

Collapse scenarios of WTC 1 & 2 with extension to generic tall buildings

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ABSTRACT

This paper presents a summary of the author's investigation into the collapse of tall buildings. A large number of computational analyses have been carried out at the University of Edinburgh (UoE) over the last 4 years in order to understand the collapse of the tall buildings of the World Trade Center (WTC) complex on September 11, 2001 following the terrorist attacks that day. The aim of these analyses has not been to carry out a "forensic" investigation (as this was done by official US government sponsored investigation by NIST, see wtc.nist.gov). The primary purpose of the UoE investigations was to understand the global collapse mechanisms of tall buildings as a result of extensive (involving multiple floors) fires and from these analyses identify any "generic" collapse mechanisms that may or may not exist. Such identification will allow the development of new design methods resulting in enhancing the safety and robustness of tall buildings against fire.

1 INTRODUCTION

University of Edinburgh researchers have been the global leaders in modelling whole structures in fire and have produced a great deal of new understanding on the integrated structural response to fire. This work began with a 4 year modelling project soon after the Cardington fire tests [1] in the mid-90s and has continued ever since.

Since the events of September 11, 2001 there has been considerable interest in understanding the collapse of the tall buildings in fire. Whole structure response analyses with the aim of establishing the precise collapse mechanisms for WTC tower like structures were carried out by the research group at the University of Edinburgh [2-4] with the sponsorship of Arup. A large number of finite element models were constructed, beginning with small 2D models [2,3] and more recently some very large 3D models [4] (some of over a million degrees of freedom). Very interestingly the results obtained in the earlier 2D analyses have shown a great deal of consistency with the results from the larger 3D analyses. This is very encouraging as it suggests that it may be possible to understand tall building collapses using relatively simple models (if the key aspects of the structure and fire can be included).

All WTC analyses were carried out using models structurally consistent with the structure of the towers (using tubular columns and truss members for the floor system support). This raised the question that whether the collapse scenarios and mechanisms discovered so far are somehow unique to the WTC tower frames. A recent paper [5] extended the previous work by investigating more "generic" tall building frames made of standard universal beam and column sections to determine whether the same collapse mechanisms are obtained. This indeed turned out to be the case which again presents a great opportunity for developing general rules, guidelines and design methods for ensuring the robustness and stability of tall buildings in fire. Furthermore, initial ideas will be discussed on developing generally usable indicators of the propensity of a fire induced collapse in a tall building based on the key parameters of, fire severity, number of floors affected and relative column and floor stiffness.

2 MODELLING OF WTC TOWERS

Results from range of small 2D models to large 3D models of the WTC towers structure will be presented in the following subsections.

2.1 A Simple 2D model of the WTC towers

A fully non-linear finite model of a typical 2D slice of the Twin-Towers structure encompassing 12 floors around the impact level in the North Tower was constructed [2] using the commercial software package ABAQUS. To simplify the model as much as possible, it was assumed that the columns in the core region of the building would be relatively cool and that collapse would initiate at the external columns. Because the external floor system is likely to be laterally well restrained by the floor system in the core, all translations of the composite trusses and concrete slab at the core end are assumed fully restrained (rotation remains free). Typical dimensions of all members that make up the model (the truss components, concrete deck slab and the external column) are shown in Figure 1 [6]. The concrete deck slab is modelled to act compositely with the truss. Temperature dependent material properties for steel (stress-strain curves and coefficient of thermal expansion) were taken from Eurocode 3. Loading has been applied based on the approximate numbers mentioned in the FEMA Report [6]. A total load of approximately 1300 to 1400 tonnes is assumed to be uniformly distributed over the floors. The Floor loading area for the slice of structure modelled is assumed to be 2m wide (consistent with the spacing of the double floor trusses). This gives a load of 8kN/m on the horizontal elements. 40% of this load on eight storeys (representing the storeys above the model) is applied to the top of the columns.

