

Figure M–7. Vertical deflections (mm) of floors versus temperature (K).

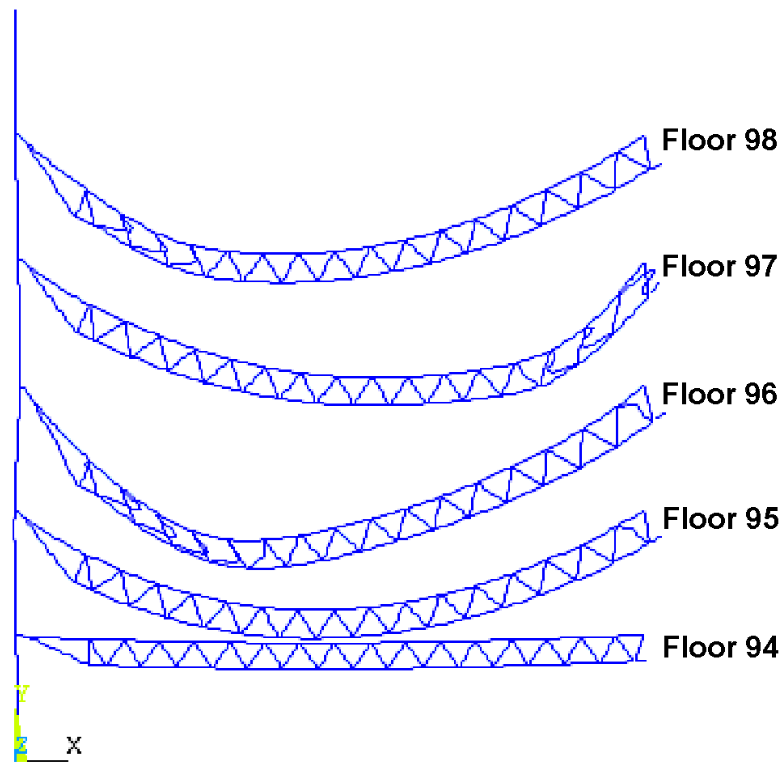

Figure M–8. Overall deflected shape for fire scenario 2-04 (not to scale).

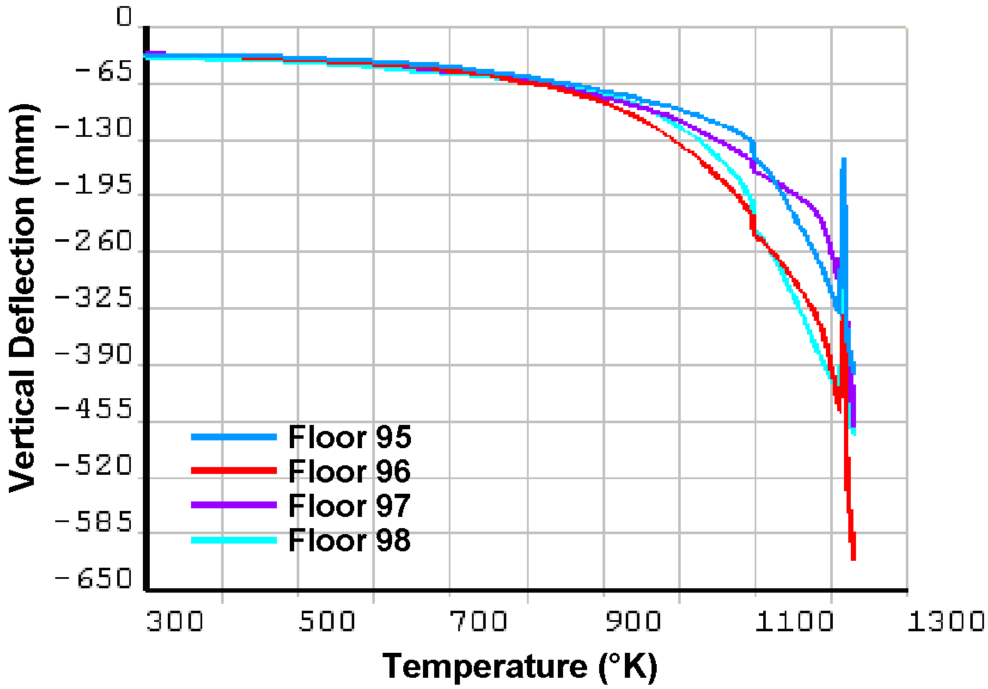

Figure M–9a. Maximum deflection of floors 95–98 for fire scenario 2-04.

