TALL BUILDING COLLAPSE MECHANISMS INTIATED BY FIRE

ASIF USMANI[1], CHARLOTTE ROBEN[2], LOUISE JOHNSTON[2], GRAEME FLINT[1],
ALLAN JOWSEY[3]

ABSTRACT

This paper introduces the hypothesis of two possible failure mechanisms for tall buildings in multiple floor fires. This paper extends the previous work done on the WTC towers by investigating more "generic" tall building frames made of standard universal beam and column sections to determine whether the same collapse mechanisms are obtained. The outcome of this paper enables the development of a simple stability assessment method for tall buildings in multiple floor fires.

1. INTRODUCTION

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 University of Edinburgh in collaboration with Arup. The two main failure mechanisms established in this work are illustrated in Figure 1. Figure 1 (a) shows a mechanism that would occur if a stiff column was supported by a relatively weak (in membrane compression) floor system[1,2]. If however the floors were stiff enough a conventional plastic hinge mechanism seems to establish[3] as a result of the moments imposed upon the column by the floors in tension and P-δ moments, shown in Figure 1 (b). 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

[1] Doctor, University of Edinburgh, School of Civil and Environmental Engineering,
email: asif.usmani@ed.ac.uk, G.R.Flint@ed.ac.uk
[2] Undergraduate student, University of Edinburgh, Department of Civil and Environmental Engineering, ,
email: C.C.Roben@sms.ed.ac.uk, LS.Johnston@sms.ed.ac.uk
[3] PhD student, University of Edinburgh, School of Civil and Environmental Engineering, ,
email: A.I.Jowsey@ed.ac.uk

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.

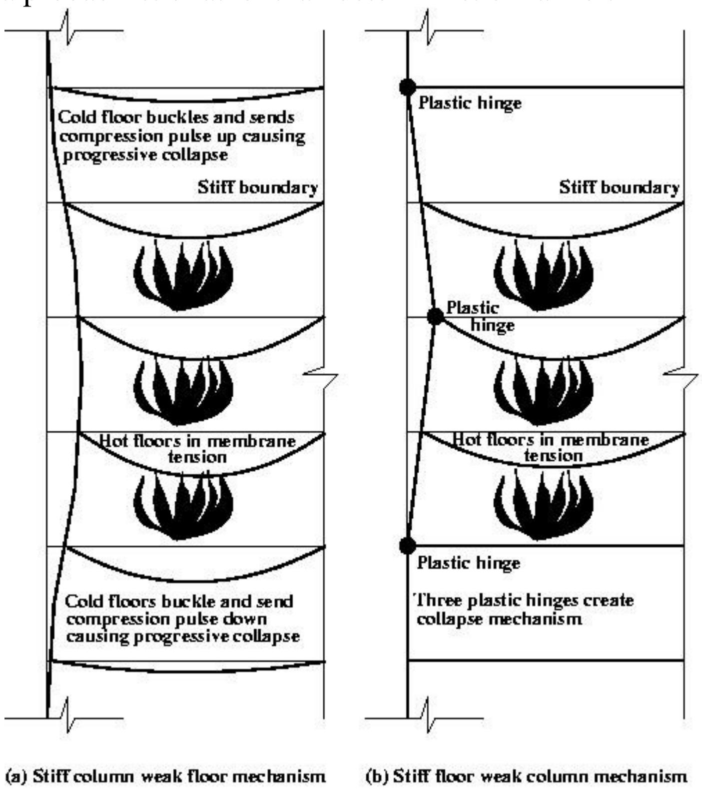


Fig.1 – Suggested collapse mechanisms for WTC towers structure in fire

All previous analyses were carried out using models similar to the WTC towers (using tubular column and truss members for the floor support). This paper extends 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. Furthermore, a first attempt is 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

2. MULTI-STOREY FRAME MODEL

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 2 shows the model details.

This is a composite floor system, where the beams and columns are universal beam and column 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.

To model the fire, a generalised exponential curve is chosen to represent the time-temperature relationship and is given by

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

where, T_{max} is maximum compartment temperature, T_0 is the initial or ambient temperature, and α is an arbitrary 'rate of heating' parameter. For the purpose of this research the maximum and ambient temperature are taken as 800ºC and 20ºC respectively, α is taken to be 0.005 and the time t is taken as 3600 seconds.

The fire is 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 3. The columns are assumed to be protected and are restricted to a maximum temperature of 400ºC at the end of the heating period, which is a conservative estimate. The concrete slabs have a non-uniform temperature distribution and follow the temperatures shown in Figure 4.