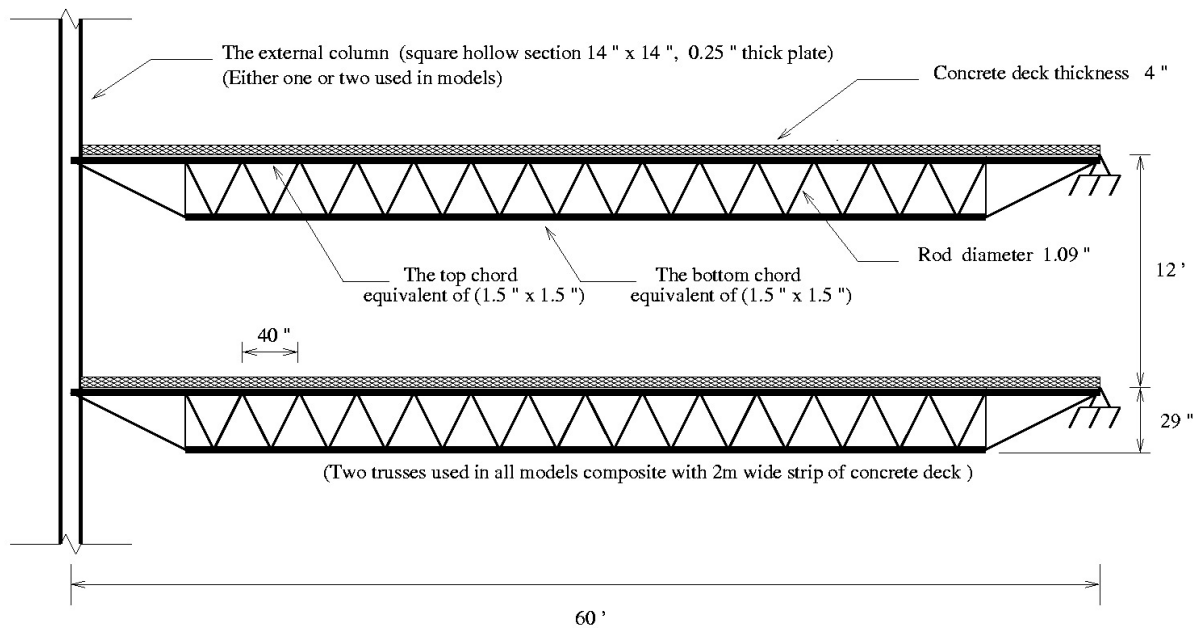


Figure 1: Details of the floor system

A generalised exponential curve is chosen to represent the fire time-temperature relationship, and is given by:

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

Where, T_{\max} is maximum compartment temperature, T_0 is the initial or ambient temperature, and a is an arbitrary 'rate of heating' parameter. This fire is then applied for a range of values

of maximum temperature T_{\max} and parameter a with various assumed temperature distributions along the length of the composite floor for fire scenarios encompassing one, two or three floors. Many analyses were carried out and the results are reported in detail other published articles [2,3], only a brief description of the results will be provided here for a three floor fire (on floors 4, 5 and 6) with a maximum temperature of 500°C and value of a of 0.005. The columns are protected and are restricted to a maximum temperature of 400 °C.

The key failure mechanism obtained from these sets of analyses is illustrated by Figure 2, which shows the membrane force development in all the twelve floors through the whole duration of the analysis. Initially we see that floors 5 and 7 have increasing compressions until about 200 seconds into the fire after which they begin to buckle simultaneously. The other hottest floor which initially goes into tension (as it gets stretched by the elastic curve of the column assuming the column-floor joint displacements remain compatible) also moves in to compression (as the other two lose it). All of the hottest fire floors go into tension at approximately 1000 seconds (it may be noted that the temperature at this point of the steel truss is still under 400°C). This causes floors 8 and 4 (immediately adjacent to the other three) to attract large compressions from the column and the tensile action of the three fire floors. As these floors are not hot they resist much larger compressions before they also begin to buckle, floor 4 first and then floor 8. These are followed by adjacent floors 3 and 9 which form the next front of resistance to the compression pulse travelling outward. The structure begins to collapse when floors 3 and 9 buckle and move into tensile membrane behaviour.