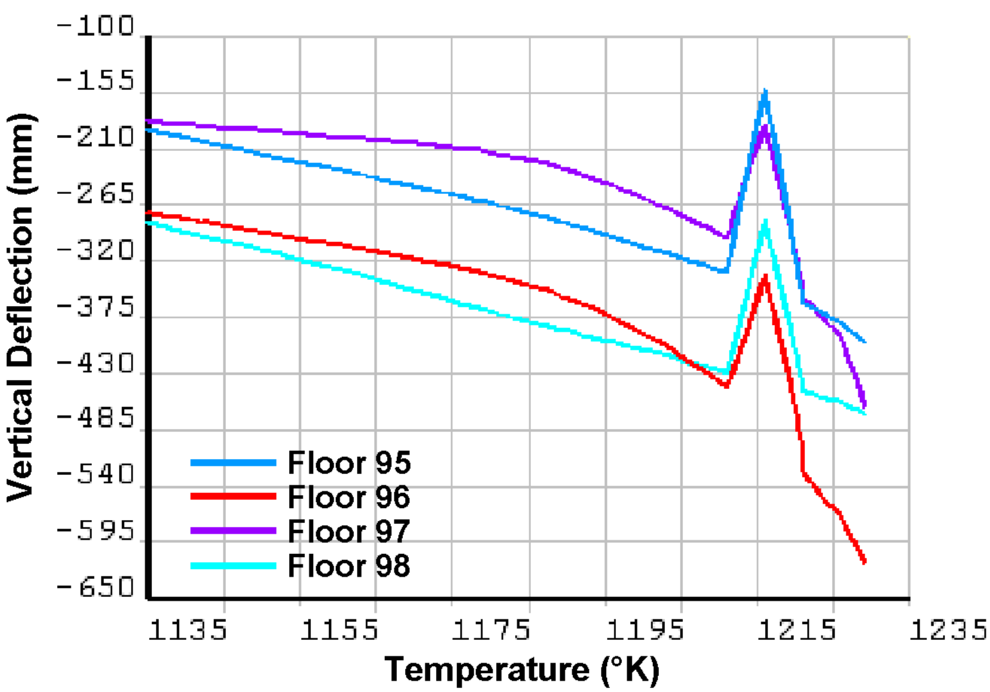

Figure M–9b. Maximum deflection of floors 95–98 for fire scenario 2-04: details of Fig. M–9a at high temperatures.