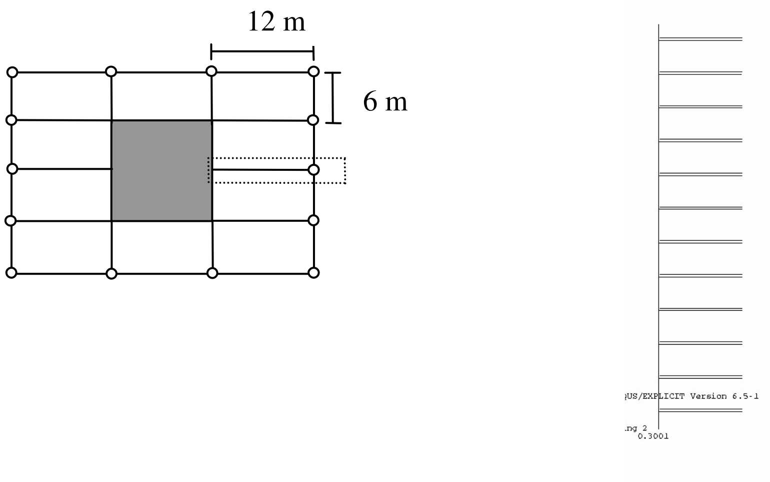


Fig. 2 – Typical plan of a multi-storey frame model and the Finite Element Model cross section adopted

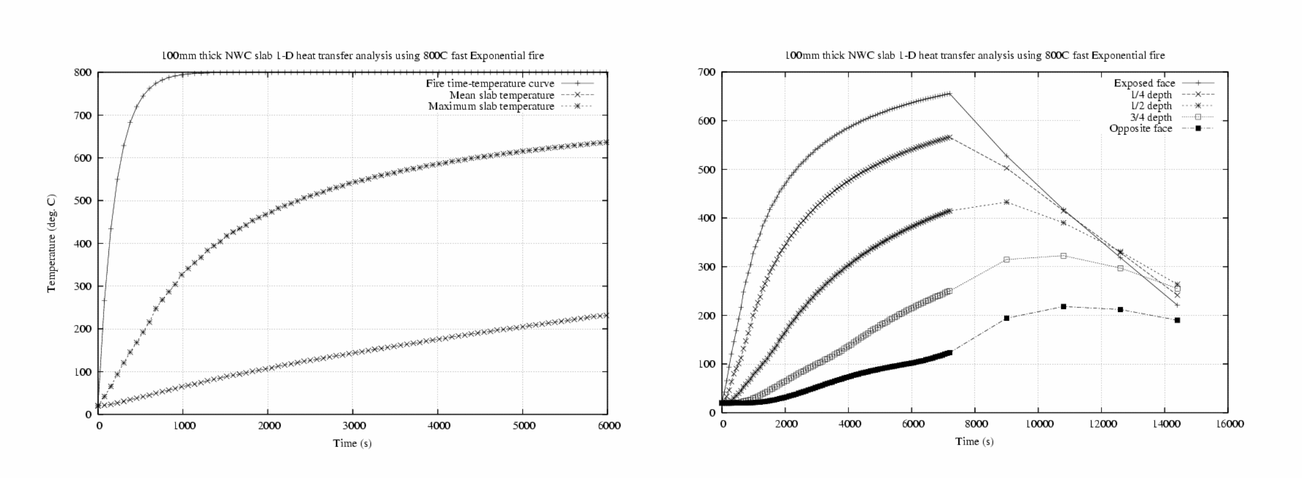


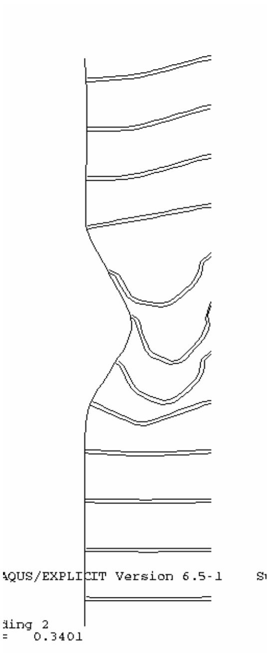
Fig.3 – Generalised fire curve and concrete temperatures through the slab

3. MODELLING RESULTS

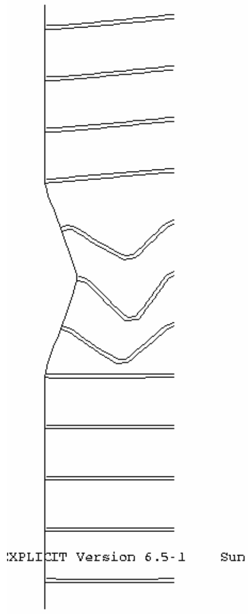
Figure 4 shows the deformed collapsed shapes for two different models, essentially reproducing the two mechanisms shown in Figure 1. The weak floor model shows a clear plastic collapse with three hinges forming at the floors above and below the fire floors and at the centre fire floor. The stiff floor model 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 horizontal deflection of the column is plotted for both models and can be found in Figure 5. Initially both show a negative displacement, indicating the outward movement of the column due to the thermal expansion of the beams. The weak floor model shows that the fire floors quickly deflect in the positive direction as the beams are pulling it in. As the column increasingly pushes against the floors below the fire these buckle and the column moves inward at these lower floors.