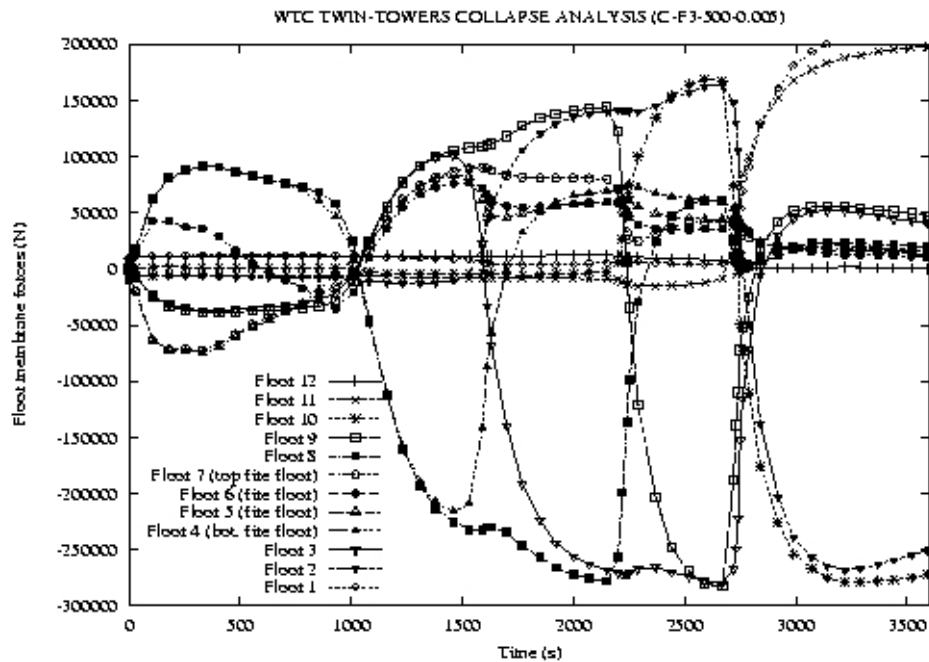


Figure 2: The floor membrane forces in the simple 2D model

In fact a careful analysis of the model results reveals a very interesting progressive collapse mechanism despite the obviously high buckling load capacity of the column(s). This mechanism is illustrated in detail schematically in Figure 3.