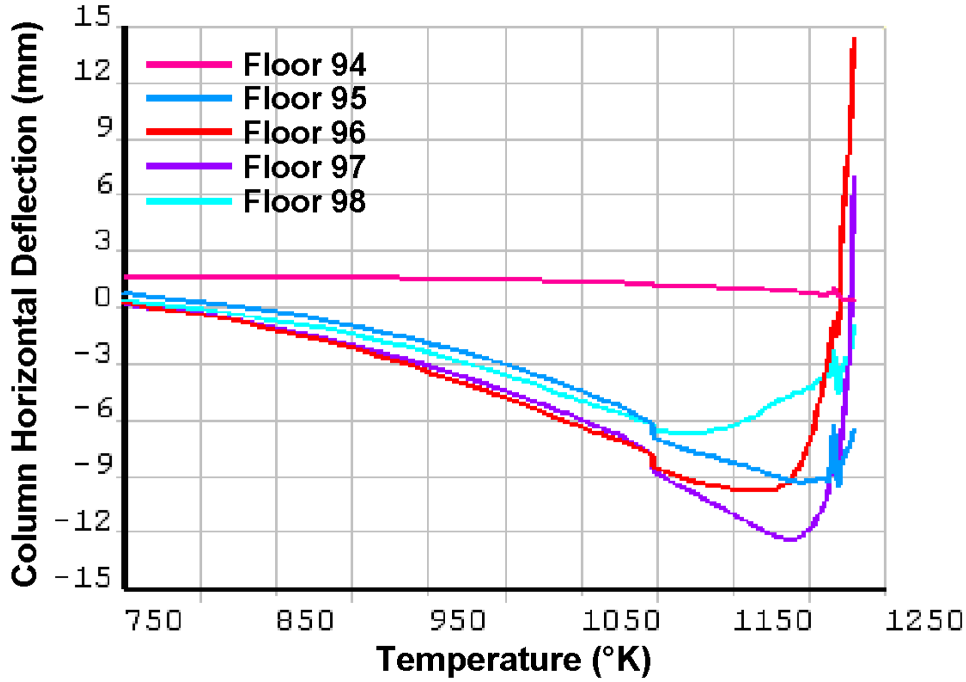

Figure M–10. Perimeter column lateral deflection for fire scenario 2-04.

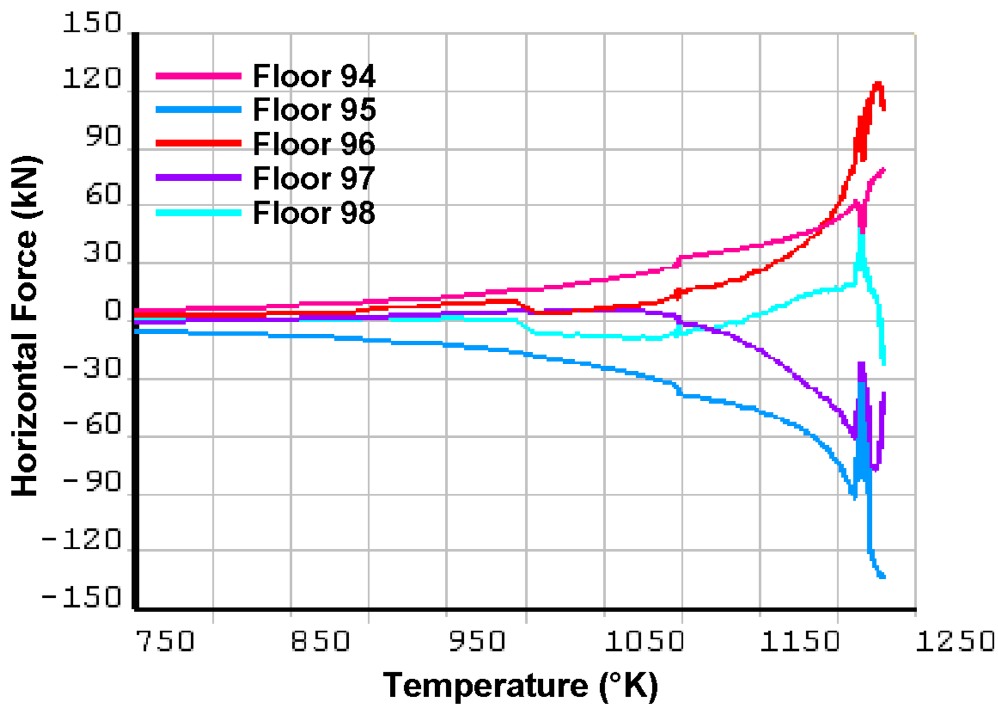

Figure M–11. Horizontal force at connection between floor and internal column for fire scenario 2-04.