The stiff floor model 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.



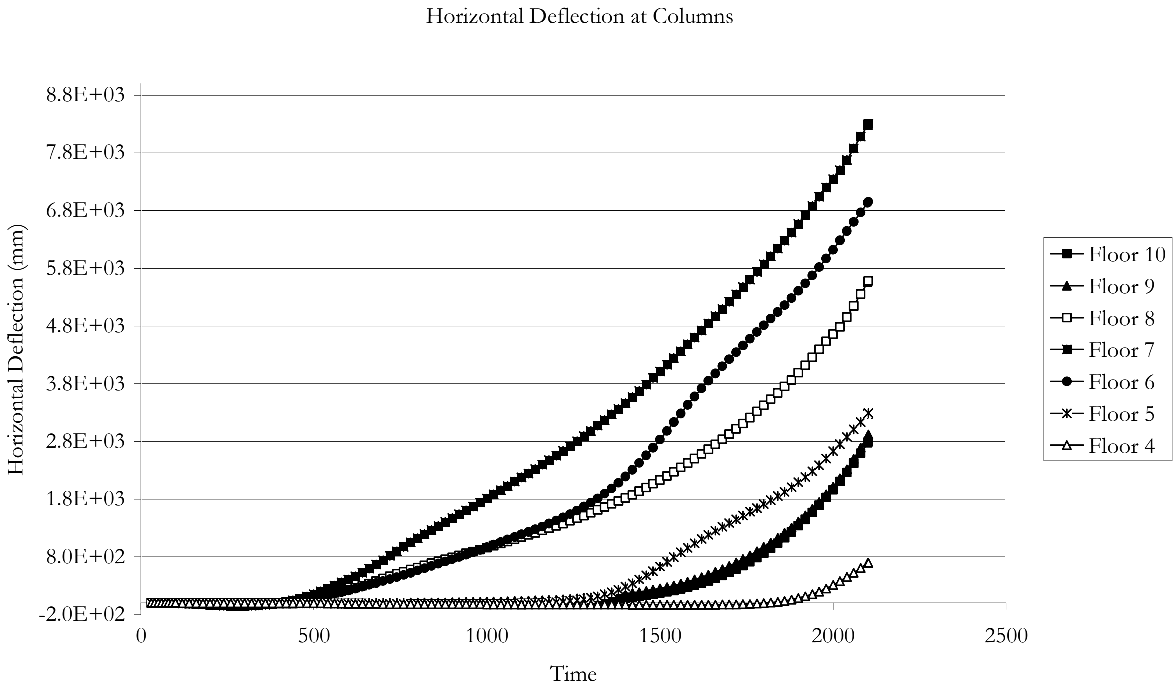
(a) Weak floor mechanism



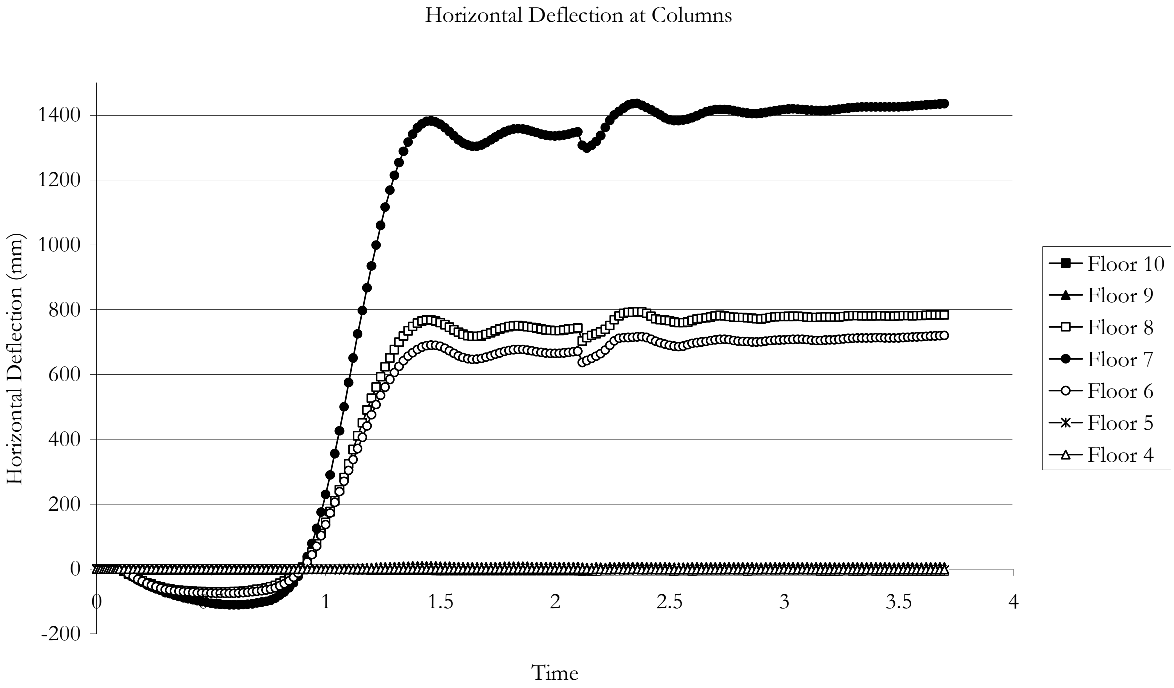
(b) Stiff floor mechanism

Fig. 4 – Deflected shapes with a buckling and plastic collapse respectively

The vertical deflection for the weak beam model shown in Figure 6 indicates that each section of the column deflects downwards starting with all the floors above the fire floors and gradually each consecutive floor follows. The stiff floor model initially has an upward movement due to the thermal expansion of the column. As the column is being pulled in and the collapse movement is initiated there is a sharp increase in vertical deflection for all the fire floors and those above. Floors 4 and below do not encounter any deflection.