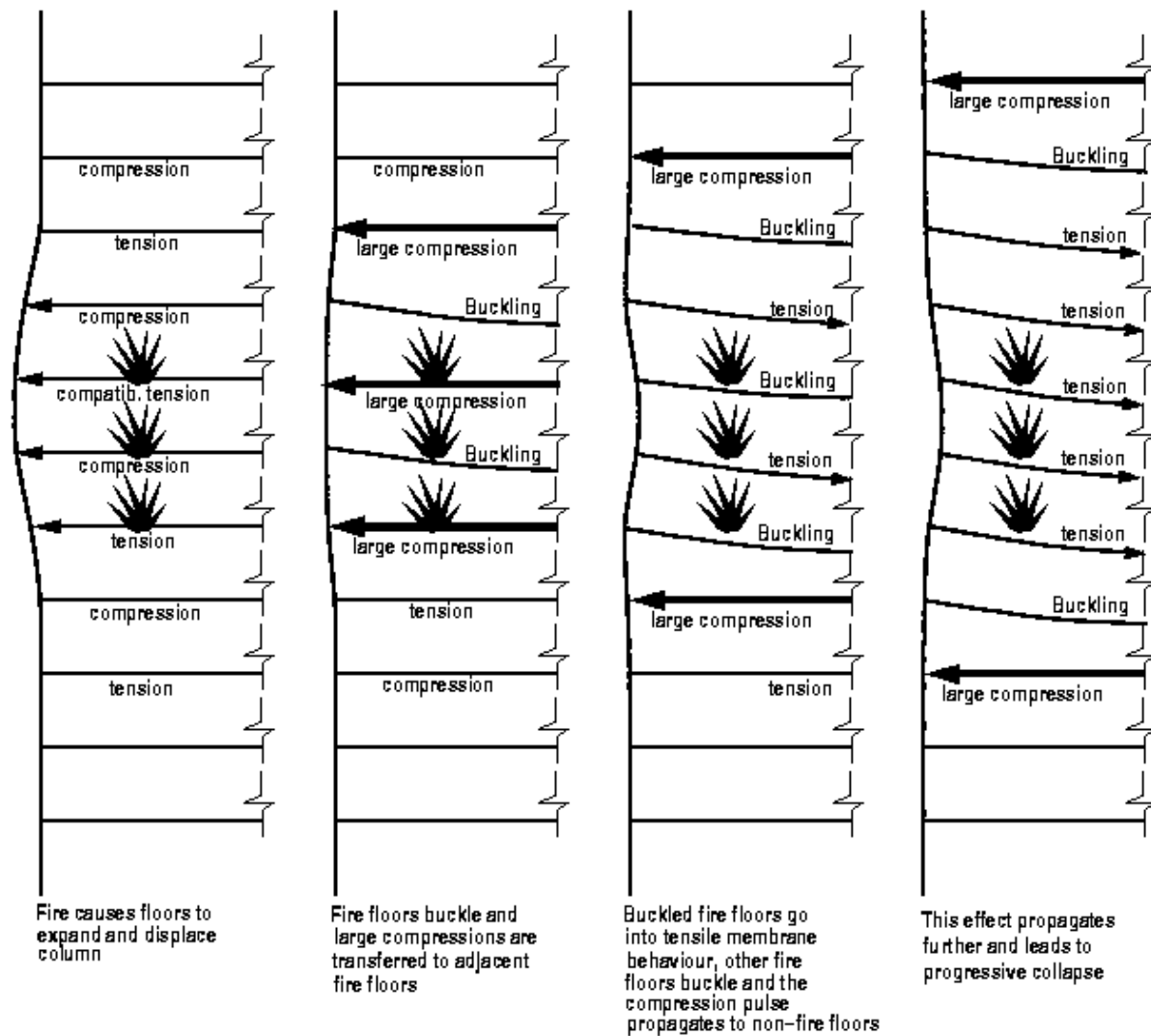


Figure 3: The “compression pulse” collapse mechanism

2.2 A Larger 2D model of the WTC towers

Figure 4 shows the model the finite element mesh for the larger 2D analysis. A 2D slice was taken through the full 63m width of the building, including the core. This slice was intended to capture primarily the reaction of the composite floor system over the main floor truss in the outer areas as in reference [2]. Again 12 storeys are considered in this model to allow multiple floor fires to be considered while still allowing a reasonable distance between the fire compartments and the boundaries of the model. Additional loading was applied to the top of the columns to represent further floors above.

The main floors are split into outer areas supported by long span trusses and the inner, core area, supported by a beam and slab floor system. The slab has been modelled as composite with all beams and trusses but is not directly connected to the columns. Again a three-floor fire is applied on floors 5, 6 and 7 with a maximum temperature of 800°C and value of α of 0.005. The columns are protected and are restricted to a maximum temperature of 400 °C.

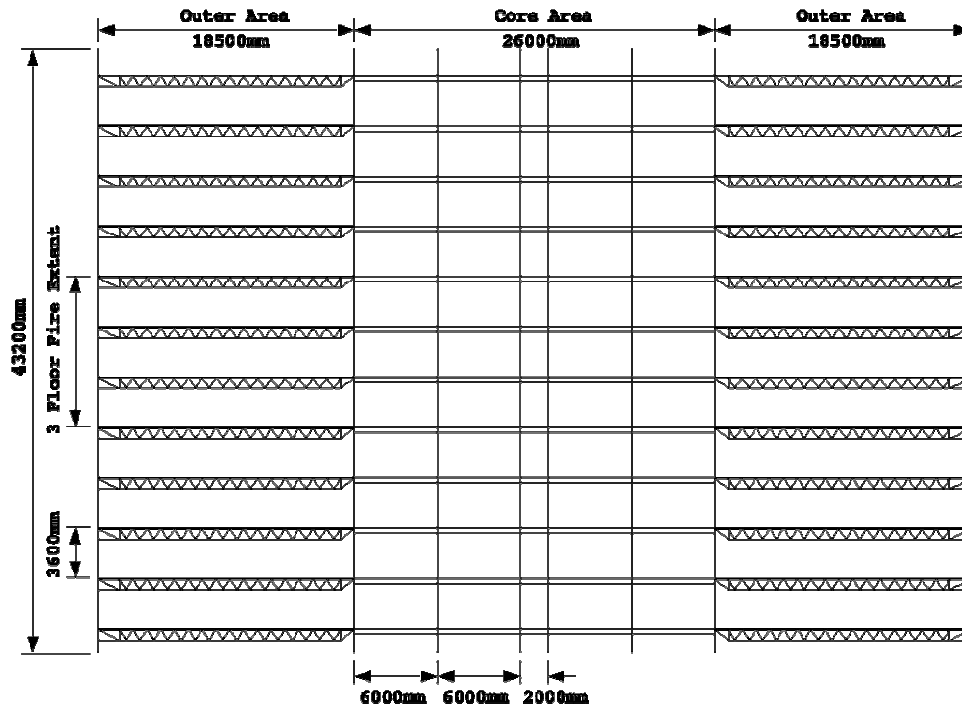


Figure 4 : Larger 2D Model Mesh

The final displaced shape of the structure can be seen in Figure 5. The collapse mechanism observed using this model is different. All the floors affected by fire (6, 7 & 8) start to expand. As the fire floors 6 and 8 are adjacent to cool floors (5 & 9), they go into high compression as they expand against the restraint provided by the column (see **Error! Reference source not found.**). The cool floors (5 and 9) adjacent to the fire floors go into tension as the column is pushed out and pulls these floors along.

As the interface fire floors (6 & 8) begin to lose stiffness they retract and the outer columns also begin to return until the floors move into tensile membrane/catenary action and actively pull the column inward. To counter this, the cool interface floors (5 & 9) go into compression. The forces again pivot their way through the rest of the cool structure. As the interface fire floors lose stiffness they move into tension and middle fire floor moves into compression as the column moves back in. The middle fire floor (7) is subjected to compression for up to 330s before large deflections in the floor bring it into tension.

The movement of the column over the fire floors is directly related to the forces applied to it by the displacements of the fire floors. The outer columns at this point in the analysis can be assumed to be acting as linear springs connected to the ends of the trusses. Such movement induces moments in the columns. Toward the end of the analysis (at the collapse of the structure) the moments in the outer columns begin to reach the yield surface defined by the moment and axial force interaction for the column section. Thus the failure of the structure is due to plastic hinges forming at the column connections of floors 4, 7 and 9 as shown in Figure 5. Full details of this and other similar analyses can be found in [4].