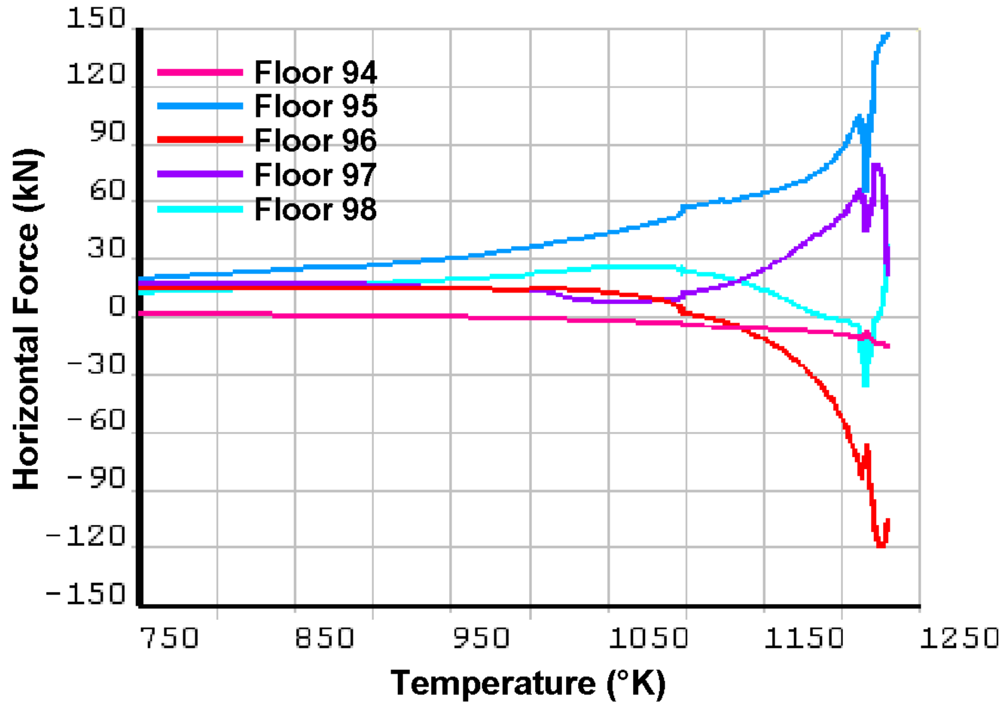

Figure M–12. Horizontal force at connection between floor and external column for fire scenario 2-04.

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 loss of composite behavior of the concrete slab with the steel truss may occur at a temperature and strain level yet to be determined.

For comparison with Quintiere et al. (2002), the present 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 diagonal and the vertical web members had buckled in each of the floors heated by the conventional fire. This conclusion assumes that the various structural connections maintain their integrity throughout the fire.

M.7 CONCLUSIONS

A model has been developed to provide insight and evaluate some aspects of a possible collapse sequence of the WTC towers. Its results are subject to the following qualifications: (1) the approximate nature of the material properties used, especially the concrete slab; (2) connection failures are not considered, although information is provided on demand experienced by the connections; and (3) the model is two-dimensional. In one of the two cases covered by this report, the temperature distribution of the members is selected from among those assumed by Usmani et al. (2003). In the second case, the temperature distribution is physically based, and was obtained by using the NIST Fire Dynamics Simulator with reasonable initial conditions associated with a damaged tower. In both cases, the diagonals buckle inelastically, causing considerable sag in the fire floors. This behavior puts a high-tension demand on the column, which remains at moderate temperatures in this model (same temperature as in Usmani et al. [2003]), and does not experience buckling. This is the major difference between these results and Usmani et al., even though the heated trusses in the present 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 sensitive to material properties.

M.8 NOTE

For confirmation, a 12-floor model (from floors 91 to 102) was developed and loaded with the same floor load and conventional temperature distribution described by Eq. 1 (Usmani et al. 2003) and mentioned above. Compared to case 1 reported earlier, the hinged column end conditions, the heated floors (95 and 96) and the bending moment applied on top of the perimeter column are the same, but the axial compression is reduced (950 kN or 210 kip) because of the fewer floors above the model. Results of the 12-floor model, shown in Figs. M–13, M–14, and M–15, are very similar to those of the 5-floor model, thus confirming the discussion in Section M.3.