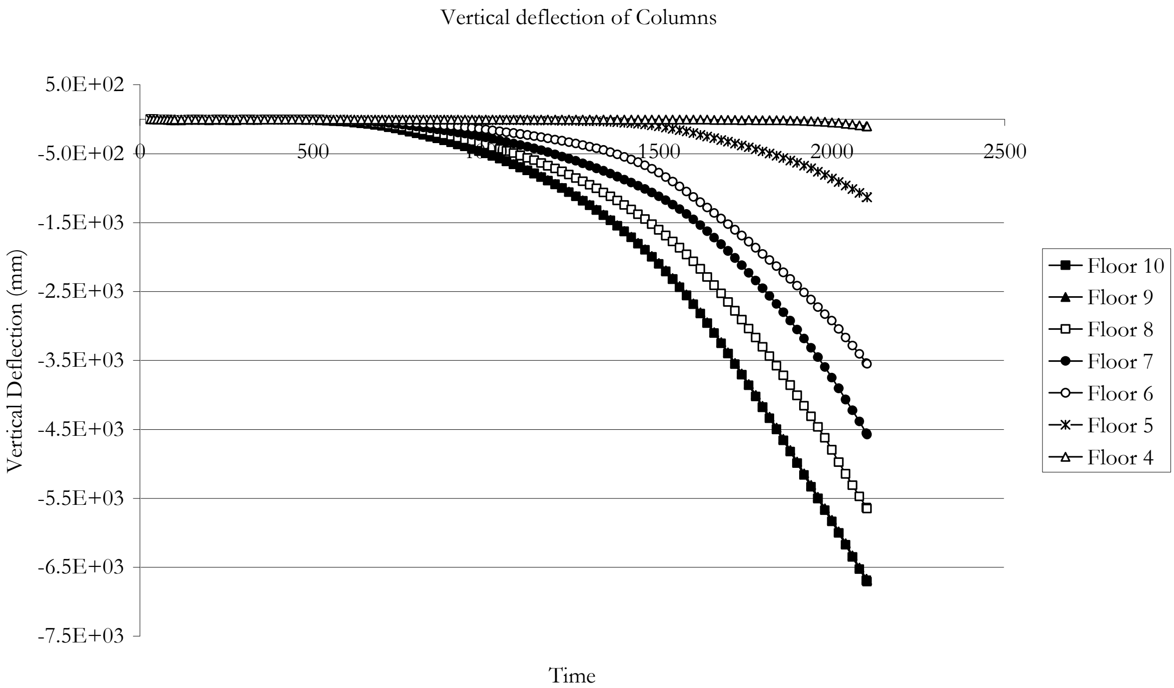


(a) Weak floor mechanism

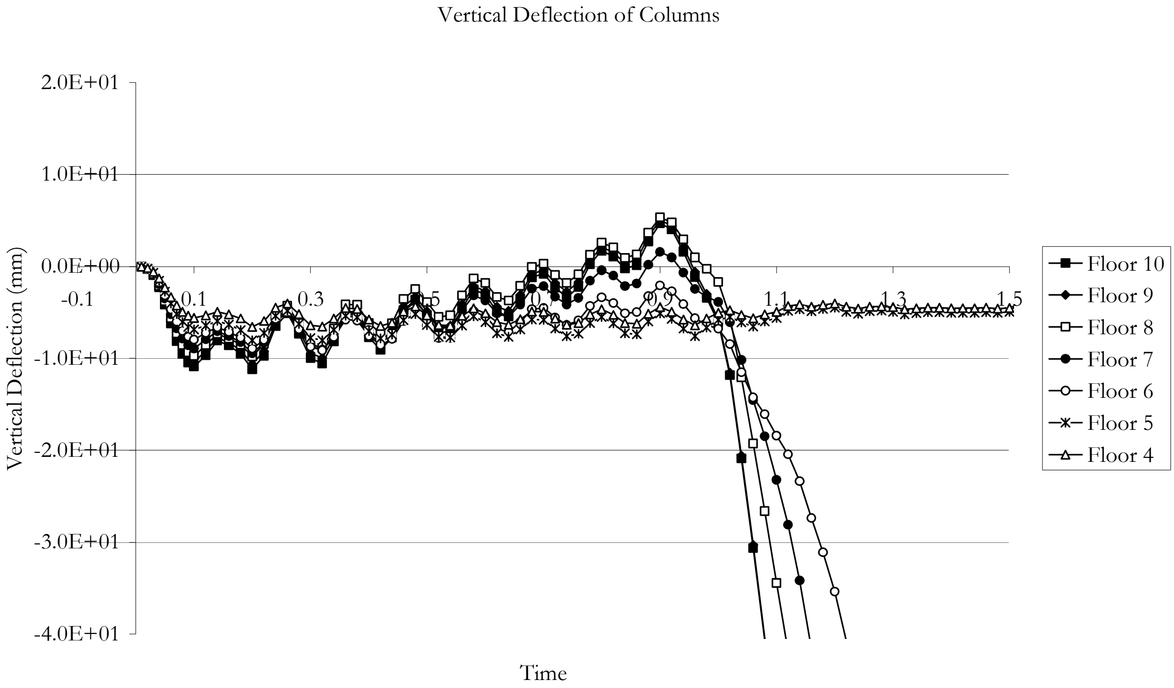


(b) Stiff floor mechanism

Fig.5. – Horizontal deflections of columns



(a) Weak floor mechanism



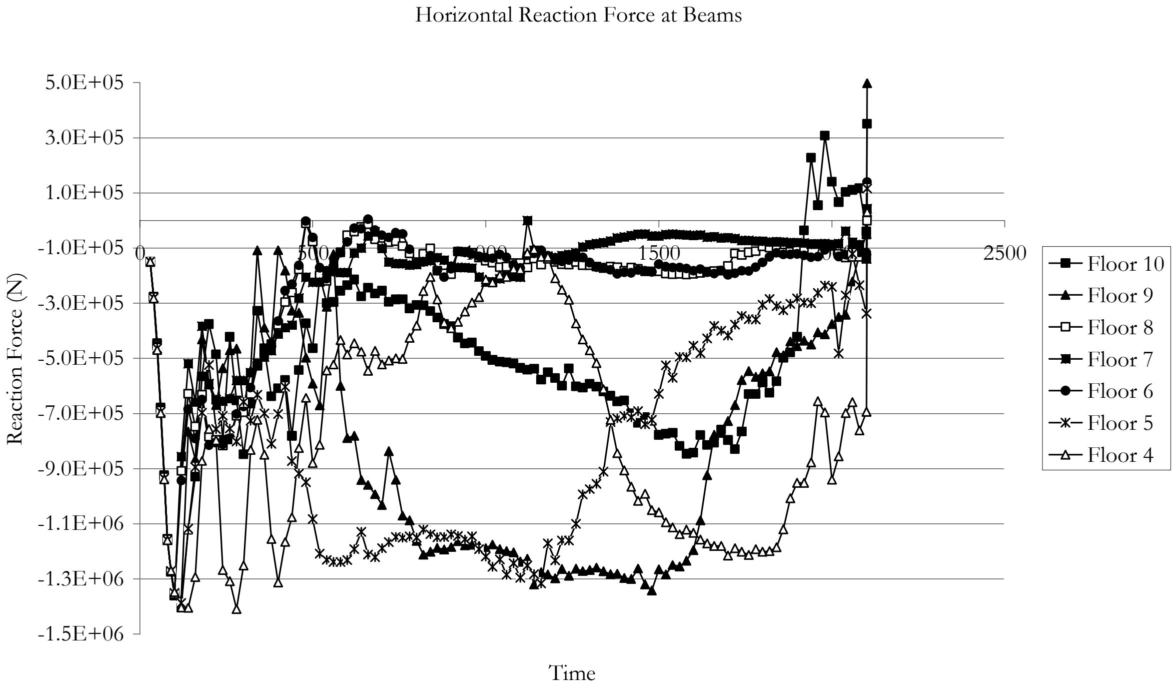
(b) Stiff floor mechanism

Fig.6 – Vertical deflection of columns

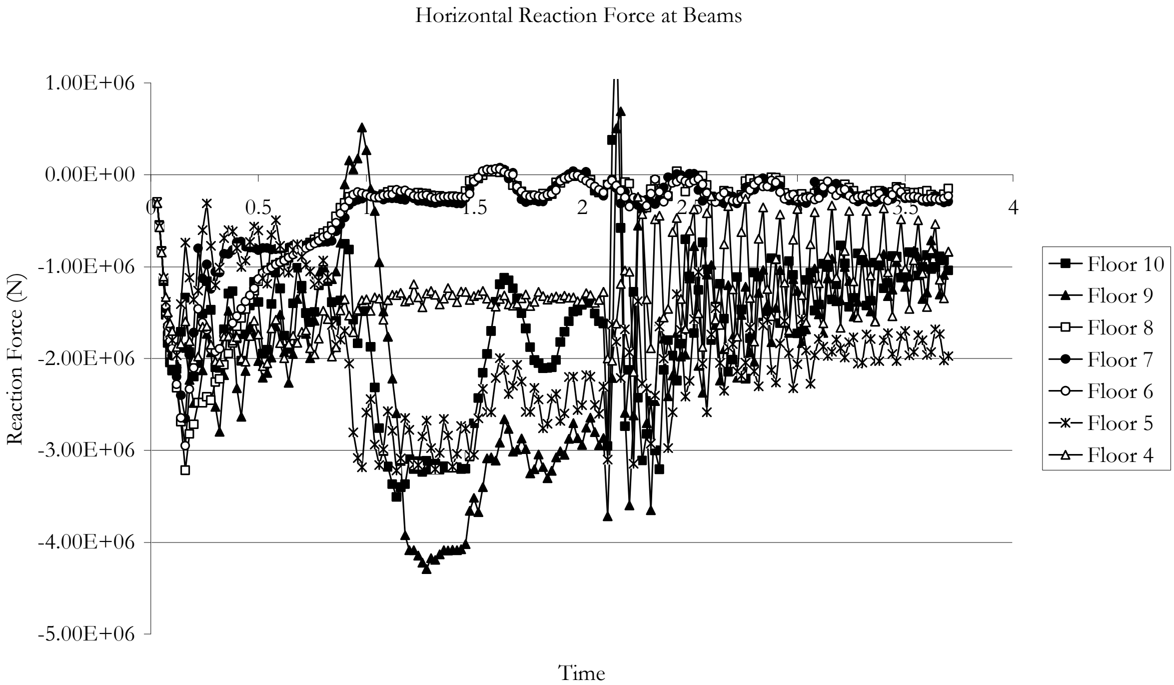
The horizontal reactions at the beam connection to the stiff core show the change in membrane forces over time in Figure 7. The weak floor model indicates that all floors go into an initial state of compression. The three fire floors rapidly reduce in compression until a very small reaction remains. All three floors have buckled at this stage. Floor 5, immediately below the fire floors, experiences an increased reaction as the floors above take a reduced amount. When floor five buckles due to the increased force from the column, the reaction quickly reduces. Now floor 4 sees a rapid increase, until this floor buckles. The progressive failure of floors is thus clearly visible from this graph.

The stiff floor system in Figure 7 (b) also starts off with an immediate compression. The three fire floors buckle and during this process the reaction force reduces. At the same time the force is being redistributed to floors 5 and 9, immediately above and below the fire floors. As these floors are relatively strong no further buckling occurs and the column forms hinges to allow for inward movement of the column due to the deformation of the beams.

Research done by Flint[3] shows several floors are in tension rather than compression. The exact reason for why the behaviour seen here is different is yet unknown.



(a) Weak floor mechanism



(b) Stiff floor mechanism

Fig.7 – Horizontal reaction forces at the beams

The section capacity of the column is shown in the interaction diagram of the loading and moments in Figure 8. This relates to the section moment for both models in Figure 9 as it shows when plastic hinges are formed. The weak floor model shows that hinges are formed at floor 5, 7 and 9. Although this is similar to the stiff floor model, the overall behaviour is

significantly different. As the hinge forms at floor 5, the moment at floor 4 increases until that too hinges. This in turn affects the column at floor 3 which also hinges soon after. This clearly indicates the progressive collapse of the floors and column.
When comparing the section moments at the column and beam connections with the section capacity of the column, hinges can be seen.