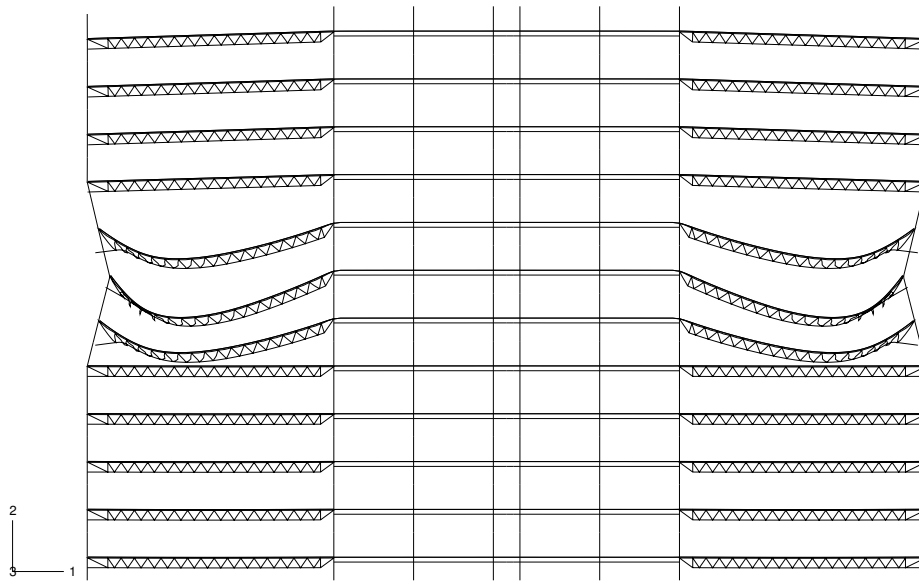


Figure 5: Final Displaced Shape

2.2 A 3D model of the WTC towers

2D models have inherent limitations. The assumption in the 2D case described above that the structure was effectively of infinite extent of closely spaced truss lines may give an unrealistic response when compared to a more realistic structure. In order to determine if the collapse mechanisms indicated by the 2D work were realistic a set of 3D models was investigated. As described in Section 2.2 the 3D model was based closely on the 2D model but also included a realistic distribution of structure in the 3rd dimension.

The response of the multi-storey 3D structure was very similar to that of the 2D analyses that preceded it. The response was again driven by the thermal expansion in the floor system. The large displacements imposed upon the floor system by the fire initially pushed the boundary columns out before increasing midspan deflections lead to catenary/tensile membrane action. This change in the floor geometry draws the column lines inward until they reach yield in combined bending/axial force. Figure 6 shows that the 3D model also produces the large midspan deflections typical of the 2D analyses. Inward movement of the columns is also of the same order as that indicated in the 2D analyses. The primary difference between the 2D and the 3D analyses is that in the 3D analysis the response is more gradual down the long span sides of the building. As each column is affected it gradually sheds load onto the column next to it. Thus the centre of the long span sides shows much greater deflections than the more stiff corners of the building.