M.9 REFERENCES

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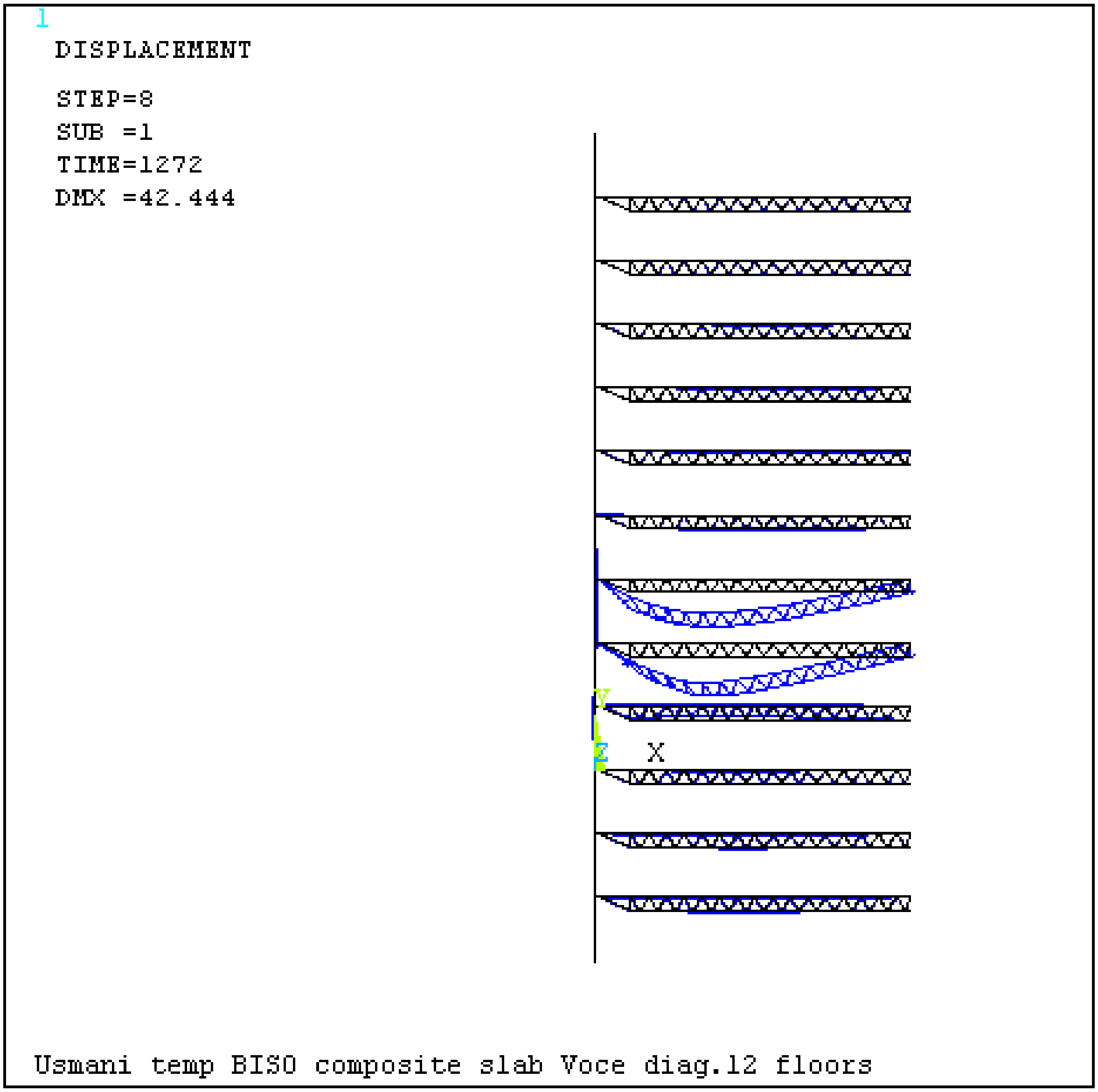

Figure M–13. Overall deflection (not to scale) of 12-floor model (floors 91–102) subjected to gravity loads and with floors 95 and 96 under conventional fire: at the maximum temperature of 1,272 K, the maximum deflection is 1.08 m.