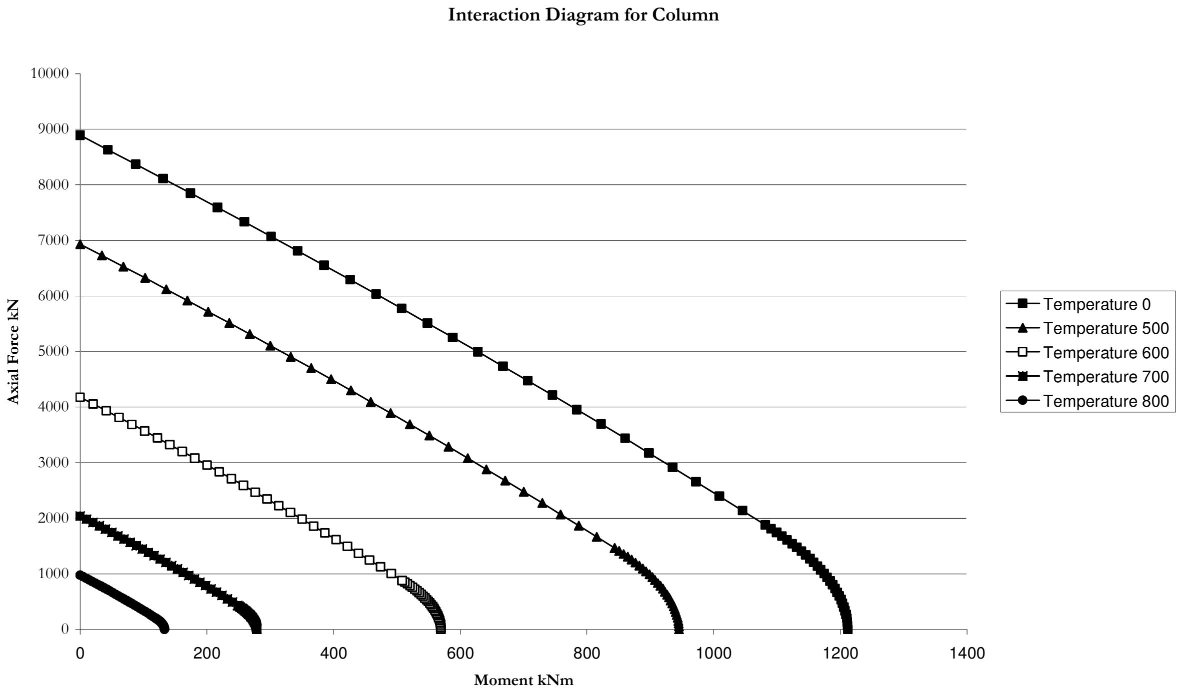
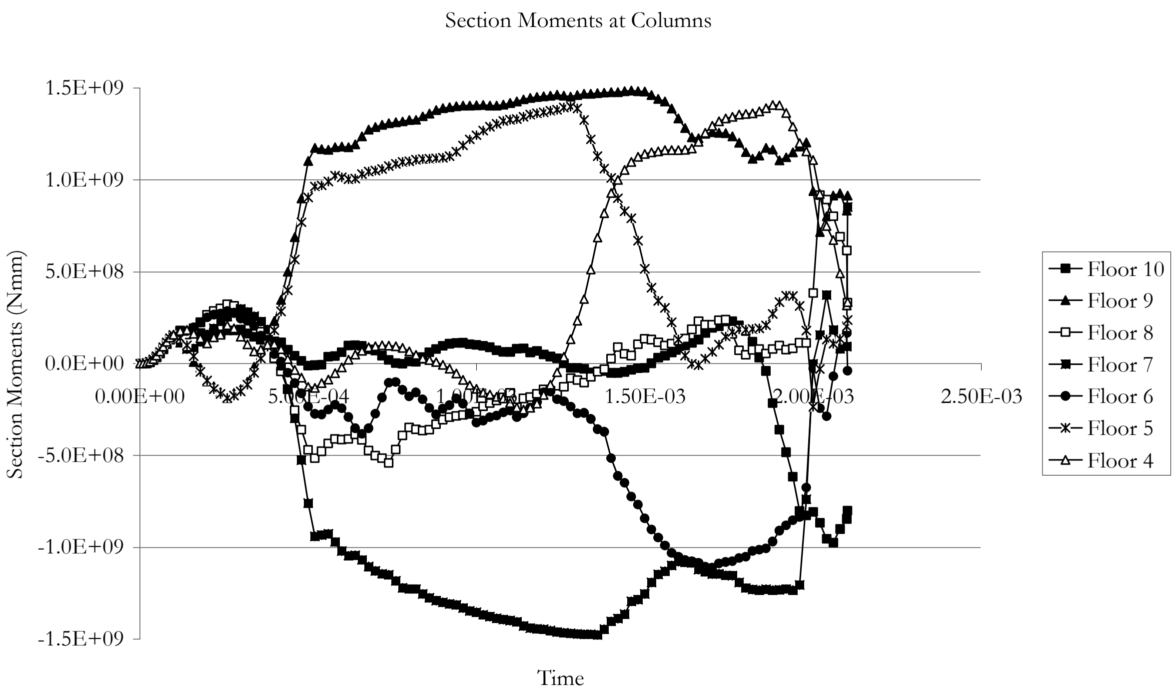