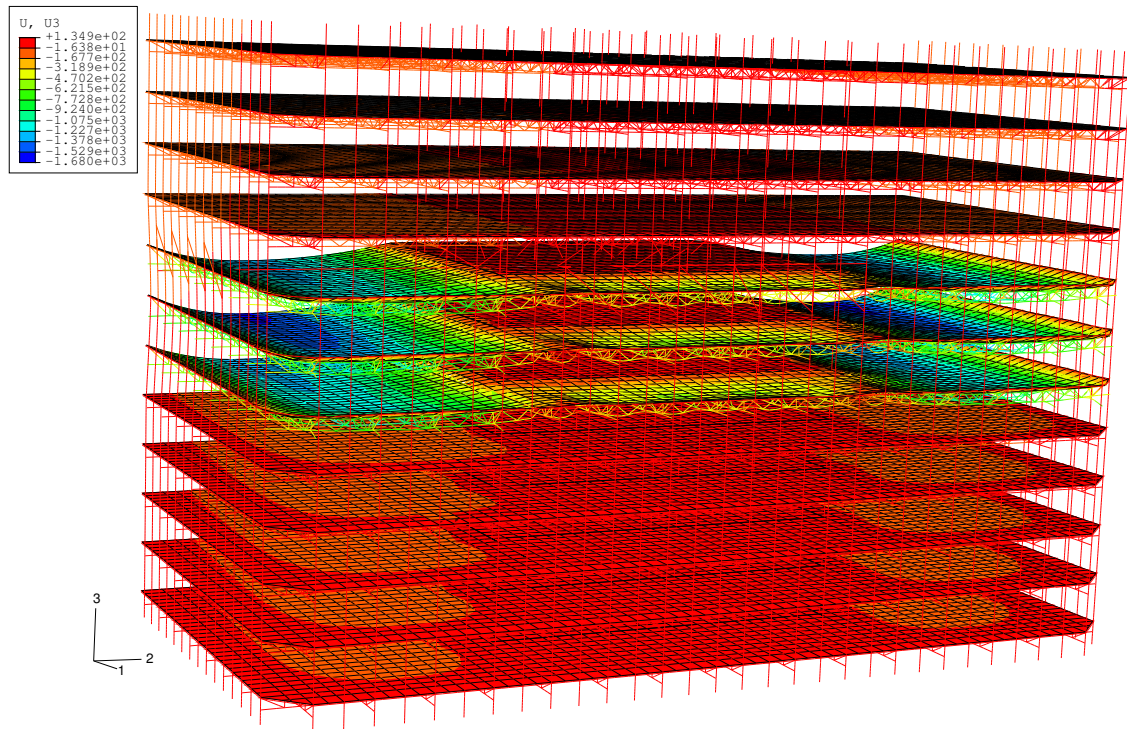


Figure 6: Typical 3D Floor Displacements

3 MODELLING OF GENERIC TALL BUILDINGS

The two main failure mechanisms established in the previous section illustrated in Figure 7. Figure 7 (a) shows a mechanism that would occur if a stiff column was supported by a relatively weak (in membrane compression) floor system [2,3]. If however the floors were stiff enough a conventional plastic hinge mechanism seems to establish [4] as a result of the moments imposed upon the column by the floors in tension and $P-\delta$ moments. These mechanisms are based on analyses that assume that no connection failure occurs. This assumption allows the focus to be on “global” behaviour as it can be reasonably assumed that this would produce a useful upper bound reference collapse scenario. Local effects such as connection failure, local cracking of concrete, failure shear connectors and their endless permutations could potentially produce a whole range of alternative collapse scenarios, which could reasonably be assumed to produce earlier failures than the reference scenarios (although this is not by any means certain). In a design context local effects can really only be considered properly in a probabilistic rather than deterministic manner.

All previous analyses were carried out using models close to the WTC towers (using tubular column and truss members for the floor support). This work was extended [5] by investigating more “generic” tall building frames made of standard universal beam and column sections to determine whether the same collapse mechanisms are obtained. Furthermore, a first attempt was also made to develop some generally usable indicator of the propensity of a fire induced collapse in a tall building based on the key parameters of, fire severity, number of floors affected and relative column and floor stiffness.

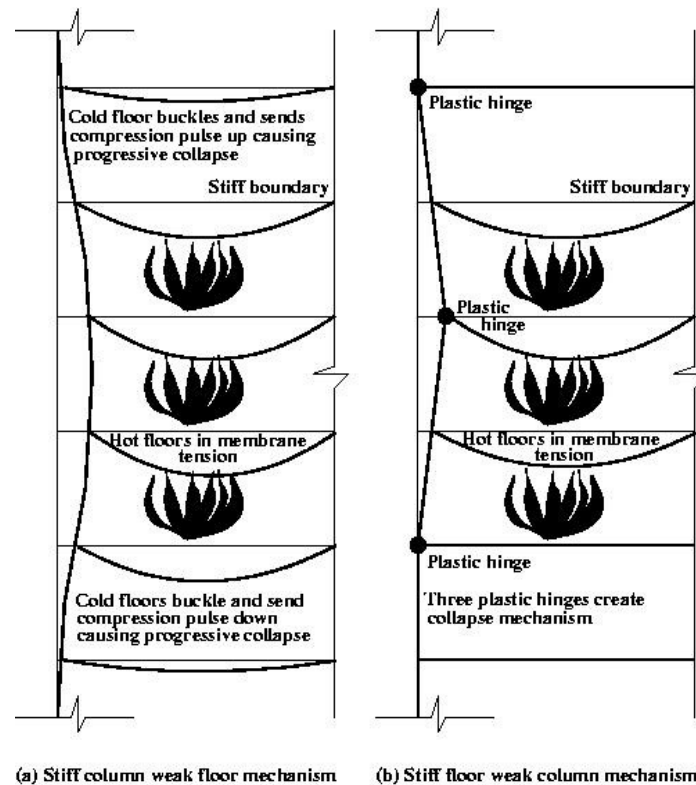


Figure 7: Suggested collapse mechanisms for WTC towers structure in fire

3.1 Generic Multi-Storey Frame Models

A more conventional composite steel frame model was constructed to determine that the collapse mechanisms discovered in the context of WTC towers analyses based on the long span truss floor system could be generalised to include more conventional structures. Figure 8 shows the model details.