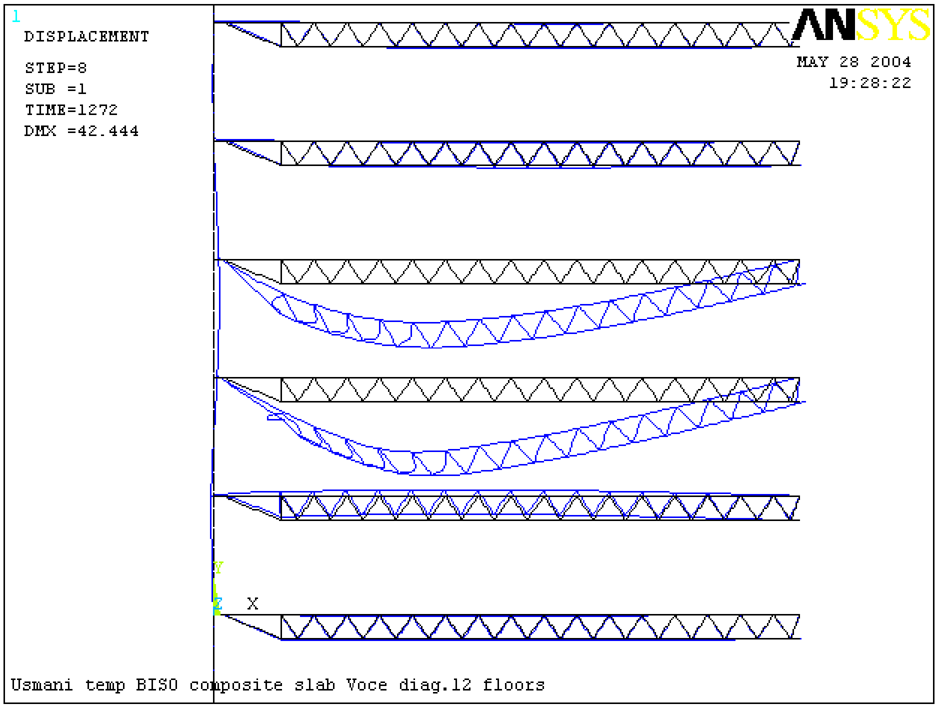

Figure M–14. Details of floors 93–98 for the 12-floor model shown in Fig. M–13 (compare with Fig. M–5).

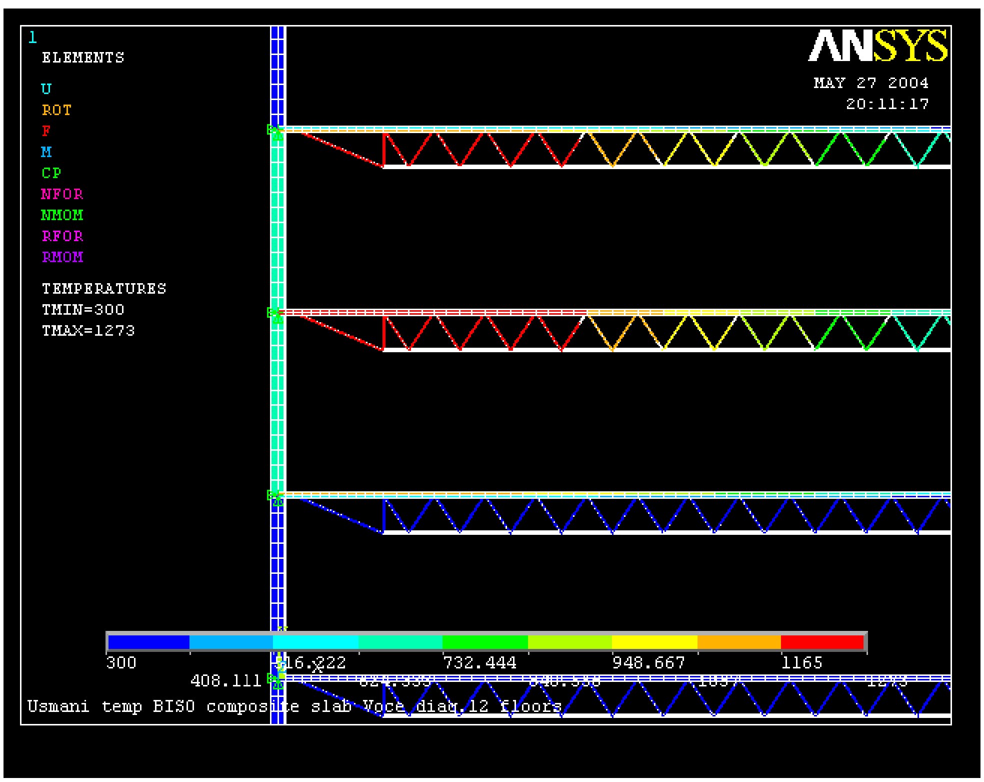

Figure M–15. Details of temperature distribution of floors 92–95 for 12-floor model: note temperature gradients across slab thickness of floors 93 and 95, and cool column.