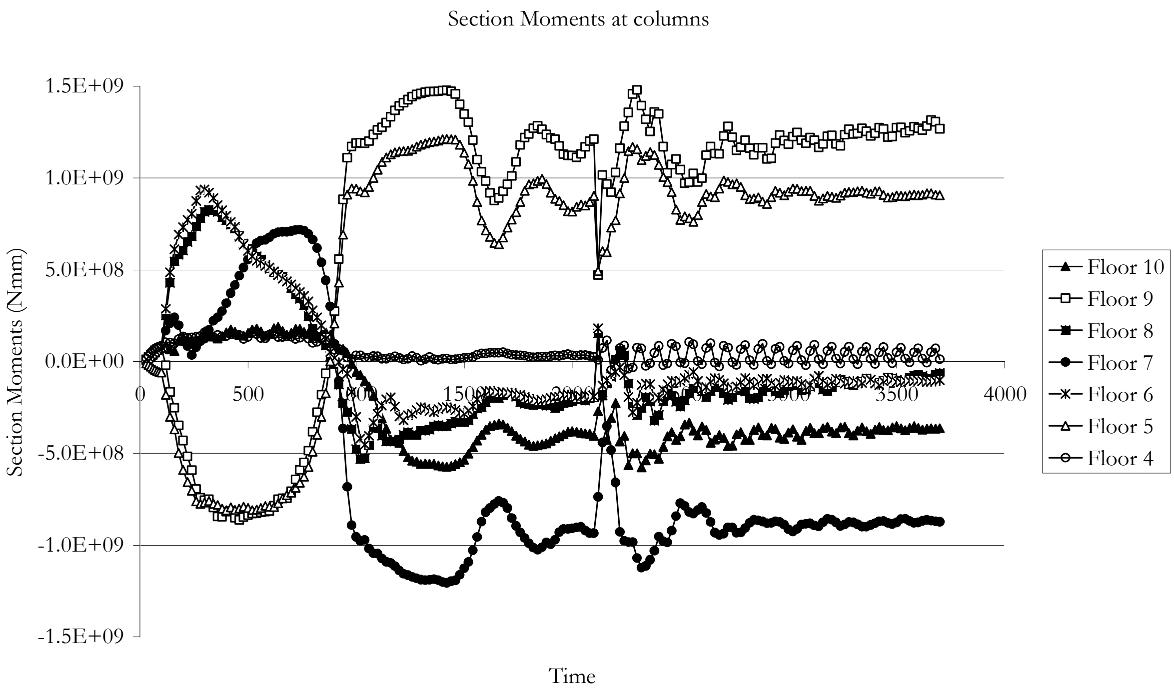