This is a composite floor system, where the beams and columns are UB and UC sections respectively. The beams are laterally restrained by the stiff concrete core but are free to rotate. They are fully fixed to the column, which in turn is fixed at the bottom but restrained only in the horizontal direction at the top. The concrete slabs are designed to act compositely with the beams and are connected with multiple point constraints. All sections are modelled using 2-D beam elements. The structure is subjected to loading on the beams and the column. Each beam supports a UDL which includes the self weight of the concrete slab as well as the imposed load. The column is subjected to a point load which represents the additional floors above the analysed structure. To compare the behaviour of the models several parameters were changed to obtain a wide variety of results. This includes changing loads, section sizes and spans. The assumed material properties are in accordance with Euro Code 3-1.

The fire affects floors 6, 7 and 8. The steel is assumed to be unprotected and thus has a uniform temperature equal to that of the fire, shown in Figure 8. The maximum and ambient temperatures are taken as 800°C and 20°C respectively and α is taken to be 0.005. The columns are protected and are restricted to a maximum temperature of 400 °C.

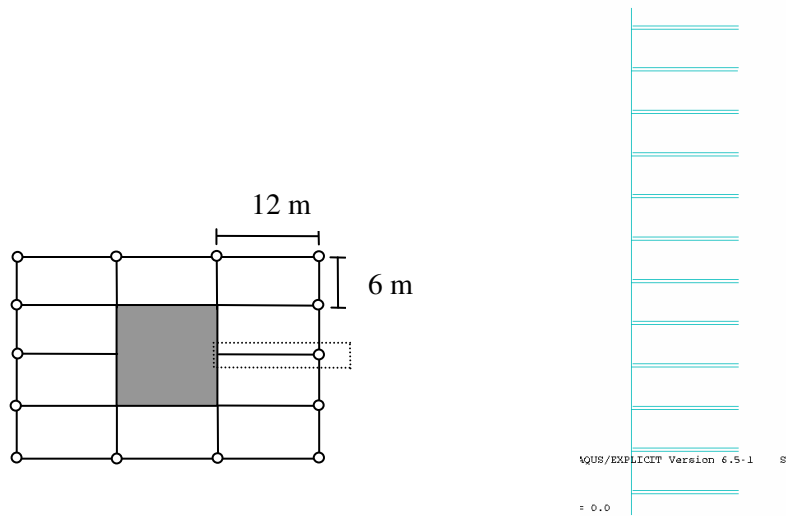
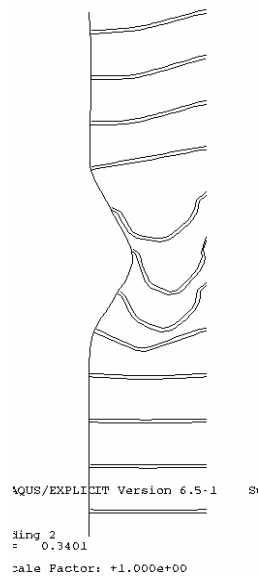


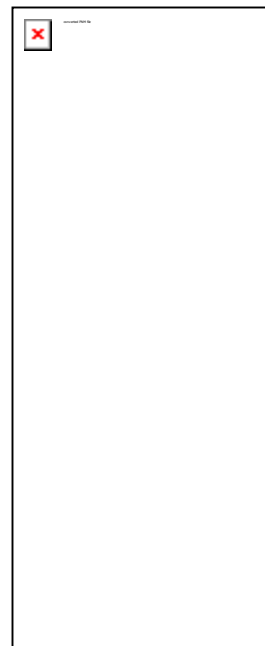
Fig. 8: Typical cross section of a multi-storey frame model and the Finite Element Model

Figure 9 shows the deformed collapsed shapes for two different models, essentially reproducing the two mechanisms shown in Figure 7. The stiff column weak floor mechanism shows a clear plastic collapse with three hinges forming at the floors above and below the fire floors and at the centre fire floor. The stiff floor weak column shows that the column forces the floor below the fire floors to buckle, thus increasing the loading on the floor below and starting a progressive collapse.

The weak column stiff floor however, shows that only the fire floors deflect further and that no movement of the column occurs at any other point. This coincides with the three hinge failure assumption that the collapse is localised.