Fig.8 – Interaction Diagram for Column



(a) Weak floor mechanism



(b) Stiff floor mechanism

Fig.9 – Section moments at the column and beam connection

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) and the middle fire floor. If an approximation of the column internal displacement can be made, additional P-Δ 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.

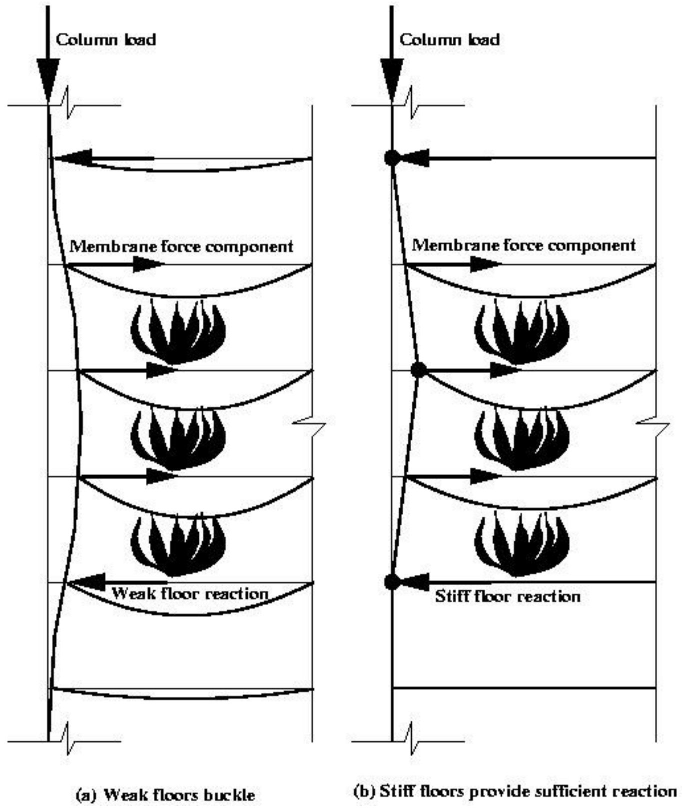


Fig.10 – Mechanics of fire induced collapse in weak and stiff floor buildings

5. CONCLUSION

This paper introduces the hypothesis of two possible failure mechanisms for tall buildings in multiple floor fires. The hypothesis is tested by creating a finite element model of a standard steel frame composite structure. The results of the modelling indicate that the two different failure mechanisms do indeed occur. 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.

6. REFERENCES

[1]. 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.

[2]. 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

[3]. G.R.Flint, *Fire Induced Collapse of Tall Buildings*, PhD thesis, University of Edinburgh, 2005. Available at http://www.civ.ed.ac.uk/research/fire/thesis.html

[4]. N.J.K.Cameron, The Behaviour and Design of Composite Floor Systems in Fire, University of Edinburgh, 2004. Available at http://www.civ.ed.ac.uk/research/fire/thesis.html