(a) Stiff column weak floor mechanism



(b) Stiff floor weak column mechanism

Figure 9: Deflected shapes with a buckling and plastic collapse respectively

4. A SIMPLE STABILITY ASSESSMENT METHOD FOR TALL BUILDINGS IN MULTIPLE FLOOR FIRE

Figure 10 illustrates a simple method for assessing the stability of columns in tall buildings in multiple (or single) floor fires. The method may be described as follows:

1. Determine the limiting tensile membrane forces in the floors affected by fire. This will involve calculations to obtain the thermally induced displacements and membrane forces in the floor. A detailed description of these can be seen in reference 4.
2. From the membrane forces obtain the moments induced in the columns at the “pivot” floors (adjacent to the fire floors). If an approximation of the column internal displacement can be made, additional $P-\Delta$ moments can be calculated.
3. At this point there are two possible mechanisms:
 - a. Calculate the reaction of the pivot floors as shown in Figure 9 (lowest pivot floor is most critical) counteracting the membrane “pull-in” forces (include an appropriate percentage of the column load to this, as the column lateral support requirement is increased due to loss of support at the fire floors). If the floor membrane is unable to provide the reaction calculated a weak floor failure becomes possible.
 - b. If the floor is able to provide the reaction required, check the temperature dependent moment-force interaction diagram for the column to ensure that the column has not reached the yield surface (and thus formed a plastic hinge). If this is the case then stiff floor failure can occur.

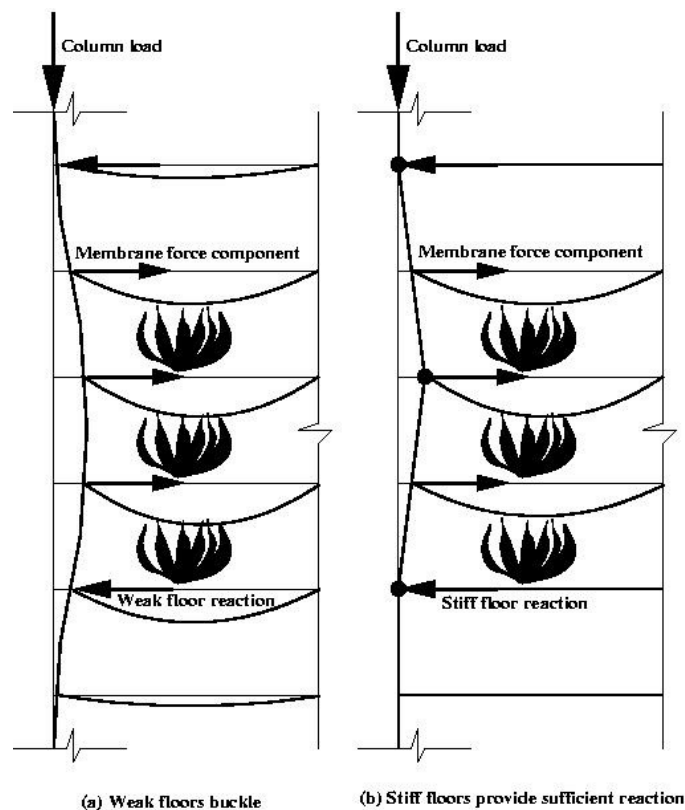


Figure 10. Mechanics of fire induced collapse in weak and stiff floor buildings

5. CONCLUSIONS

A review of previous work by the authors on modelling the collapse of tall buildings in multiple floor fires has been presented. This work has produced two possible failure mechanisms for tall buildings in multiple floor fires. The mechanisms are confirmed to exist by reproducing them in a finite element model of a standard or “generic” steel frame composite structure. This conclusion is very important and powerful as it enables the development of a simple stability assessment method for tall buildings in multiple floor fires. A very preliminary exposition of what such a method may entail is also described in the previous section.

REFERENCES

1. University of Edinburgh research reports on the UK government sponsored modelling of the full-scale Cardington tests (www.civ.ed.ac.uk/research/fire/cardington.html)
2. A.S.Usmani, Y.C.Chung and J.L.Torero. How did the WTC Towers Collapse? A New Theory, *Fire Safety Journal*, 38:501--533, 2003.
3. A.Usmani, Stability of the World Trade Center Twin Towers structural frame in multiple floor fires, *Journal of Engineering Mechanics*, ASCE, 131:654--657, 2005
4. G.R.Flint, *Fire Induced Collapse of Tall Buildings*, PhD thesis, University of Edinburgh, 2005.
5. Asif Usmani, Charlotte Roben, Louise Johnston, Graeme Flint and Allan Jowsey, Tall building collapse mechanisms initiated by fire, Proceeding of the 4th Workshop on Structures in Fire (SiF'06), Aveiro, Portugal.
6. Federal Emergency Management Agency, "World Trade Center Building Performance Study: Data Collection, Preliminary Observations and Recommendations", FEMA, May 2003.